CODE REQUIREMENTS FOR ENVIRONMENTAL ENGINEERING CONCRETE STRUCTURES AND COMMENTARY (ACI 350-06)

REPORTED BY ACI COMMITTEE 350

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The code portion of this document covers the structural design, materials selection, and construction of environmental engineering concrete structures. Such structures are used for conveying, storing, or treating liquid or other materials such as solid waste. They include ancillary structures for dams, spill-ways, and channels.

They are subject to uniquely different loadings, more severe exposure conditions, and more restrictive serviceability requirements than non-environmental building structures.

Loadings include normal dead and live loads and vibrating equipment or hydrodynamic forces. Exposures include concentrated chemicals, alternate wetting and drying, and freezing and thawing of saturated concrete. Serviceability requirements include liquid-tightness or gas-tightness.

Typical structures include conveyance, storage, and treatment structures.

Proper design, materials, and construction of environmental engineering concrete structures are required to produce serviceable concrete that is dense, durable, nearly impermeable, and resistant to chemicals, with limited deflections and cracking. Leakage must be controlled to minimize contamination of ground water or the environment, to minimize loss of product or infiltration, and to promote durability.

This code presents new material as well as modified portions of the ACI 318-02 Building Code that are applicable to environmental engineering concrete structures.

Because ACI 350-06 is written as a legal document, it may be adopted by reference in a general building code or in regulations governing the design and construction of environmental engineering concrete structures. Thus, it cannot present background details or suggestions for carrying out its requirements or intent. It is the function of the commentary to fill this need.

ACI 350-06 was adopted as a standard of the American Concrete Institute on July 3, 2006 to supersede ACI 350/350R-01 in accordance with the Institute's standardization procedure.

The 2006 "Code Requirements for Environmental Engineering Concrete Structures and Commentary" are presented in a side-by-side column format, with code text placed in the left column and the corresponding commentary text aligned in the right column. To further distinguish the Code from the Commentary, the Code has been printed in Helvetica, the same type face in which this paragraph is set.

This paragraph is set in Times Roman, and all portions of the text exclusive to the Commentary are printed in this type face. Commentary section numbers are preceded by an "R" to further distinguish them from Code section numbers.

The commentary discusses some of the considerations of the committee in developing the ACI 350 Code, and its relationship with ACI 318. Emphasis is given to the explanation of provisions that may be unfamiliar to some users of the code. References to much of the research data referred to in preparing the code are given for those who wish to study certain requirements in greater detail.

The chapter and section numbering of the code are followed throughout the commentary.

Among the subjects covered are: permits, drawings and specifications, inspections, materials, concrete quality, mixing and placing, forming, embedded pipes, construction joints, reinforcement details, analysis and design, strength and serviceability, flexural and axial loads, shear and torsion, development of reinforcement, slab systems, walls, footings, precast concrete, prestressed concrete, shell structures, folded plate members, provisions for seismic design, and an alternate design method in Appendix I.

The quality and testing of materials used in the construction are covered by reference to the appropriate standard specifications. Welding of reinforcement is covered by reference to the appropriate AWS standard. Criteria for liquid-tightness testing may be found in 350.1.

Keywords: Chemical attack; coatings; concrete durability; concrete finishing (fresh concrete); concrete slabs, crack width, and spacing; cracking (fracturing); environmental engineering; inspection; joints (junctions); joint sealers; liquid; patching; permeability; pipe columns; pipes (tubes); prestressed concrete; prestressing steels; protective coatings; reservoirs; roofs; serviceability; sewerage; solid waste facilities; tanks (containers); temperature; torque; torsion; vibration; volume change; walls; wastewater treatment; water; water-cementitious material ratio; water supply; water treatment.

INTRODUCTION

The code and commentary includes excerpts from ACI 318-02 that are pertinent to ACI 350. The commentary discusses some of the considerations of Committee ACI 350 in developing "Code Requirements for Environmental Engineering Concrete Structures (ACI 350-06)," hereinafter called the code. Emphasis is given to the explanation of provisions that may be unfamiliar to users of the standard. Comments on specific provisions are made under the corresponding chapter and section numbers of the code and commentary.

This commentary is not intended to provide a complete historical background concerning the development of the code, nor is it intended to provide a detailed summary of the studies and research data reviewed by the committee in formulating the provisions of the code. However, references to some of the research data are provided for those who wish to study the background material in depth.

As the name implies, "Code Requirements for Environmental Engineering Concrete Structures" may be used as part of a legally adopted code and, as such, must differ in form and substance from documents that provide detailed specifications, recommended practice, complete design procedures, or design aids.

The code is intended to cover environmental engineering concrete structures, but is not intended to supersede ASTM standards for precast structures.

Requirements more stringent than the code provisions may be desirable for unusual structures. This code and this commentary cannot replace sound engineering knowledge, experience, and judgment.

A code for design and construction states the minimum requirements necessary to provide for public health and safety. ACI 350 is based on this principle. For any structure, the owner or the structural designer may require the quality of materials and construction to be higher than the minimum requirements necessary to provide serviceability and to protect the public as stated in the code. Lower standards, however, are not permitted.

ACI 350 has no legal status unless it is adopted by government bodies having the power to regulate building design and construction. Where the code has not been adopted, it may serve as a reference to good practice.

The code provides a means of establishing minimum standards for acceptance of design and construction by a legally appointed building official or his designated representatives. The code and commentary are not intended for use in settling disputes between the owner, engineer, architect, contractor, or their agents, subcontractors, material suppliers, or testing agencies. Therefore, the code cannot define the contract responsibility of each of the parties in usual construction. General references requiring compliance with ACI 350 in the job specifications should be avoided, since the contractor is rarely in a position to accept responsibility for design details or construction requirements that depend on a detailed knowledge of the design. Generally, the drawings, specifications, and contract documents should contain all of the necessary requirements to ensure compliance with the code. In part, this can be accomplished by reference to specific code sections in the job specifications. Other ACI publications, such as ACI 301, "Specifications for Structural Concrete," are written specifically for use as contract documents for construction.

Committee 350 recognizes the desirability of standards of performance for individual parties involved in the contract documents. Available for this purpose are the certification programs of the American Concrete Institute, the plant certification programs of the Precast/Prestressed Concrete Institute, the National Ready Mixed Concrete Association, and the qualification standards of the American Society of Concrete Constructors. Also available are "Standard Specification for Agencies Engaged in Construction Inspection and/or Testing" (ASTM E 329) and "Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation" (ASTM C 1077).

Design aids (general concrete design aids are listed in ACI 318-02):

"Rectangular Concrete Tanks," Portland Cement Association, Skokie, IL, 1994, 176 pp. (Presents data for design of rectangular tanks.)

"Circular Concrete Tanks Without Prestressing," Portland Cement Association, Skokie, IL, 1993, 54 pp. (Presents design data for circular concrete tanks built in or on ground. Walls may be free or restrained at the top. Wall bases may be fixed, hinged, or have intermediate degrees of restraint. Various layouts for circular roofs are presented.)

"Concrete Manual," U.S. Department of Interior, Bureau of Reclamation, 8th edition, 1981, 627 pp. (Presents technical information for the control of concrete construction, including linings for tunnels, impoundments, and canals.)

GENERAL COMMENTARY

Because of stringent service requirements, environmental engineering concrete structures should be designed and detailed with care. The quality of concrete is important, and close quality control must be performed during construction to obtain impervious concrete.

Environmental engineering concrete structures for the containment, treatment, or transmission of liquid such as water and wastewater as well as solid waste disposal facilities, should be designed and constructed to be essentially liquid-tight, with minimal leakage under normal service conditions.

The liquid-tightness of a structure will be reasonably assured if:

a) The concrete mixture is well proportioned, well consolidated without segregation, and properly cured.
b) Crack widths and depths are minimized.
c) Joints are properly spaced, sized, designed, waterstopped, and constructed.
d) Adequate reinforcing steel is provided, properly detailed, fabricated, and placed.
e) Impervious protective coatings or barriers are used where required.

Usually it is more economical and dependable to resist liquid permeation through the use of quality concrete, proper design of joint details, and adequate reinforcement, rather than by means of an impervious protective barrier or coating. Liquid-tightness can also be obtained by appropriate use of shrinkage-compensating concrete. However, to achieve success, the engineer must recognize and account for the limitations, characteristics, and properties of shrinkage-compensating concrete as described in ACI 223 and ACI 224.2R.

Minimum permeability of the concrete will be obtained by using water-cementitious materials ratios as low as possible, consistent with satisfactory workability and consolidation. Impermeability increases with the age of the concrete and is improved by extended periods of moist curing. Surface treatment is important and use of smooth forms or troweling improves impermeability. Air entrainment reduces segregation and bleeding, increases workability, and provides resistance to the effect of freeze-thaw cycles. Because of this, use of an air-entraining admixture results in better consolidated concrete. Other admixtures, such as water-reducing agents and pozzolans, are useful when they lead to increased workability and consolidation, and lower water-cementitious ratios. Pozzolans also reduce permeability.

Joint design should also account for movement resulting from thermal dimensional changes and differential settlements. Joints permitting movement along predetermined control planes, and which form a barrier to the passage of fluids, shall include waterstops in complete, closed circuits. Proper rate of concrete placement operations, adequate consolidation, and proper curing are also essential to control of cracking in environmental engineering concrete structures. Additional information on cracking is contained in ACI 224R and ACI 224.2R.

The design of the whole environmental engineering concrete structure as well as all individual members should be in accordance with ACI 350-06, which has been adapted from ACI 318-02. When all relevant loading conditions are considered, the design should provide adequate safety and serviceability, with a life expectancy of 50 to 60 years for the structural concrete. Some components of the structure, such as jointing materials, have a shorter life expectancy and will require maintenance or replacement.

The size of elements and amount of reinforcement should be selected on the basis of the serviceability crack-width limits and stress limits to promote long service life.

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PART 1 — GENERAL

CHAPTER 1 — GENERAL REQUIREMENTS

CODE

1.1 — Scope

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R1.1 — Scope

The American Concrete Institute **"Code Requirements for Environmental Engineering Concrete Structures (ACI 350-06),"** hereinafter referred to as the code, provide minimum requirements for environmental engineering concrete structural design and construction practices.

The 2006 edition of the code revised the previous code, "**Code Requirements of Environmental Engineering Concrete Structures (ACI 350-01)**." This code includes in one document the rules for all reinforced concrete used for environmental engineering structural purposes. This covers the spectrum of structural applications of concrete containing nonprestressed reinforcement, prestressing steel, or composite steel shapes, pipe, or tubing.

Prestressed concrete is included under the definition of reinforced concrete. Provisions of ACI 350-06 apply to prestressed concrete except in cases in which the provisions of the code are stated to apply specifically to nonprestressed concrete.

Chapter 21 of the code contains special provisions for design and detailing of earthquake-resistant structures. See 1.1.8.

Appendix I of the 2006 code, formerly Appendix A of the 2001 code, contains provisions for an alternate method of design for nonprestressed reinforced concrete members using service loads (without load factors) and permissible service load stresses. The strength design method of this code is intended to give design results similar to the Alternate Design Method.

Appendix A of the ACI 318-02 code has not yet been adopted for environmental engineering concrete structures. Applicability of strut-and-tie models to environmental structures may be addressed in future revisions to ACI 350.

Appendix B of the 2006 code contains provisions for reinforcement limits based on $0.75\rho_b$, determination of the strength reduction factor ϕ, and moment redistribution that have been in the 318 codes for many years, including the 1999 318 code. The provisions are applicable to reinforced and prestressed concrete members. Designs made using the provisions of Appendix B are used in their entirety.

Appendix C of the 2006 code allows the use of load, environmental durability, strength reduction factors, and flexural reinforcement distribution provisions similar to those In Chapters 9 and 10 of ACI 350-01. Designs made using the provisions of Appendix C are equally acceptable as those

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based on the body of the code, provided the provisions of Appendix C are used in their entirety.

Appendix D contains provisions for anchoring to concrete.

1.1.1 — Except for primary containment of hazardous materials, this code provides minimum requirements for the design and construction of reinforced concrete structural elements of any environmental engineering concrete structure erected under the requirements of the legally adopted building code where this code has been adopted to be a part of such code. In areas without a legally adopted building code, this code defines minimum acceptable standards of design and construction practice.

For structural concrete, the specified concrete strength shall not be less than 4000 psi. No maximum specified compressive strength shall apply unless restricted by a specific code provision.

1.1.1.1 — Environmental engineering concrete structures are defined as concrete structures intended for conveying, storing, or treating water, wastewater, or other liquids and non-hazardous materials such as solid waste, and for secondary containment of hazardous liquids or solid waste. Ancillary structures for which liquid-tightness, gas-tightness, or enhanced durability are essential design considerations shall also conform to requirements of environmental engineering concrete structures. Precast concrete environmental structures designed and constructed in accordance with ASTM or AWWA standards, with the exception of circular tanks, are not covered in this code.

R1.1.1 — A hazardous material is defined as having one or more of the following characteristics: ignitable (NFPA 49), corrosive, reactive, or toxic. The Environmental Protection Agency (EPA)-listed wastes are organized into three categories under RCRA: source specific wastes, generic wastes, and commercial chemical products. Source specific wastes include sludges and wastewaters from treatment and production processes in specific industries such as petroleum refining and wood preserving. The list of generic wastes includes wastes from common manufacturing and industrial processes such as solvents used in degreasing operations. The third list contains specific chemical products such as benzine, creosote, mercury, and various pesticides.

Below-grade structures, such as pump stations and pipe galleries, which are part of treatment facilities and which may be exposed to external groundwater pressures, generally are designed as environmental concrete structures. Above-grade building structures that are not directly exposed to liquids, solid wastes, corrosive chemicals, corrosive gases, or high humidity associated with treatment facilities generally may be designed in accordance with the general building code or applicable industry standards. Nevertheless, consideration of corrosive effects on such structures may still be advisable.

1.1.2 — This code supplements the general building code and shall govern in all matters pertaining to design and construction of reinforced concrete structural elements of any environmental engineering concrete structure, except wherever this code is in conflict with requirements in legally-adopted applicable codes addressing environmental engineering concrete structures.

R1.1.2 — The American Concrete Institute recommends that the code be adopted in its entirety; however, it is recognized that when the code is made a part of a legally adopted general building code, that general building code may modify some provisions of this code.

1.1.3 — This code shall apply in all matters pertaining to design, construction, and material properties wherever this code is in conflict with requirements contained in other standards referenced in this code.

1.1.4 — The provisions of this code shall govern for tanks, reservoirs, and other reinforced concrete elements of any environmental engineering concrete structure. Special structures such as arches, bins and silos, blast-resistant structures, and chimneys are not covered in this code.

R1.1.4 — Environmental engineering projects can contain several types of structures. For example, a treatment plant can contain environmental engineering concrete structures such as tanks and reservoirs, as well and building structures. The ACI 350 code would apply to the environmental structures, while the ACI 318 code or the following ACI publications could apply to the other structures.

"Design and Construction of Reinforced Concrete Chimneys" reported by ACI Committee 307.[1.1] (Gives

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material, construction, and design requirements for circular cast-in-place reinforced chimneys. It sets forth minimum loadings for the design of reinforced concrete chimneys and contains methods for determining the stresses in the concrete and reinforcement required as a result of these loadings.)

"Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials" reported by ACI Committee 313.[1.2] (Gives material, design, and construction requirements for reinforced concrete bins, silos, and bunkers and stave silos for storing granular materials. It includes recommended design and construction criteria based on experimental and analytical studies plus worldwide experience in silo design and construction.)

(Bins, silos, and bunkers are special structures, posing special problems not encountered in normal building design. While this standard practice refers to **"Building Code Requirements for Structural Concrete"** (ACI 318) for many applicable requirements, it provides supplemental detail requirements and ways of considering the unique problems of static and dynamic loading of silo structures. Much of the method is empirical, but this standard practice does not preclude the use of more sophisticated methods that give equivalent or better safety and reliability.)

(This standard practice sets forth recommended loadings and methods for determining the stresses in the concrete and reinforcement resulting from these loadings. Methods are recommended for determining the thermal effects resulting from stored material and for determining crack width in concrete walls due to pressure exerted by the stored material. Appendices provide recommended minimum values of overpressure and impact factors.)

"Code Requirements for Nuclear Safety Related Concrete Structures" reported by ACI Committee 349.[1.3] (Provides minimum requirements for design and construction of concrete structures that form part of a nuclear power plant and which have nuclear safety related functions. The code does not cover concrete reactor vessels and concrete containment structures that are covered by ACI 359.)

"Code for Concrete Reactor Vessels and Containments" reported by Joint ACI-ASME Committee 359.[1.4] (Provides requirements for the design, construction, and use of concrete reactor vessels and concrete containment structures for nuclear power plants.)

1.1.5 — This code does not govern design and installation of portions of concrete piles and drilled piers embedded in ground except for structures in regions of high seismic risk or assigned to high seismic performance or design categories. See 21.10.4 for requirements for concrete piles, drilled piers, and caissons in structures in regions of high seismic risk or assigned to high seismic performance or design categories.

R1.1.5 — The design and installation of piling fully embedded in the ground is regulated by the general building code. For portions of piling in air or water, or in soil not capable of providing adequate lateral restraint throughout the piling length to prevent buckling, the design provisions of this code govern where applicable.

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Recommendations for concrete piles are given in detail in **"Design, Manufacture, and Installation of Concrete Piles"** reported by ACI Committee 543.[1.5] (Provides recommendations for the design and use of most types of concrete piles for many kinds of construction.)

Recommendations for drilled piers are given in detail in **"Design and Construction of Drilled Piers"** reported by ACI Committee 336.[1.6] (Provides recommendations for design and construction of foundation piers 2-1/2 ft in diameter or larger made by excavating a hole in the soil and then filling it with concrete.)

Detailed recommendations for precast, prestressed concrete piles are given in **"Recommended Practice for Design, Manufacture, and Installation of Prestressed Concrete Piling"** prepared by the PCI Committee on Prestressed Concrete Piling.[1.7]

1.1.6 — This code governs the design and construction of soil-supported slabs as required by Appendix H. Slabs that transmit vertical loads from other portions of the structure to the soil shall meet the requirements of other chapters of this code as applicable.

R1.1.6 — Since tank floor slabs frequently directly transfer the loads from liquid contents to the soil below, Appendix H has been added to this code to provide appropriate requirements.

1.1.7 — Concrete on steel form deck

R1.1.7 — Concrete on steel form deck

In steel-framed structures, it is common practice to cast concrete floor slabs on stay-in-place steel form deck. In all cases, the deck serves as the form and may, in some cases, serve an additional structural function.

1.1.7.1 — Design and construction of structural concrete slabs cast on stay-in-place, noncomposite steel form deck are governed by this code.

R1.1.7.1 — In its most basic application, the steel form deck serves as a form, and the concrete serves a structural function and, therefore, must be designed to carry all superimposed loads.

1.1.7.2 — This code does not govern the design of structural concrete slabs cast on stay-in-place, composite steel form deck. Concrete used in the construction of such slabs shall be governed by Parts 1, 2, and 3 of this code, where applicable.

R1.1.7.2 — Another type of steel form deck commonly used develops composite action between the concrete and steel deck. In this type of construction, the steel deck serves as the positive moment reinforcement. The design of composite slabs on steel deck is regulated by **"Standard for the Structural Design of Composite Slabs"** (ANSI/ASCE 3).[1.8] However, ANSI/ASCE 3 references the appropriate portions of ACI 318 for the design and construction of the concrete portion of the composite assembly. Guidelines for the construction of composite steel deck slabs are given in **"Standard Practice for the Construction and Inspection of Composite Slabs"** (ANSI/ASCE 9).[1.9]

1.1.8 — Special provisions for earthquake resistance

R1.1.8 — Special provisions for earthquake resistance

Special provisions for seismic design were first introduced in Appendix A of the ACI 318-71 Building Code and were continued without revision in ACI 318-77. These provisions were originally intended to apply only to reinforced concrete structures located in regions of highest seismicity.

The special provisions were extensively revised in the ACI 318-83 code edition to include new requirements for certain

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1.1.8.1 — In all regions of seismic risk, and all seismic performance or design categories, provisions of Chapter 21 shall be satisfied as described in 21.2.1.

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earthquake-resisting systems located in regions of moderate seismicity. In the 318-89 code, the special provisions were moved to Chapter 21.

R1.1.8.1 — Some structures and elements of structures will have their design governed by hydrodynamic forces, even when located in areas of low seismic risk, due to their configuration and position. Portions of Chapter 21 (21.2, 21.7, 21.8, and 21.9) apply to liquid-containing structures for all levels of seismic risk.

Aside from provisions given in 21.2, 21.7, 21.8, and 21.9, for structures located in regions of low seismic risk, or for structures assigned to low seismic performance or design categories, no special design or detailing is required; the general requirements of the main body of the code apply for proportioning and detailing environmental engineering concrete structures. It is the intent of this code that concrete structures proportioned by Chapters 1 to 18 of this code and the provisions given in 21.2, 21.7, 21.8, and 21.9 will provide a level of toughness adequate for low earthquake intensity.

For structures in regions of moderate seismic risk, or for structures assigned to intermediate seismic performance or design categories, reinforced concrete moment frames proportioned to resist earthquake effects require some special reinforcement details, as specified in 21.12. The special details apply only to beams, columns, and slabs to which the earthquake-induced forces have been assigned in design. The special reinforcement details will serve to provide a suitable level of inelastic behavior if the frame is subjected to an earthquake of such intensity as to require it to perform inelastically. There are no Chapter 21 requirements for cast-in-place structural walls provided to resist seismic effects, or for other structural components that are not part of the lateral-force-resisting system of structures in regions of moderate seismic risk, or assigned to intermediate seismic performance or design categories. For precast wall panels designed to resist forces induced by earthquake motions, special requirements are specified in 21.13 for connections between panels or between panels and the foundation. Cast-in-place structural walls proportioned to meet provisions of Chapters 1 through 18 and Chapter 21 are considered to have sufficient toughness at anticipated drift levels for these structures.

For structures located in regions of high seismic risk, all structure components, structural and nonstructural, should satisfy requirements of 21.2 through 21.10. In addition, frame members that are not assumed in the design to be part of the lateral-force-resisting system should comply with 21.11. The special proportioning and detailing provisions of Chapter 21 are intended to provide a monolithic reinforced concrete structure with adequate toughness to respond inelastically under severe earthquake motions. See also R21.2.1.

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1.1.8.2 — Seismic risk level of a region, or seismic performance or design category of a structure, shall be regulated by the legally adopted general building code of which this code forms a part, or determined by local authority.

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R1.1.8.2 — Seismic risk levels (seismic zone maps) are under the jurisdiction of a general building code rather than ACI 350. This edition of ACI 350 adopts the changes in terminology made to the 1999 and 2002 editions of the 318 code to make it compatible with the latest editions of model building codes in use in the United States. For example, the phrase "seismic performance or design categories" was introduced. Over the past decade, the manner in which seismic risk levels have been expressed in United States building codes has changed. Previously, they have been represented in terms of seismic zones. Recent editions of the "BOCA National Building Code" (NBC)[1.10] and "Standard Building Code" (SBC),[1.11] which are based on the 1991 NEHRP,[1.12] have expressed risk not only as a function of expected intensity of ground shaking on solid rock, but also on the nature of the occupancy and use of the structure. These two items are considered in assigning the structure to a seismic performance category (SPC), which in turn is used to trigger different levels of detailing requirements for the structure. The 2000 "International Building Code" (IBC)[1.13] also uses the two criteria of the NBC and SBC and also considers the effects of soil amplification on the ground motion when assigning a seismic risk. Under the IBC, each structure is assigned a seismic design category (SDC). Among its several uses, it triggers different levels of detailing requirements. Table R1.1.8.2 correlates low, moderate/intermediate, and high seismic risk, which has been the terminology used in the 318 code for several editions, to the various methods of assigning risk in use in the U.S. under the various model building codes, the ASCE 7 standard, and the NEHRP Recommended Provisions.

In the absence of a general building code that addresses earthquake loads and seismic zoning, it is the intent of Committee 350 that the local authorities (engineers, geologists, and building code officials) should decide on proper need and proper application of the special provisions for seismic design. Seismic ground-motion maps or zoning

TABLE R1.1.8.2—CORRELATION BETWEEN SEISMIC-RELATED TERMINOLOGY IN MODEL CODES

Code, standard, or resource document and edition	Level of seismic risk or assigned seismic performance or design categories as defined in the code section		
	Low (21.2.1.2)	Moderate/ intermediate (21.2.1.3)	High (21.2.1.4)
International Building Code 2000; NEHRP 1997	SDC[1] A, B	SDC C	SDC D, E, F
BOCA National Building Code 1993, 1996, 1999; Standard Building Code 1994, 1997, 1999; ASCE 7-93, 7-95, 7-98; NEHRP 1991, 1994	SPC[2] A, B	SPC C	SPC D, E
Uniform Building Code 1991, 1994, 1997	Seismic Zone 0, 1	Seismic Zone 2	Seismic Zone 3, 4

[1]SDC = *Seismic design category* as defined in code, standard, or resource document.
[2]SPC = *Seismic performance category* as defined in code, standard, or resource document.

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1.1.9 — For prestressed concrete environmental structures, Chapters 1 through 21 cover prestressing in general. Chapters 1 through 21 plus Appendix G cover the use of circular wire and strand wrapped prestressed concrete environmental structures.

1.2 — Drawings and specifications

1.2.1 — Copies of design drawings, typical details, and specifications for all structural concrete construction shall bear the seal of a licensed engineer or architect. These drawings, details, and specifications shall show:

(a) Name and date of issue of the applicable building code and supplement to which design conforms;

(b) Live load and other loads used in design;

(c) Specified compressive strength of concrete at stated ages or stages of construction for which each part of structure is designed;

(d) Specified strength or grade of reinforcement;

(e) Size and location of all structural elements and reinforcement and anchors;

(f) Provision for dimensional changes resulting from creep, shrinkage, and change in temperature;

(g) Magnitude and location of prestressing forces;

(h) Anchorage length of reinforcement and location and length of lap splices;

(i) Type and location of welded splices and mechanical connections of reinforcement;

(j) The design liquid level for any structure designed to contain liquid;

(k) Stressing sequence for post-tensioning tendons;

(l) Statement if slab on grade is designed as a structural diaphragm, see 21.10.3.4;

(m) Design gas pressure for structural elements subjected to pressurized gas or liquid;

(n) Concrete properties and ingredients including type of cement, water-cementitious materials ratio, and, if allowed, admixtures, additives, and pozzolans;

(o) Additional requirements, such as limitations on drying shrinkage;

(p) Requirements for liquid-tightness testing, including liquid-tightness testing before backfilling.

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maps, such as recommended in References 1.9, 1.14, and 1.15, are suitable for correlating seismic risk.

R1.1.9 — Appendix G is incorporated to address those aspects of circular wrapped prestressed concrete environmental structures that are not directly covered within the main body of the code. Thus, Appendix G deals with items that are unique to circular wrapped prestressed structures, such as steel diaphragm, wrapped prestressing, and shotcrete.

R1.2 — Drawings and specifications

R1.2.1 — The provisions for preparation of design drawings and specifications are, in general, consistent with those of most general building codes and are intended as supplements.

The code lists some of the more important items of information that must be included in the design drawings, details, or specifications. The code does not imply an all inclusive list, and additional items may be required by the building official.

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1.2.2 — Calculations pertinent to design shall be filed with the drawings when required by the building official. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. Model analysis shall be permitted to supplement calculations.

1.2.3 — Building official means the officer or other designated authority charged with the administration and enforcement of this code, or his duly authorized representative.

1.3 — Inspection

1.3.1 — Concrete construction shall be inspected as required by the legally adopted general building code. In the absence of such requirements, concrete construction shall be inspected throughout the various work stages by or under the supervision of a licensed design professional or by a qualified inspector.

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R1.2.2 — Documented computer output is acceptable in lieu of manual calculations. The extent of input and output information required will vary, according to the specific requirements of individual building officials. When a computer program has been used by the designer, however, only skeleton data should normally be required. This should consist of sufficient input and output data and other information to allow the building official to perform a detailed review and make comparisons using another program or manual calculations. Input data should be identified as to member designation, applied loads, and span lengths. The related output data should include member designation and the shears, moments, and reactions at key points in the span. For column design, it is desirable to include moment magnification factors in the output where applicable.

The code permits model analysis to be used to supplement structural analysis and design calculations. Documentation of the model analysis should be provided with the related calculations. Model analysis should be performed by an engineer or architect having experience in this technique.

R1.2.3 — "Building official" is the term used by many general building codes to identify the person charged with administration and enforcement of the provisions of the building code. Such terms as "building commissioner" or "building inspector," however, are variations of the title, and the term "building official" as used in this code is intended to include those variations as well as others that are used in the same sense.

R1.3 — Inspection

The quality of concrete structures depends largely on workmanship in construction. The best of materials and design practice will not be effective unless the construction is performed well. Inspection is provided to assure satisfactory work in accordance with the design drawings and specifications. Proper performance of the structure depends on construction that accurately represents the design and meets code requirements within the tolerances allowed. In the public interest, local building ordinances should require the owner to provide inspections.

R1.3.1—Inspection of construction by or under the supervision of the licensed design professional responsible for the design should be recommended because the person in charge of the design is the best qualified to inspect for conformance with the design. When such an arrangement is not feasible, inspection of construction through other licensed design professionals or through separate accredited inspection organizations, with demonstrated capability for performing the inspection, may be used.

Qualified inspectors should establish their qualifications by becoming certified to inspect and record the results of concrete construction, including preplacement, placement, and postplacement operations through the Reinforced

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1.3.2 — The inspector shall require compliance with design drawings and specifications. Unless specified otherwise in the legally adopted general building code, inspection records shall include:

(a) Quality and proportions of concrete materials and strength of concrete;

(b) Construction and removal of forms and reshoring;

(c) Placing of reinforcement and anchors;

(d) Mixing, placing, and curing of concrete;

(e) Sequence of erection and connection of precast members;

(f) Tensioning of tendons;

(g) Any significant construction loadings on completed floors, members, or walls;

(h) Description and results of tightness testing of liquid and/or gas-containing structures;

(i) General progress of work.

COMMENTARY

Concrete Special Inspection program sponsored by ACI, ICBO, BOCA, and SBCCI or equivalent.

When inspection is done independently of the licensed design professional responsible for the design, it is recommended that the licensed design professional responsible for the design be employed to oversee inspection and observe the work to see that his design requirements are properly executed.

In some jurisdictions, legislation has established special registration or licensing procedures for persons performing certain inspection functions. A check should be made in the general building code or with the building official to ascertain if any such requirements exist within a specific jurisdiction.

Inspection reports should be promptly distributed to the owner, licensed design professional responsible for the design, contractor, appropriate subcontractors, appropriate suppliers, and the building official to allow timely identification of compliance or the need for corrective action.

Inspection responsibility and the degree of inspection required should be set forth in the contracts between the owner, architect, engineer, and contractor. Adequate fees should be provided consistent with the work and equipment necessary to properly perform the inspection.

R1.3.2 — By “inspection,” the code does not mean that the inspector should supervise the construction. Rather, it means that the one employed for inspection should visit the project with the frequency necessary to observe the various stages of work and ascertain that it is being done in compliance with contract documents and code requirements. The frequency should be at least enough to provide general knowledge of each operation, whether this is several times a day or once in several days.

Inspection in no way relieves the contractor from his obligation to follow the plans and specifications implicitly and to provide the designated quality and quantity of materials and workmanship for all job stages. The inspector should be present as frequently as he/she deems necessary to judge whether the quality and quantity of the work complies with the contract documents; to counsel on possible ways of obtaining the desired results; to see that the general system proposed for formwork appears proper (though it remains the contractor’s responsibility to design and build adequate forms and to leave them in place until it is safe to remove them); to see that reinforcement is properly installed; to see that concrete is of the correct quality, properly placed, and cured; and to see that tests for quality control are being made as specified.

The code prescribes minimum requirements for inspection of all structures within its scope. It is not a construction specification and any user of the code may require higher standards of inspection than cited in the legal code if additional requirements are necessary.

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Recommended procedures for organization and conduct of concrete inspection are given in detail in **"Guide for Concrete Inspection."**[1.16] (Sets forth procedures relating to concrete construction to serve as a guide to owners, architects, and engineers in planning an inspection program.)

Detailed methods of inspecting concrete construction are given in **"ACI Manual of Concrete Inspection"** (SP-2) reported by ACI Committee 311.[1.17] (Describes methods of inspecting concrete construction that are generally accepted as good practice. Intended as a supplement to specifications and as a guide in matters not covered by specifications.)

ACI 311.5 provides the "Guide for Concrete Plant Inspection and Testing of Ready-Mixed Concrete."[1.18]

1.3.3 — When the ambient temperature falls below 40 °F or rises above 95 °F, a record shall be kept of concrete temperatures and of protection given to concrete during placement and curing.

R1.3.3 — The term "ambient temperature" means the temperature of the environment to which the concrete is directly exposed. Concrete temperature as used in this section may be taken as the air temperature near the surface of the concrete; however, during mixing and placing, it is practical to measure the temperature of the mixture.

1.3.4 — Records of inspection required in 1.3.2 and 1.3.3 shall be preserved by the inspecting engineer or architect for 2 years after completion of the project.

R1.3.4 — A record of inspection in the form of a job diary is required in case questions subsequently arise concerning the performance or safety of the structure or members. Photographs documenting job progress may also be desirable.

Records of inspection must be preserved for at least 2 years after the completion of the project. The completion of the project is the date at which the owner accepts the project, or when a certificate of occupancy is issued, whichever date is later. The general building code or other legal requirements may require a longer preservation of such records.

1.3.5 — For special moment frames resisting seismic loads in regions of high seismic risk, or in structures assigned to high seismic performance or design categories, continuous inspection of the placement of the reinforcement and concrete shall be made by a qualified inspector. The inspector shall be under the supervision of the engineer responsible for the structural design or under the supervision of an engineer with demonstrated capability for supervising inspection of special moment frames resisting seismic loads in regions of high seismic risk, or in structures assigned to high seismic performance or design categories.

R1.3.5 — The purpose of this section is to ensure that the special detailing required in concrete ductile frames is properly executed through inspection by personnel who are qualified to do this work. Qualifications of inspectors should be determined by the jurisdiction enforcing the general building code.

1.4 — Approval of special systems of design or construction

Sponsors of any system of design or construction within the scope of this code, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by this code, shall have the right to present the data on which their design is based to the building official or to a board of examiners appointed by the building official.

R1.4 — Approval of special systems of design or construction

New methods of design, new materials, and new uses of materials must undergo a period of development before being specifically covered in a code. Hence, good systems or components might be excluded from use by implication if means were not available to obtain acceptance.

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This board shall be composed of competent engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of this code. These rules when approved by the building official and promulgated shall be of the same force and effect as the provisions of this code.

COMMENTARY

For special systems considered under this section, specific tests, load factors, deflection limits, and other pertinent requirements should be set by the board of examiners, and should be consistent with the intent of the code.

The provisions of this section do not apply to model tests used to supplement calculations under 1.2.2 or to strength evaluation of existing structures under Chapter 20.

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COMMENTARY

Notes

CHAPTER 2 — DEFINITIONS

CODE

2.1 — The following terms are defined for general use in this code. Specialized definitions appear in individual chapters.

COMMENTARY

R2.1 — For consistent application of the code, it is necessary that terms be defined where they have particular meanings in the code. The definitions given are for use in application of this code only and do not always correspond to ordinary usage. A glossary of most used terms relating to cement manufacturing, concrete design and construction, and research in concrete is contained in **"Cement and Concrete Terminology"** reported by ACI Committee 116.[2.1]

By code definition, "sand-lightweight concrete" is structural lightweight concrete with *all* of the fine aggregate replaced by sand. This definition may not be in agreement with usage by some material suppliers or contractors where the majority, but not all, of the lightweight fines are replaced by sand. For proper application of the code provisions, the replacement limits must be stated, with interpolation when partial sand replacement is used.

Deformed reinforcement is defined as that meeting the deformed bar specifications of 3.5.3.1, or the specifications of 3.5.3.3, 3.5.3.4, 3.5.3.5, or 3.5.3.6. No other bar or fabric qualifies. This definition permits accurate statement of anchorage lengths. Bars or wire not meeting the deformation requirements or fabric not meeting the spacing requirements are "plain reinforcement," for code purposes, and may be used only for spirals.

A number of definitions for loads are given as the code contains requirements that must be met at various load levels. The terms "dead load" and "live load" refer to the unfactored loads (service loads) specified or defined by the general building code. Service loads (loads without load factors) are to be used where specified in the code to proportion or investigate members for adequate serviceability as in 9.5, Control of Deflections. Loads used to proportion a member for adequate strength are defined as "factored loads." Factored loads are service loads multiplied by the appropriate load factors specified in 9.2 for required strength. The term "design loads," as used in the ACI 318-71 code to refer to loads multiplied by appropriate load factors, was discontinued in the ACI 318-77 code to avoid confusion with the design load terminology used in general building codes to denote service loads or posted loads in buildings. The factored load terminology, first adopted in the ACI 318-77 code, clarifies when the load factors are applied to a particular load, moment, or shear value as used in the code provisions.

Reinforced concrete is defined to include prestressed concrete. Although the behavior of a prestressed member with unbonded tendons may vary from that of members with continuously bonded tendons, bonded and unbonded prestressed concrete are combined with conventionally reinforced concrete under the generic term "reinforced

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concrete." Provisions common to both prestressed and conventionally reinforced concrete are integrated to avoid overlapping and conflicting provisions.

Strength of a member or cross section calculated using standard assumptions and strength equations, and nominal (specified) values of material strengths and dimensions is referred to as "nominal strength." The subscript n is used to denote the nominal strengths; nominal axial load strength P_n, nominal moment strength M_n, and nominal shear strength V_n. "Design strength" or usable strength of a member or cross section is the nominal strength reduced by the strength reduction factor ϕ.

The required axial load, moment, and shear strengths used to proportion members are referred to either as factored axial loads, factored moments, and factored shears, or required axial loads, moments, and shears. The factored load effects are calculated from the applied factored loads and forces in such load combinations as are stipulated in the code (see 9.2).

The subscript u is used only to denote the required strengths: required axial load strength P_u, required moment strength M_u, and required shear strength V_u, calculated from the applied factored loads and forces.

The basic requirement for strength design may be expressed as follows:

Design strength ≥ Required strength

$$\phi P_n \geq P_u$$

$$\phi M_n \geq M_u$$

$$\phi V_n \geq V_u$$

For additional discussion on the concepts and nomenclature for strength design, see Commentary, Chapter 9.

The term "compression member" is used in the code to define any member in which the primary stress is longitudinal compression. Such a member need not be vertical, but may have any orientation in space. Bearing walls, columns, and pedestals qualify as compression members under this definition.

The differentiation between columns and walls in the code is based on the principal use rather than on arbitrary relationships of height and cross-sectional dimensions. The code, however, permits walls to be designed using the principles stated for column design (see 14.4), as well as by the empirical method (see 14.5).

While a wall always encloses or separates spaces, it may also be used to resist horizontal or vertical forces or bending. For example, a retaining wall or a basement wall also supports various combinations of loads.

A column is normally used as a main vertical member carrying axial loads combined with bending and shear. It may, however, form a small part of an enclosure or separation.

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Admixture — Material other than water, aggregate, or hydraulic cement, used as an ingredient of concrete and added to concrete before or during its mixing to modify its properties.

Aggregate — Granular material, such as sand, gravel, crushed stone, and iron blast-furnace slag, used with a cementing medium to form a hydraulic cement concrete or mortar.

Aggregate, lightweight — Aggregate with a dry, loose weight of 70 lb/ft^3 or less.

Anchorage device — In post-tensioning, the hardware used for transferring a post-tensioning force from the prestressing steel to the concrete.

Anchorage zone — In post-tensioned members, the portion of the member through which the concentrated prestressing force is transferred to the concrete and distributed more uniformly across the section. Its extent is equal to the largest dimension of the cross section. For anchorage devices located away from the end of a member, the anchorage zone includes the disturbed regions ahead of and behind the anchorage devices.

Basic monostrand anchorage device — Anchorage device used with any single strand or a single 5/8 in. or smaller diameter bar that satisfies 18.21.1 and the anchorage device requirements of ACI 423.6, "Specification for Unbonded Single-Strand Tendons."

Basic multistrand anchorage device — Anchorage device used with multiple strands, bars, or wires, or with single bars larger than 5/8 in. diameter, that satisfies 18.21.1 and the bearing stress and minimum plate stiffness requirements of AASHTO Bridge Specifications, Division I, Articles 9.21.7.2.2 through 9.21.7.2.4.

Bonded tendon — Tendon in which prestressing steel is bonded to concrete either directly or through grouting.

Building official — See 1.2.3.

Cementitious materials — Materials as specified in Chapter 3, which have cementing value when used in concrete either by themselves, such as portland cement, blended hydraulic cements, and expansive cement, or such materials in combination with fly ash, other raw or calcined natural pozzolans, silica fume, and/or ground granulated blast-furnace slag.

COMMENTARY

Anchorage device — Most anchorage devices for post-tensioning are standard manufactured devices available from commercial sources. In some cases, designers or constructors develop "special" details or assemblages that combine various wedges and wedge plates for anchoring prestressing steel with specialty end plates or diaphragms. These informal designations as standard anchorage devices or special anchorage devices have no direct relation to the ACI 350 Code classification of anchorage devices as basic anchorage devices or special anchorage devices.

Anchorage zone — The terminology "ahead of" and "behind" the anchorage device is illustrated in Fig. R18.13.1(b).

Basic anchorage devices are those devices that are so proportioned that they can be checked analytically for compliance with bearing stress and stiffness requirements without having to undergo the acceptance-testing program required of special anchorage devices.

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Column — Member with a ratio of height-to-least lateral dimension exceeding 3 used primarily to support axial compressive load.

Composite concrete flexural members — Concrete flexural members of precast and/or cast-in-place concrete elements constructed in separate placements but so interconnected that all elements respond to loads as a unit.

Concrete — Mixture of portland cement or any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixtures.

Concrete, specified compressive strength of, (f'_c) — Compressive strength of concrete used in design and evaluated in accordance with provisions of Chapter 5, expressed in pounds per square inch (psi). Whenever the quantity $\sqrt{f'_c}$ is under a radical sign, square root of numerical value only is intended, and result has units of pounds per square inch (psi).

Concrete, structural lightweight — Concrete containing lightweight aggregate that conforms to 3.3 and has an air-dry unit weight as determined by "Test Method for Unit Weight of Structural Lightweight Concrete" (ASTM C 567), not exceeding 115 lb/ft^3. In this code, a lightweight concrete without natural sand is termed "all-lightweight concrete" and lightweight concrete in which all of the fine aggregate consists of normalweight sand is termed "sand-lightweight concrete."

Contraction joint — Formed, sawed, or tooled groove in a concrete structure to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure.

Curvature friction — Friction resulting from bends or curves in the specified prestressing tendon profile.

Deformed reinforcement — Deformed reinforcing bars, bar mats, deformed wire, welded plain wire fabric, and welded deformed wire fabric conforming to 3.5.3.

Development length — Length of embedded reinforcement required to develop the design strength of reinforcement at a critical section. See 9.3.3.

Duct — A conduit (plain or corrugated) to accommodate prestressing steel for post-tensioned installation. Requirements for post-tensioning ducts are given in Section 18.15.

Effective depth of section (d) — Distance measured from extreme compression fiber to centroid of tension reinforcement.

Effective prestress — Stress remaining in prestressing steel after all losses have occurred.

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Embedment length — Length of embedded reinforcement provided beyond a critical section.

Extreme tension steel — The reinforcement (prestressed or nonprestressed) that is the farthest from the extreme compression fiber.

Isolation joint — A separation between adjoining parts of a concrete structure, usually a vertical plane, at a designed location such as to interfere least with performance of the structure, yet such as to allow relative movement in three directions and avoid formation of cracks elsewhere in the concrete and through which all or part of the bonded reinforcement is interrupted.

Jacking force — In prestressed concrete, temporary force exerted by device that introduces tension into prestressing steel.

Load, dead — Dead weight supported by a member, as defined by general building code of which this code forms a part (without load factors).

Load, factored — Load, multiplied by appropriate load factors, used to proportion members by the strength design method of this code. See 8.1.1 and 9.2.

Load, live — Live load specified by general building code of which this code forms a part (without load factor).

Load, service — Load specified by general building code of which this code forms a part (without load factors).

Modulus of elasticity — Ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material. See 8.5.

Moment frame — Frame in which members and joints resist forces through flexure, shear, and axial force. Moment frames shall be categorized as follows:

> ***Intermediate moment frame*** — A cast-in-place frame complying with the requirements of 21.2.2.3 and 21.12 in addition to the requirements for ordinary moment frames.
>
> ***Ordinary moment frame*** — A cast-in-place or precast concrete frame complying with the requirements of Chapters 1 through 18.
>
> ***Special moment frame*** — A cast-in-place frame complying with the requirements of 21.2 through 21.5, or a precast frame complying with the requirements of 21.2 through 21.6. In addition, the requirements for ordinary moment frames shall be satisfied.

Net tensile strain — The tensile strain at nominal strength exclusive of strains due to effective prestress, creep, shrinkage, and temperature.

COMMENTARY

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Normal environmental exposure — Exposure to liquids with a pH greater than 5, or exposure to sulfate solutions 1000 ppm or less.

Pedestal — Upright compression member with a ratio of unsupported height to average least lateral dimension of less than 3.

Plain concrete — Structural concrete with no reinforcement or with less reinforcement than the minimum amount specified for reinforced concrete.

Plain reinforcement — Reinforcement that does not conform to definition of deformed reinforcement. See 3.5.4.

Post-tensioning — Method of prestressing in which prestressing steel is tensioned after concrete has hardened.

Precast concrete — Structural concrete element cast elsewhere than its final position in the structure.

Prestressed concrete — Structural concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.

Prestressing steel — High-strength steel element such as wire, bar, or strand, or a bundle of such elements, used to impart prestress forces to concrete.

Pretensioning — Method of prestressing in which prestressing steel is tensioned before concrete is placed.

Reinforced concrete — Structural concrete reinforced with no less than the minimum amounts of prestressing steel or nonprestressed reinforcement specified in Chapters 1 through 21 and Appendices A through C.

Reinforcement — Material that conforms to 3.5, excluding prestressing steel unless specifically included.

Reshores — Shores placed snugly under a concrete slab or other structural member after the original form and shores have been removed from a larger area, thus requiring the new slab or structural member to deflect and support its own weight and existing construction loads applied prior to the installation of the reshores.

Severe environmental exposure — Exposure conditions in which the limits defining normal environmental exposure are exceeded.

COMMENTARY

CODE

Sheathing — A material encasing prestressing steel to prevent bonding of the prestressing steel with the surrounding concrete, to provide corrosion protection, and to contain the corrosion inhibiting coating.

Shores — Vertical or inclined support members designed to carry the weight of the formwork, concrete, and construction loads above.

Span length — See 8.7.

Special anchorage device — Anchorage device that satisfies 18.15.1 and the standardized acceptance tests of AASHTO "Standard Specifications for Highway Bridges," Division II, Article 10.3.2.3.

Spiral reinforcement — Continuously wound reinforcement in the form of a cylindrical helix.

Splitting tensile strength (f_{ct}) — Tensile strength of concrete determined in accordance with ASTM C 496 as described in "Specification for Lightweight Aggregates for Structural Concrete" (ASTM C 330).

Stirrup — Reinforcement used to resist shear and torsion stresses in a structural member; typically bars, wires, or welded wire fabric (plain or deformed) either single leg or bent into L, U, or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. The term "stirrups" is usually applied to lateral reinforcement in flexural members and the term "ties" to those in compression members.) See also *Tie*.

Strength, design — Nominal strength multiplied by a strength reduction factor ϕ. See 9.3.

Strength, nominal — Strength of a member or cross section calculated in accordance with provisions and assumptions of the strength design method of this code before application of any strength reduction factors. See 9.3.1.

Strength, required — Strength of a member or cross section required to resist factored loads or related internal moments and forces in such combinations as are stipulated in this code. See 9.1.1.

Stress — Intensity of force per unit area.

Structural concrete — All concrete used for structural purposes including plain and reinforced concrete.

COMMENTARY

Sheathing—Typically, sheathing is a continuous, seamless, high-density polyethylene material extruded directly on the coated prestressing steel. In environmental concrete structures, only seamless, extruded sheathing is acceptable for use in unbonded, monostrand prestressing steel.

Special anchorage devices are any devices (monostrand or multistrand) that do not meet the relevant PTI or AASHTO bearing stress and, where applicable, stiffness requirements. Most commercially marketed multibearing surface anchorage devices are special anchorage devices. As provided in 18.15.1, such devices can be used only when they have been shown experimentally to be in compliance with the AASHTO requirements. This demonstration of compliance will ordinarily be furnished by the device manufacturer.

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Structural walls — Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shearwall is a structural wall. Structural walls shall be categorized as follows:

Intermediate precast structural wall — A wall complying with all applicable requirements of Chapters 1 through 18 in addition to 21.13.

Ordinary reinforced concrete structural wall — A wall complying with the requirements of Chapters 1 through 18.

Special precast structural wall — A precast wall complying with the requirements of 21.8. In addition, the requirements of ordinary reinforced concrete structural walls and the requirements of 21.2 shall be satisfied.

Special reinforced concrete structural wall — A cast-in-place wall complying with the requirements of 21.2 and 21.7 in addition to the requirements for ordinary reinforced concrete structural walls.

Tendon — In pretensioned applications, the tendon is the prestressing steel. In post-tensioned applications, the tendon is a complete assembly consisting of anchorages, prestressing steel, and sheathing with coating for unbonded applications or ducts with grout for bonded applications.

Tie — Loop of reinforcing bar or wire enclosing longitudinal reinforcement. A continuously wound bar or wire in the form of a circle, rectangle, or other polygon shape without re-entrant corners is acceptable. See also *Stirrup.*

Transfer — Act of transferring stress in prestressing steel from jacks or pretensioning bed to concrete member.

Unbonded tendon — Tendon in which the prestressing steel is prevented from bonding to the concrete and is free to move relative to the concrete. The prestressing force is permanently transferred to the concrete at the tendon ends by the anchorages only.

Wall — Member, usually vertical, used to enclose or separate spaces.

Wobble friction — In prestressed concrete, friction caused by unintended deviation of prestressing sheath or duct from its specified profile.

Yield strength — Specified minimum yield strength or yield point of reinforcement in pounds per square inch. Yield strength or yield point shall be determined in tension according to applicable ASTM standards as modified by 3.5 of this code.

COMMENTARY

CODE

2.2 — The following additional terms are defined for general use in this code

Backer rod — A compressible rod placed between joint filler and sealant and used to provide support for and to control the depth of sealant.

Environmental durability factor — Factor used to control reinforcement stresses and crack widths in members designed using the strength design approach.

Joint filler — A compressible, preformed material used to fill an expansion joint to prevent the infiltration of debris and to provide support for backer rod and sealants.

Joint sealant — A synthetic elastomeric material used to finish a joint and to exclude solid foreign materials.

Waterstop — A continuous preformed strip of metal, rubber, plastic, or other material inserted across a joint to prevent the passage of liquid through the joint.

COMMENTARY

R2.2

An ideal **backer rod** will permit compression to one-half its original width and will re-expand to fill the joint when the adjacent members contract. Neoprene and open or closed cell plastic foams are satisfactory materials for backer rods. The backer rod should be compatible with the adjacent joint sealant.

Refer to Chapter 9 of this Code for rules on the application of this factor. When the **environmental durability factor** is used, members proportioned using the strength design approach are similar to those proportioned using the alternate design method (Appendix I).

Cork, neoprene, rubber, foam, and other materials conforming to ASTM D 1056 and D 1752 are satisfactory **joint fillers**. The preformed filler should be compatible with adjacent joint sealant.

Sealants used in water treatment plants, reservoirs, and other structural facilities that will be in contact with potable water should be certified as compliant with ANSI/NSF 61. In addition, the sealant should be resistant to chlorinated water and suitable for immersion service.

Waterstops are available in various sizes, shapes, and materials. Environmental concrete structures commonly use waterstops of preformed rubber or polyvinyl chloride with a minimum thickness of 3/8 in. They should normally be at least 9 in. wide for expansion joints and 6 in. wide for other types of joints to provide adequate embedment in the concrete. Metal waterstops are used for special exposure environments. Expansive rubber or adhesive waterstops may be used in joints cast against previously placed concrete, or in new construction when approved by the engineer. Chemical resistance, joint movement capacity, and design temperature range are among the items that should be investigated when selecting waterstops. Joint movement capacity and design temperature range are among the items that should be investigated when selecting waterstops. Joint details are further described in ACI 350.4R.

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COMMENTARY

Notes

PART 2 — STANDARDS FOR TESTS AND MATERIALS

CHAPTER 3 — MATERIALS

CODE

3.0 — Notation

f_y = specified yield strength of nonprestressed reinforcement, psi

3.1 — Tests of materials

3.1.1 — Building official shall have the right to order testing of any materials used in concrete construction to determine if materials are of quality specified.

3.1.2 — Tests of materials and of concrete shall be made in accordance with standards listed in 3.8.

3.1.3 — A complete record of tests of materials and of concrete shall be retained by the inspector for 2 years after completion of the project, and made available for inspection during the progress of the work.

3.2 — Cements

3.2.1 — Cement shall conform to one of the following specifications:

(a) "Specification for Portland Cement" (ASTM C 150);

(b) "Specification for Blended Hydraulic Cements" (ASTM C 595), excluding Types S and SA, which are not intended as principal cementing constituents of structural concrete;

(c) "Specification for Expansive Hydraulic Cement" (ASTM C 845).

3.2.2 — Cement used in the work shall correspond to that on which selection of concrete proportions was based. See 5.2.

COMMENTARY

R3.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip.

R3.1 — Tests of materials

R3.1.3 — The record of tests of materials and of concrete must be preserved for at least 2 years after completion of the project. Completion of the project is the date at which the owner accepts the project or when the certificate of occupancy is issued, whichever date is later. Local legal requirements may require longer preservation of such records.

R3.2 — Cements

R3.2.1 — Different cements or cements from different producers should not be used interchangeably in the same element or portion of the work. Additional guidance on cement may be found in ACI 225R.[3.1]

Concrete made with expansive cement can be used to reduce drying-shrinkage cracking in environmental engineering concrete structures, but the ACI 350 committee is not yet in a position to recommend detailed requirements for its use. For the design to be successful, the engineer must recognize the characteristics and properties of shrinkage compensating concrete and cement as described in ACI 223 and ASTM C 845 (Type E1-K), respectively. Type K cement has historically shown very satisfactory resistance to sulfate attack in both the laboratory and the field. Additional care and control should be exercised during design and construction. Detailed information on shrinkage-compensating concrete is contained in ACI 223.[3.2]

R3.2.2 — Depending on the circumstances, the provision of 3.2.2 may require only the same type of cement or may require cement from the identical source. The latter would be the case if the standard deviation[3.3] of strength tests used in establishing the required strength margin was based on a cement from a particular source. If the standard deviation was based on tests involving a given type of cement obtained from several sources, the former interpretation would apply.

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3.3 — Aggregates

3.3.1 — Concrete aggregates shall conform to one of the following specifications:

(a) "Specification for Concrete Aggregates" (ASTM C 33);

(b) "Specification for Lightweight Aggregates for Structural Concrete" (ASTM C 330).

Exception: Aggregates that have been shown by special test or actual service to produce concrete of adequate strength and durability and approved by the building official.

3.3.2 — Nominal maximum size of coarse aggregate shall be not larger than:

(a) 1/5 the narrowest dimension between sides of form; nor

(b) 1/3 the depth of slabs; nor

(c) 3/4 the minimum clear spacing between individual reinforcing bars or wires, bundles of bars, individual tendons, bundled tendons, or ducts.

These limitations shall not apply if, in the judgment of the engineer, workability and methods of consolidation are such that concrete can be placed without honeycomb or voids.

3.3.3 — Where aggregates are alkali-reactive, impose restrictions on materials to minimize deterioration.

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R3.3 — Aggregates

R3.3.1 — It is recognized that aggregates conforming to the ASTM specifications are not always economically available and that, in some instances, noncomplying materials have a long history of satisfactory performance. Such nonconforming materials are permitted with special approval when acceptable evidence of satisfactory performance is provided. It should be noted, however, that satisfactory performance in the past does not guarantee good performance under other conditions and in other localities. Whenever possible, aggregates conforming to the designated specifications should be used.

R3.3.2 — The size limitations on aggregates are provided to ensure proper encasement of reinforcement and to minimize honeycomb. Note that the limitations on maximum size of the aggregate may be waived if, in the judgment of the engineer, the workability and methods of consolidation of the concrete are such that the concrete can be placed without honeycomb or voids. In this instance, the engineer must decide whether or not the limitations on maximum size of aggregate may be waived.

R3.3.3 — Alkali-aggregate reactions can cause an expansive action when reactive aggregates come in contact with alkali hydroxides in the hardened concrete. These reactions can result in long-term deterioration of concrete, usually the interior of the concrete. It is recommended to specify testing and quality aggregates conforming to ASTM C 33.

Reactivity testing of aggregates should be required when local aggregates are suspected of being alkali reactive. Unless all local aggregates are known to be nonreactive, a low-alkali cement should be used. Pozzolans and lithium hydroxide admixtures may also be considered. The use of lithium hydroxide admixtures to control reactive aggregates, however, is technology that is not widely accepted at this time.

Alkali-aggregate reactivity potential should be determined for local aggregates when local aggregates are suspected of being alkali reactive. On projects where alkali reactivity is a known problem, prescreening of aggregate sources before completing design of the project may be advisable.

Aggregates that do not indicate a potential for alkali reactivity or reactive constituents may be used without further testing. Aggregates that indicate a potential for alkali reactivity should be tested for potential reactivity using the mortar-bar test, ASTM C 227 and C 289. Nonreactive aggregates may need to be imported if local aggregates exhibit unacceptable potential reactivity.

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3.4 — Water

3.4.1 — Water used in mixing concrete shall be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances deleterious to concrete or reinforcement.

3.4.2 — Mixing water for prestressed concrete, including that portion of mixing water contributed in the form of free moisture on aggregates, shall not contain deleterious amounts of chloride ion. See 4.4.1.

3.4.3 — Nonpotable water shall not be used in concrete unless the following are satisfied:

3.4.3.1 — Selection of concrete proportions shall be based on concrete mixes using water from the same source.

3.4.3.2 — Mortar test cubes made with nonpotable mixing water shall have 7-day and 28-day strengths equal to at least 90 percent of strengths of similar specimens made with potable water. Strength test comparison shall be made on mortars, identical except for the mixing water, prepared and tested in accordance with "Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in.or 50-mm Cube Specimens)" (ASTM C 109).

3.5 — Steel reinforcement

3.5.1 — Reinforcement shall be deformed reinforcement, except that plain reinforcement shall be permitted for spirals or prestressing steel; and reinforcement consisting of structural steel, steel pipe, or steel tubing shall be permitted as specified in this code.

3.5.2 — Welding of reinforcing bars shall conform to "Structural Welding Code—Reinforcing Steel," ANSI/AWS D1.4 of the American Welding Society. Type and location of welded splices and other required welding of reinforcing bars shall be indicated on the design drawings or in the project specifications. ASTM reinforcing bar specifications, except for ASTM A 706, shall be supplemented to require a report of material properties necessary to conform to the requirements in ANSI/AWS D1.4.

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R3.4 — Water

R3.4.1 — Almost any natural water that is drinkable (potable) and has no pronounced taste or odor is satisfactory as mixing water for making concrete. Impurities in mixing water, when excessive, may affect not only setting time, concrete strength, and volume stability (length change), but may also cause efflorescence or corrosion of reinforcement. Where possible, water with high concentrations of dissolved solids should be avoided.

Salts or other deleterious substances contributed from the aggregate or admixtures are additive to the amount that might be contained in the mixing water. These additional amounts must be considered in evaluating the acceptability of the total impurities that may be deleterious to concrete or steel.

R3.5 — Steel reinforcement

R3.5.1 — Materials permitted for use as reinforcement are specified. Other metal elements, such as inserts, anchor bolts, or plain bars for dowels at isolation or contraction joints, are not normally considered to be reinforcement under the provisions of this code.

R3.5.2 — When welding of reinforcing bars is required, the weldability of the steel and compatible welding procedures need to be considered. The provisions in ANSI/AWS D1.4 Welding Code cover aspects of welding reinforcing bars, including criteria to qualify welding procedures. The welder needs to be certified to the requirements of ANSI/AWS D1.4.

Weldability of the steel is based on its chemical composition or carbon equivalent (CE). The Welding Code establishes preheat and interpass temperatures for a range of carbon equivalents and reinforcing bar sizes. CE is calculated from

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the chemical composition of the reinforcing bars. The Welding Code has two expressions for calculating CE. A relatively short expression, considering only the elements carbon and manganese, is to be used for bars other than ASTM A 706 material. A more comprehensive expression is given for ASTM A 706 bars. The CE formula in the Welding Code for A 706 bars is identical to the CE formula in the ASTM A 706 specification.

The engineer should realize that the chemical analysis for bars other than A 706 required to calculate the CE is not routinely provided by the producer of the reinforcing bars. Hence, for welding reinforcing bars other than A 706 bars, the contract documents should specifically require results of the chemical analysis to be furnished.

The ASTM A 706 specification covers low-alloy steel reinforcing bars intended for applications requiring controlled tensile properties or welding. Weldability is accomplished in the A 706 specification by limits or controls on chemical composition and on carbon equivalent.[3.4] The producer is required by the A 706 specification to report the chemical composition and carbon equivalent.

The ANSI/AWS D1.4 Welding Code requires the contractor to prepare written welding procedure specifications conforming to the requirements of the Welding Code. Appendix A of the Welding Code contains a suggested form that shows the information required for such a specification for each joint welding procedure.

It is often necessary to weld to existing reinforcing bars in a structure when no mill test report of the existing reinforcement is available. This condition is particularly common in alterations or building expansions. ANSI/AWS D1.4 states for such bars that a chemical analysis may be performed on representative bars. If the chemical composition is not known or obtained, the Welding Code requires a minimum preheat. For bars other than A 706 material, the minimum preheat required is 300 °F for bars No. 6 or smaller, and 400 °F for No. 7 bars or larger. The required preheat for all sizes of A 706 is to be the temperature given in the Welding Code's table for minimum preheat corresponding to the range of CE "over 45 percent to 55 percent." Welding of the particular bars must then be performed in accordance with ANSI/AWS D 1.4. It should also be determined if additional precautions are in order, based on other considerations such as stress level in the bars, consequences of failure, and heat damage to existing concrete due to welding operations.

Welding of wire to wire, and of wire or welded wire fabric to reinforcing bars or structural steel elements, is not covered by ANSI/AWS D1.4. If welding of this type is required on a project, the engineer should specify requirements or performance criteria for this welding. If cold drawn wires are to be welded, the welding procedures should address the potential loss of yield strength and ductility achieved by the cold working process (during manufacture) when such wires are heated by welding.

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3.5.3 — Deformed reinforcement

3.5.3.1 — Deformed reinforcing bars shall conform to one of the following specifications:

(a) "Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement" (ASTM A 615);

(b) "Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement" (ASTM A 706);

(c) "Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement" (ASTM A 996). Bars from rail-steel shall be Type R.

3.5.3.2 — Deformed reinforcing bars with a specified yield strength f_y exceeding 60,000 psi shall be permitted, provided f_y shall be the stress corresponding to a strain of 0.35 percent and the bars otherwise conform to one of the ASTM specifications listed in 3.5.3.1. See 9.4.

3.5.3.3 — Bar mats for concrete reinforcement shall conform to "Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement" (ASTM A 184).

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Machine and resistance welding as used in the manufacture of welded wire fabrics is covered by ASTM A 185 and A 497, and is not part of this concern.

R3.5.3 — Deformed reinforcement

R3.5.3.1 — ASTM A 615 covers deformed billet-steel reinforcing bars that are currently the most widely used type of steel bar in reinforced concrete construction in the United States. The specification requires that the bars be marked with the letter *S* for type of steel.

ASTM A 706 covers low-alloy steel deformed bars intended for applications where controlled tensile properties, restrictions on chemical composition to enhance weldability, or both, are required. The specification requires that the bars be marked with the letter *W* for type of steel.

Deformed bars produced to meet both ASTM A 615 and A 706 are required to be marked with the letters *S* and *W* for type of steel.

Rail-steel reinforcing bars used with this code are required to conform to ASTM A 996 including the provisions for Type R bars, and marked with the letter *R* for type of steel.

Type R bars are required to meet more restrictive provisions for bend tests.

Previous standards ASTM A 616 and ASTM A 617 have been replaced by ASTM A 996.

R3.5.3.2 — ASTM A 615 includes provisions for Grade 75 bars in sizes No. 6 through 18.

The 0.35 percent strain limit is necessary to ensure that the assumption of an elasto-plastic stress-strain curve in 10.2.4 will not lead to unconservative values of the member strength.

The 0.35 strain requirement is not applied to reinforcing bars having yield strengths of 60,000 psi or less. For steels having strengths of 40,000 psi, as were once used extensively, the assumption of an elasto-plastic stress-strain curve is well justified by extensive test data. For higher strength steels, up to 60,000 psi, the stress-strain curve may or may not be elasto-plastic as assumed in 10.2.4, depending on the properties of the steel and the manufacturing process. When the stress-strain curve is not elasto-plastic, however, there is limited experimental evidence to suggest that the actual steel stress at ultimate strength may not be enough less than the specified yield strength to warrant the additional effort of testing to the more restrictive criterion applicable to steels having f_y greater than 60,000 psi. In such cases, the ϕ-factor can be expected to account for the strength deficiency.

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Reinforcing bars used in bar mats shall conform to one of the specifications listed in 3.5.3.1.

3.5.3.4 — Deformed wire for concrete reinforcement shall conform to "Specification for Steel Wire, Deformed, for Concrete Reinforcement" (ASTM A 496), except that wire shall not be smaller than size D4 and for wire with a specified yield strength f_y exceeding 60,000 psi, f_y shall be the stress corresponding to a strain of 0.35 percent if the yield strength specified in the design exceeds 60,000 psi.

3.5.3.5 — Welded plain wire fabric for concrete reinforcement shall conform to "Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement" (ASTM A 185), except that for wire with a specified yield strength f_y exceeding 60,000 psi, f_y shall be the stress corresponding to a strain of 0.35 percent if the yield strength specified in the design exceeds 60,000 psi. Welded intersections shall not be spaced farther apart than 12 in. in direction of calculated stress, except for wire fabric used as stirrups in accordance with 12.13.2.

3.5.3.6 — Welded deformed wire fabric for concrete reinforcement shall conform to "Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement" (ASTM A 497), except that for wire with a specified yield strength f_y exceeding 60,000 psi, f_y shall be the stress corresponding to a strain of 0.35 percent if the yield strength specified in the design exceeds 60,000 psi. Welded intersections shall not be spaced farther apart than 16 in. in direction of calculated stress, except for wire fabric used as stirrups in accordance with 12.13.2.

3.5.3.7 — Galvanized reinforcing bars shall comply with "Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement" (ASTM A 767). Epoxy-coated reinforcing bars shall comply with "Specification for Epoxy-Coated Reinforcing Steel Bars" (ASTM A 775) or with "Specification for Epoxy Coated Prefabricated Steel Reinforcing Bars" (ASTM A 934). Bars to be galvanized or epoxy-coated shall conform to one of the specifications listed in 3.5.3.1.

3.5.3.8 — Epoxy-coated wires and welded wire fabric shall comply with "Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement" (ASTM A 884). Wires to be epoxy-coated shall conform to 3.5.3.4 and welded wire fabric to be epoxy-coated shall conform to 3.5.3.5 or 3.5.3.6.

3.5.4 — Plain reinforcement

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R3.5.3.5 — Welded plain wire fabric must be made of wire conforming to "Specification for Steel Wire, Plain, for Concrete Reinforcement" (ASTM A 82). ASTM A 82 has a minimum yield strength of 70,000 psi. The code has assigned a yield strength value of 60,000 psi, but makes provision for the use of higher yield strengths provided the stress corresponds to a strain of 0.35 percent.

R3.5.3.6 — Welded deformed wire fabric must be made of wire conforming to "Specification for Steel Wire, Deformed, for Concrete Reinforcement" (ASTM A 496). ASTM A 496 has a minimum yield strength of 70,000 psi. The code has assigned a yield strength value of 60,000 psi, but makes provision for the use of higher yield strengths provided the stress corresponds to a strain of 0.35 percent.

R3.5.3.7 — Galvanized reinforcing bars (A 767) and epoxy-coated reinforcing bars (A 775) were added to the ACI 318-83 Code, and epoxy-coated prefabricated reinforcing bars (A 934) were added to the ACI 318-95 Code recognizing their usage, especially for conditions where corrosion resistance of reinforcement is of particular concern. They have typically been used in parking decks, bridge decks, and other highly corrosive environments.

R3.5.4 — Plain reinforcement

Plain bars and plain wire are permitted only for spiral reinforcement (either as lateral reinforcement for compression members, for torsion members, or for confining reinforcement for splices).

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3.5.4.1 — Plain bars for spiral reinforcement shall conform to the specification listed in 3.5.3.1(a) or (b).

3.5.4.2 — Plain wire for spiral reinforcement shall conform to "Specification for Steel Wire, Plain, for Concrete Reinforcement" (ASTM A 82), except that for wire with a specified yield strength f_y exceeding 60,000 psi, f_y shall be the stress corresponding to a strain of 0.35 percent if the yield strength specified in the design exceeds 60,000 psi.

3.5.5 — Prestressing steel

3.5.5.1 — Steel for prestressing shall conform to one of the following specifications:

(a) Wire conforming to "Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete" (ASTM A 421);

(b) Low-relaxation wire conforming to "Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete" including Supplement "Low-Relaxation Wire" (ASTM A 421);

(c) Strand conforming to "Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete" (ASTM A 416);

(d) Bar conforming to "Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete" (ASTM A 722).

3.5.5.2 — Wire, strands, and bars not specifically listed in ASTM A 421, A 416, or A 722 are allowed provided they conform to minimum requirements of these specifications and do not have properties that make them less satisfactory than those listed in ASTM A 421, A 416, or A 722.

3.5.6 — Structural steel, steel pipe, or tubing

3.5.6.1 — Structural steel used with reinforcing bars in composite compression members meeting requirements of 10.16.7 or 10.16.8 shall conform to one of the following specifications:

(a) "Standard Specification for Structural Steel Shapes" (ASTM A 992);

(b) "Specification for High-Strength Low-Alloy Structural Steel" (ASTM A 242);

(c) "Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality" (ASTM A 572);

(d) "Specification for High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in. (100 mm) Thick" (ASTM A 588).

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R3.5.5 — Prestressing steel

R3.5.5.1 — Because low-relaxation prestressing steel is addressed in a supplement to ASTM A 421, which applies only when low-relaxation material is specified, the appropriate ASTM reference is listed as a separate entity.

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3.5.6.2 — Steel pipe or tubing for composite compression members composed of a steel encased concrete core meeting requirements of 10.16.6 shall conform to one of the following specifications:

(a) Grade B of "Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless" (ASTM A 53);

(b) "Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes" (ASTM A 500);

(c) "Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing" (ASTM A 501).

3.6 — Admixtures

3.6.1 — Admixtures to be used in concrete shall be subject to prior approval by the engineer.

3.6.2 — An admixture shall be shown capable of maintaining essentially the same composition and performance throughout the work as the product used in establishing concrete proportions in accordance with 5.2.

3.6.3 — Calcium chloride or admixtures containing chloride ion other than from impurities in admixture ingredients shall not be used.

3.6.4 — Air-entraining admixtures shall conform to "Specification for Air-Entraining Admixtures for Concrete" (ASTM C 260). All air-entraining admixtures that are not vinsol resin or vinsol rosin based shall be tested in accordance with ASTM C 260 at the highest percent air allowed in the specifications.

3.6.5 — Water-reducing admixtures, retarding admixtures, accelerating admixtures, water-reducing and retarding admixtures, and water-reducing and accelerating admixtures shall conform to "Specification for Chemical Admixtures for Concrete" (ASTM C 494) or "Specification for Chemical Admixtures for Use in Producing Flowing Concrete" (ASTM C 1017).

3.6.6 — Fly ash or other pozzolans used as admixtures shall conform to "Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete" (ASTM C 618).

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R3.6 — Admixtures

R3.6.3 — Admixtures containing any chloride, other than from impurities in admixture ingredients, should not be used in concrete. Calcium chloride in concrete is particularly detrimental in the wet conditions encountered in environmental engineering concrete structures.

R3.6.4 — Vinsol resin based air-entraining admixtures have been used in the concrete industry since the 1960s or earlier. They have performed the function of entraining air in concrete with few problems. The use of non-vinsol resin or non-vinsol rosin based air-entraining admixtures in concrete have on occasion, especially at specified air contents of 6 percent or higher, resulted in problems such as unacceptable reduction in compressive strength, increase in shrinkage, and surface weakness (similar to scaling).

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3.6.7 — Ground-granulated blast-furnace slag used as an admixture shall conform to "Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars" (ASTM C 989).

3.6.8 — Admixtures used in concrete containing ASTM C 845 expansive cements shall be compatible with the cement and produce no deleterious effects.

3.6.9 — Silica fume used as an admixture shall conform to "Specification for Silica Fume for Use in Hydraulic-Cement Concrete and Mortar" (ASTM C 1240).

3.7 — Storage of materials

3.7.1 — Cementitious materials and aggregates shall be stored in such manner as to prevent deterioration or intrusion of foreign matter.

3.7.2 — Any material that has deteriorated or has been contaminated shall not be used for concrete.

3.8 — Reference standards

COMMENTARY

R3.6.7 — Ground-granulated blast-furnace slag conforming to ASTM C 989 is used as an admixture in concrete in much the same way as fly ash. Generally, it should be used with portland cements conforming to ASTM C 150, and only rarely would it be appropriate to use ASTM C 989 slag with an ASTM C 595 blended cement that already contains a pozzolan or slag. Such use with ASTM C 595 cements might be considered for massive concrete placements where slow strength gain can be tolerated and where low heat of hydration is of particular importance. ASTM C 989 includes appendices that discuss effects of ground-granulated blast-furnace slag on concrete strength, sulfate resistance, and alkali-aggregate reaction.

R3.6.8 — The use of admixtures in concrete containing ASTM C 845 expansive cements has reduced levels of expansion or increased shrinkage values. See ACI 223.[3.3]

R3.8 — Referenced standards

The ASTM standard specifications listed are the latest editions at the time these code provisions were adopted. Because these specifications are revised frequently, generally in minor details only, the user of the code should check directly with the sponsoring organization if it is desired to reference the latest edition. Such a procedure, however, obligates the user of the specification to evaluate if any changes in the later edition are significant in the use of the specification.

Standard specifications or other material to be legally adopted by reference into a building code must refer to a specific document. This can be done by simply using the complete serial designation because the first part indicates the subject and the second part the year of adoption. All standard documents referenced in this code are listed in 3.8, with the title and complete serial designation. In other sections of the code, the designations do not include the date so that all may be kept up-to-date by simply revising 3.8.

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3.8.1 — Standards of the American Society for Testing and Materials referred to in this code are listed below with their serial designations, including year of adoption or revision, and are declared to be part of this code as if fully set forth herein:

A 992/ A 992M-02	Standard Specification for Structural Steel Shapes
A 53/ A 53M-02	Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless
A 82-97a	Standard Specification for Steel Wire, Plain, for Concrete Reinforcement
A 108-99	Standard Specification for Steel Bars, Carbon, Cold-Finished, Standard Quality
A 184/ A 184M-01	Standard Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement
A 185-97	Standard Specification for Steel Welded Wire Fabric, Plain, for Concrete
A 227/ A 227M-99	Specification for Steel Wire, Cold-Drawn for Mechanical Springs
A 242/ A 242M-03a	Standard Specification for High-Strength Low-Alloy Structural Steel
A 307-03	Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength
A 336/ A 336M-03	Standard Specification for Alloy Steel Forgings for Pressure and High Temperature Parts
A 416/ A 416M-02	Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
A 421/ A 421M-02	Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
A 475-98	Specification for Zinc-Coated Steel Wire Strand
A 496-97a	Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement
A 497/ A 497M-02	Standard Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete
A 500-03a	Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
A 501-01	Standard Specification for Hot-Formed

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	Welded and Seamless Carbon Steel Structural Tubing
A 572/ A 572M-03	Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
A 586-98	Standard Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand and Zinc-Coated Wire for Spun-In-Place Structural Strand
A 588/ A 588M-03	Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in. (100 mm) Thick
A 603-98	Standard Specification for Zinc-Coated Steel Structural Wire Rope
A 615/ A 615M-03	Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A 648-95	Standard Specification for Steel Wire, Hard Drawn for Prestressing Concrete Pipe
A 653/ A 653M-03	Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process
A 706/ A 706M-03	Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A 722/ A 722M-98	Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete
A 767/ A 767M-00b	Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement
A 775/ A 775M-01	Standard Specification for Epoxy-Coated Steel Reinforcing Bars
A 821/ A 821M-99	Standard Specification for Steel Wire, Hard Drawn for Prestressing Concrete Tanks
A 881/ A 881M-02	Standard Specification for Steel Wire, Deformed, Stress-Relieved or Low-Relaxation for Prestressed Concrete Railroad Ties
A 882/ A 882M-02a	Standard Specification for Filled Epoxy-Coated Seven-Wire Prestressing Steel Strand
A 884/ A 884M-02	Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement

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A 934/ A 934M-03	Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars
A 996/ A 996M-03	Standard Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement
C 31/ C 31M-03a	Standard Practice for Making and Curing Concrete Test Specimens in the Field
C 33-03	Standard Specification for Concrete Aggregates
C 39/ C 39M-03a	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 42/ C 42M-03	Standard Test Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 94/ C 94M-03	Standard Specification for Ready-Mixed Concrete
C 109/ C 109M-02	Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in.or (50-mm) Cube Specimens)
C 144-03	Standard Specification for Aggregate for Masonry Mortar
C 150-02a$^{\varepsilon 1}$	Standard Specification for Portland Cement
C 172-99	Standard Practice for Sampling Freshly Mixed Concrete
C 192/ C 192M-02	Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory
C 260-01	Standard Specification for Air-Entraining Admixtures for Concrete
C 295-03	Standard Guide for Petrographic Examination of Aggregates for Concrete
C 330-03	Standard Specification for Lightweight Aggregates for Structural Concrete
C 494/ C 494M-99a	Standard Specification for Chemical Admixtures for Concrete
C 496-96	Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C 567-00	Standard Test Method for Determining Density of Structural Lightweight Concrete
C 595-03	Standard Specification for Blended Hydraulic Cements
C 618-03	Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete

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C 685/ C 685M-01	Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing
C 845-96	Standard Specification for Expansive Hydraulic Cement
C 920-02	Specification for Elastomeric Joint Sealants
C 989-99	Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars
C 1017/ C 1017M-98	Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete
C 1064/ C 1064M-03	Standard Test Method for Temperature of Freshly Mixed Portland Cement Concrete
C 1077-03	Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation
C 1138-97	Test Method for Abrasion Resistance of Concrete (Underwater Method)
C 1218/ C 1218M-99	Standard Test Method for Water-Soluble Chloride in Mortar and Concrete
C 1240-03a	Standard Specification for Silica Fume Used in Cementitious Mixtures
D 395-03	Standard Test Methods for Rubber Property Compression Set
D 412-98a$^{\varepsilon 1}$	Standard Test Methods for Rubber Properties in Tension
D 570-98	Standard Test Method for Water Absorption of Plastics
D 746-98	Standard Test Method for Brittleness Temperature of Plastics and Elastomers by Impact
D 1056-00	Standard Specification for Flexible Cellular Materials—Sponge or Expanded Rubber
D 1149-99	Standard Test Method for Rubber Deterioration-Surface Ozone Cracking in a Chamber
D 1752-84	Specification for Preformed Sponge Rubber and Cork Expansion Joint (Reapproved 1996)$^{\varepsilon 1}$ Fillers for Concrete Paving and Structural Construction
D 2000-03	Classification System for Rubber Products in Automotive Applications
D 2240-03	Test Method for Rubber Property—Durometer Hardness

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E 96-00 Test Methods for Water Vapor Transmission of Materials

E 329-03 Standard Specification for Agencies Engaged in the Testing and/or Inspection of Materials Used in Construction

3.8.2 — "Structural Welding Code—Reinforcing Steel" (ANSI/AWS D1.4-98) of the American Welding Society is declared to be part of this code as if fully set forth herein.

3.8.3 — Section 2.3 Combining Factored Loads Using Strength Design of "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-05) is declared to be part of this standard as if fully set forth herein, for the purpose cited in 9.2.4.

3.8.4 — "Specification for Unbonded Single Strand Tendons (ACI 423.6-01) and Commentary (423.6R-01)" is declared to be part of this code as if fully set forth herein.

3.8.5 — "Qualification of Post-Installed Mechanical Anchors in Concrete (ACI 355.2-04)" is declared to be part of this code as if fully set forth herein, for the purpose cited in Appendix D.

3.8.6 — "Structural Welding Code—Steel (AWS D 1.1/D 1.1M: 2004)" of the American Welding Society is declared to be part of this code as if fully set forth herein.

3.8.7 — "Acceptance Criteria for Moment Frames Based on Structural Testing (ACI T1.1-01)" is declared to be part of this code as if fully set forth herein.

3.8.8 — Standards of the following organizations are referred to in this code and are listed below with their serial designations, including year of adoption or revision, and are declared to be part of this code as if fully set forth herein:

3.8.8.1 — American Water Works Association

C 652-92 Disinfection of Water Storage Facilities

3.8.8.2 — U.S. Army Corps of Engineers Specifications

CRD C 572 U.S. Army Corps of Engineers Specification for Polyvinyl Chloride Waterstops (1999)

3.8.8.3 — Federal Specifications

TT-S-227e(3) (1969) Sealing Compound, Elastomeric Type, Multi-Component (for Calking, Sealing, and Glazing in Buildings and Other Structures)

COMMENTARY

R3.8.3 — ASCE 7 is available from ASCE Book Orders, Box 79404, Baltimore, Md., 21279-0404.

R3.8.5 — Parallel to development of the ACI 318-02 provisions for anchoring to concrete, ACI 355 developed a test method to define the level of performance required for post-installed anchors. This test method, ACI 355.2, contains requirements for the testing and evaluation of post-installed anchors for both cracked and uncracked concrete applications.

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TT-S-230c(2) (1970)	Sealing Compound, Elastomeric Type, Single Component (for Calking, Sealing, and Glazing in Buildings and Other Structures)

3.8.8.4 — American Concrete Institute

ACI 350.3/ 350.3R-06	"Seismic Design of Liquid-Containing Concrete Structures"

3.8.8.5 — American Association of State Highway and Transportation Officials

T260-84	Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials

COMMENTARY

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Notes

PART 3 — CONSTRUCTION REQUIREMENTS

CHAPTER 4 — DURABILITY REQUIREMENTS

CODE

4.0 — Notation

f_c' = specified compressive strength of concrete, psi

COMMENTARY

Chapter 4 of ACI 318 has been expanded herein for environmental concrete structures. Chapter 4 of earlier editions of ACI 318 was reformatted in 1989 to emphasize the importance of considering durability requirements before the designer selected f_c' and cover over the reinforcing steel.

Specified compressive concrete strength f_c' should correspond to maximum water-cementitious materials ratios of 0.40 to 0.42 for concretes exposed to freezing and thawing, water, wastewater, and to corrosive gases or for preventing corrosion of reinforcement, as shown in Table 4.2.2 and Table 4.3.1.

Generally, f_{cr}', defined as the required average compressive strength of concrete used as the basis for selection of concrete proportions, will be 500 to 700 psi higher than the specified compressive strength f_c'. Because it is difficult to accurately determine the water-cementitious materials ratio of concrete during production, the f_c' should be reasonably consistent with the specified water-cementitious materials ratio required for durability. Selection of an f_c' that is consistent with the water-cementitious materials ratio selected for durability will help ensure that the required water-cementitious materials ratio is actually obtained in the field. Because the usual emphasis on inspection is for strength, test results substantially higher than the specified strength may lead to a lack of concern for quality and production of concrete that exceeds the maximum water-cementitious materials ratio. Thus, a minimum f_c' of 4000 psi and a maximum water-cementitious materials ratio of 0.42 should be specified for an environmental structure exposed to freezing and thawing in a moist condition.

The code includes provisions for especially severe exposure where high resistance to chemical attack, alternate wetting and drying, freezing-and-thawing cycles, and exposure to the elements are required. It also includes special requirements for concrete cover and spacing of reinforcing bars (Chapter 7) for concrete in such environments.

The code requires the addition of protective coatings or liners in the case of severe exposures such as acids or other chemicals, and exposures to erosion by abrasion and cavitation.

Concrete ingredients and proportions must be selected to meet the minimum requirements stated in the code and the additional requirements of the contract documents.

Lightweight concrete may be used for environmental structures provided attention is given to durability requirements in general and protection against freezing and thawing, abrasion, and sulfate and chemical attack in particular.

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The code requires the use of an environmental durability factor S_d when using strength design. See R9.2.6.

4.1 — Water-cementitious materials ratio and cementitious material content

4.1.1 — The water-cementitious materials ratios specified in Tables 4.2.2 and 4.3.1 shall be calculated using the weight of cement meeting ASTM C 150, C 595, or C 845 plus the weight of fly ash and other pozzolans meeting ASTM C 618, slag meeting ASTM C 989, and silica fume meeting ASTM C 1240, if any, except that when concrete is exposed to deicing chemicals, 4.2.3 further limits the amount of fly ash, pozzolans, silica fume, slag, or the combination of these materials.

4.1.2 — Minimum cementitious materials content shall be as indicated in Table 4.1.2.1.

TABLE 4.1.2.1 — MINIMUM CEMENTITIOUS MATERIAL CONTENT

Nominal maximum aggregate size, in.	Coarse aggregate (ASTM C 33) size no.*	Minimum cementitious materials (lb/yd^3)
1-1/2	467	515
1	57	535
3/4	67	560
1/2	7	580
3/8	8	600

*For nominal maximum coarse aggregate size not indicated, interpolate the minimum cementitious material content between nominal sizes shown.

R4.1 — Water-cementitious materials ratio and cementitious material content

R4.1.1 — For concrete exposed to deicing chemicals, the quantity of fly ash, other pozzolans, silica fume, slag, or blended cements used in the concrete is subject to the percentage limits in 4.2.3. Further, in 4.3 for sulfate exposures,[4.1] the pozzolan should be Class F by ASTM C 618, or have been tested by ASTM C 1012[4.2] or determined by service record to improve sulfate resistance.

R4.1.2 — The 350 Code includes the requirements for environmental structures constructed of concrete with properties and characteristics that are suitable for long-term durability. Concrete mixtures using different types of cement, pozzolans, and admixtures should demonstrate low permeability, acceptable durability, workability, compactability, and finishability characteristics based on the requirements in the code. Upper and lower limits on the percentage of pozzolans that would be advisable for the durability of environmental structures need to be considered.

4.2 — Freezing and thawing exposures

4.2.1 — Concrete exposed to freezing and thawing or deicing chemicals shall be air-entrained with air content indicated in Table 4.2.1. Tolerance on air content as delivered shall be ±1.5 percent. For specified compressive strength f_c' greater than 5000 psi, reduction of air content indicated in Table 4.2.1 by 1.0 percent shall be permitted.

R4.2 — Freezing and thawing exposures

R4.2.1 — A table of required air contents for freezing-and-thawing resistant concrete is included in the code based on **"Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete"** (ACI 211.1).[4.3] Values are provided for both severe and moderate exposure, depending on the exposure to moisture or deicing salts. Entrained air will not protect concrete containing coarse aggregates that undergo disruptive volume changes when frozen in a saturated condition. In Table 4.2.1, a severe exposure is where the concrete in a cold climate may be in almost continuous contact with moisture before freezing, or where deicing salts are used. Examples are water tanks, tanks storing brackish water, and pavements. A moderate exposure is where the concrete in a cold climate will be only occasionally exposed to moisture prior to freezing, and where no deicing salts are used. Examples are tanks containing water or wastewater and certain exterior walls, beams, girders, and slabs not in direct contact with soil. Examples of severe exposure for water tanks are aeration basins and clarifier tanks, which incorporate walkways that are subject to the application of deicing chemicals. Section 4.2.1 permits an air content of 1 percentage point lower for concrete with f_c' greater than 5000 psi. Such high-strength

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TABLE 4.2.1 — TOTAL AIR CONTENT FOR FROST-RESISTANT CONCRETE

Nominal maximum aggregate size, in.*	Air content, percent	
	Severe exposure	Moderate exposure
3/8	7-1/2	6
1/2	7	5-1/2
3/4	6	5
1	6	4-1/2
1-1/2	5-1/2	4-1/2
2†	5	4
3†	4-1/2	3-1/2

*See ASTM C 33 for tolerance on oversize for various nominal maximum size designations.

†These air contents apply to total mixture, as for the preceding nominal maximum aggregate sizes. When testing these concretes, however, aggregate larger than 1-1/2 in. is removed by handpicking or sieving and air content is determined on the minus 1-1/2 in. fraction of mixture (tolerance on air content as delivered applies to this value). Air content of total mixture is computed from the value determined on the minus 1-1/2 in. fraction.

4.2.2 — Concrete that will be subject to the exposures given in Table 4.2.2 shall conform to the corresponding maximum water-cementitious materials ratios and minimum specified concrete compressive strength requirements of that table. In addition, concrete that will be exposed to deicing chemicals shall conform to the limitations of 4.2.3.

TABLE 4.2.2—REQUIREMENTS FOR SPECIAL EXPOSURE CONDITIONS

Exposure condition	Maximum water-cementitious materials ratio, by weight*	Minimum f_c', psi*
Concrete intended to have low permeability when exposed to water, wastewater, and corrosive gasses	0.45	4000
Concrete exposed to freezing and thawing in a saturated condition or to deicing chemicals	0.42	4500
Concrete exposed to corrosive chemicals other than deicing chemicals	0.42	4500
For corrosion protection of reinforcement in concrete exposed to chlorides in tanks containing brackish water and concrete exposed to deicing chemicals, seawater, or spray from seawater	0.40	5000

*A lower water-cementitious material ratio or higher strength may be required for durability of concrete exposed to sulfates (Table 4.3.1).

4.2.3 — For concrete exposed to deicing chemicals, the maximum weight of fly ash, other pozzolans, silica fume, or slag that is included in the concrete shall not exceed the percentages of the total weight of cementitious materials given in Table 4.2.3.

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concretes will have lower water-cementitious materials ratios and porosity and, therefore, improved freezing-and-thawing resistance.

In some severe freezing-and-thawing environments, air entrainment may not be adequate and it may be desirable to protect the concrete from an excessive number of freezing-and-thawing cycles or from reaching near-saturated conditions. Additional precautions may be required (see Reference 4.4).

R4.2.2 — Maximum water-cementitious materials ratios are not specified for lightweight aggregate concrete because determination of the absorption of these aggregates is uncertain, making calculation of water-cementitious materials ratio uncertain. The use of a minimum specified strength will ensure the use of a high-quality cement paste. For normalweight aggregate concrete, use of both minimum strength and maximum water-cementitious materials ratio provide additional assurance that this objective is met.

R4.2.3 — Section 4.2.3 and Table 4.2.3 establish limitations on the amount of fly ash, other pozzolans, silica fume, and slag that can be included in concrete exposed to deicing chemicals.[4.5-4.7] Recent research has demonstrated that the use of fly ash, slag, and silica fume produce concrete with a finer pore structure and, therefore, lower permeability.[4.8-4.10]

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TABLE 4.2.3 — REQUIREMENTS FOR CONCRETE EXPOSED TO DEICING CHEMICALS

Cementitious materials	Maximum percent of total cementitious materials by weight*
Fly ash or other pozzolans conforming to ASTM C 618	25
Slag conforming to ASTM C 989	50
Silica fume conforming to ASTM C 1240	10
Total of fly ash or other pozzolans, slag, and silica fume	50†
Total of fly ash or other pozzolans and silica fume	35†

*The total cementitious material also includes ASTM C 150, C 595, and C 845 cement.
The maximum percentages above shall include:
(a) Fly ash or other pozzolans present in Type IP or I(PM) blended cement, ASTM C 595;
(b) Slag used in the manufacture of a IS or I(SM) blended cement, ASTM C 595;
(c) Silica fume, ASTM C 1240, present in a blended cement
†Fly ash or other pozzolans and silica fume shall constitute no more than 25 and 10 percent, respectively, of the total weight of the cementitious materials.

4.3 — Sulfate exposures

4.3.1 — Concrete exposed to water or wastewater solutions or soils containing sulfates shall conform to requirements of Table 4.3.1 or shall be concrete made with a cement that provides sulfate resistance and that has a maximum water-cementitious materials ratio and minimum compressive strength from Table 4.3.1.

R4.3 — Sulfate exposures

R.4.3.1 — Concrete exposed to injurious concentrations of sulfates from soil, water, and wastewater should be made with a sulfate-resisting cement and a low water-cementitious materials ratio. A study (see Reference 4.11) showed that reducing the ratio of water to total cementitious materials, thereby reducing permeability, is a major factor in increasing concrete's resistance to sulfate attack. Table 4.3.1 lists the appropriate types of cement and the maximum water-cementitious materials ratios and minimum compressive strengths for various exposure conditions. In selecting a cement for sulfate resistance, one of the principal considerations is its C_3A (tricalcium aluminate) content. For moderate exposures, Type II cement is limited to a maximum C_3A content of 8.0 percent under ASTM C 150.[4.12]

The composition requirement for moderate sulfate resistance of blended cements such as Type IP and IS (equivalent to Type II portland cements) in the original specification, ASTM C 595-86,[4.13] was revised in 1989. Section 6.20, Moderate Sulfate Resistance, was deleted, and a performance requirement was specified in ASTM C 595, Table 2, and in ASTM C 1157,[4.14] Table 1, for expansion at 180 days

TABLE 4.3.1—REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate exposure	Water soluble sulfate (SO_4) in soil, percent by weight	Sulfate (SO_4) in water, ppm	Cement type	Maximum water-cementitious ratio, by weight*	Minimum specified compressive strength f_c', psi*
Negligible	0.00-0.10	0-150	—	0.45	—
Moderate†	0.10-0.20	150-1500	II, IP(MS), IS(MS), I(PM)(MS), I(SM)(MS)	0.42	4500
Severe	0.20-2.00	1500-10,000	V	0.40	5000
Very severe‡	Over 2.00	Over 10,000	V plus pozzolan§	0.40	5000

*A lower water-cementitious materials ratio or higher strength may be required for corrosion protection for concrete exposed to chlorides (Table 4.2.2).
†Seawater.
‡Additional corrosion barriers such as coatings or liners shall be required for very severe exposure.
§Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

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for moderate and high sulfate resistance, using the new test method, ASTM C 1012.[4.2] ASTM C 595 and C 1157 provide specification limits associated with testing specified in ASTM C 1012.[4.2]

The appropriate types under ASTM C 595 are IP (MS), IS (MS), I (PM) (MS), and I (SM) (MS). For severe exposures, Type V cement with a maximum C_3A content of 5 percent is required. In certain areas, the C_3A content of other available types such as Type III or Type I may be less than 8 or 5 percent and are usable in moderate or severe sulfate exposures. Note that sulfate-resisting cement will not increase resistance to some chemically aggressive solutions, for example, ammonium nitrate.

The judicious employment of a good-quality fly ash (ASTM C 618, Class F) or granulated blast-furnace slag (ASTM C 989) has also been shown to improve the sulfate resistance of concrete. Certain Type IP and IS cements made by blending Class F fly ash or slag with portland cement having a tricalcium aluminate (C_3A) content greater than 8 percent can provide sulfate resistance for moderate exposures. See ACI 232.1R[4.15] and 232.2R[4.16] for more information on fly ash in concrete.

A note to Table 4.3.1 lists seawater as "moderate exposure," even though it generally contains more than 1500 ppm SO_4. In seawater exposures, other types of cement with C_3A up to 10 percent may be used if the maximum water-cementitious materials ratio is reduced to 0.40.

Test method ASTM C 1012[4.2] can be used to evaluate the sulfate resistance of mixtures using combinations of cementitious materials.

In addition to the proper selection of cement, other requirements for durable concrete exposed to concentrations of sulfate, such as low water-cementitious materials ratio, strength, adequate air entrainment, low slump, adequate consolidation, uniformity, adequate cover of reinforcement, and sufficient moist curing to develop the potential properties of the concrete, are essential.

4.3.2 — Calcium chloride as an admixture shall not be used in concrete.

4.4 — Corrosion protection of metals

4.4.1 — For corrosion protection of reinforcement in concrete, maximum water soluble chloride ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the ingredients including water, aggregates, cementitious materials, and admixtures shall not exceed the limits of Table 4.4.1. When testing is performed to determine water soluble chloride ion content, test procedures shall conform to ASTM C 1218.

R4.4 — Corrosion protection of metals

R4.4.1 — Additional information on the effects of chlorides on the corrosion of reinforcing steel is given in **"Guide to Durable Concrete"** reported by ACI Committee 201[4.17] and **"Corrosion of Metals in Concrete"** reported by ACI Committee 222.[4.18] Test procedures must conform to those given in ASTM C 1218. An initial evaluation may be obtained by testing individual concrete ingredients for total chloride ion content. If total chloride ion content, calculated on the basis of concrete proportions, exceeds those

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TABLE 4.4.1—MAXIMUM CHLORIDE ION CONTENT FOR CORROSION PROTECTION OF REINFORCEMENT

Type of member	Maximum water soluble chloride ion (Cl^-) in concrete, percent by weight of cement
Prestressed concrete	0.06
Reinforced concrete	0.10

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permitted in Table 4.4.1, it may be necessary to test samples of the hardened concrete for water soluble chloride ion content described in the guide. Some of the total chloride ions present in the ingredients will either be insoluble or will react with the cement during hydration and become insoluble under the test procedures described in ASTM C 1218.

When concretes are tested for soluble chloride ion content, the tests should be made at an age of 28 to 42 days. The limits in Table 4.4.1 are to be applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.

The chloride ion limits in Table 4.4.1 differ from those recommended in ACI 318, 201.2R[4.17] and 222R.[4.18] Table 4.4.1 shows chloride ion limit for two types of members. The limit for maximum chloride ion content was lowered for environmental engineering concrete structures, due to their greater susceptibility to corrosion of metals, when experiencing prolonged exposures to chloride-saturated liquids.

When epoxy- or zinc-coated bars are used, the limits in Table 4.4.1 may be more restrictive than necessary.

4.4.2 — If concrete with reinforcement will be exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources, requirements of Table 4.2.2 for water-cementitious materials ratio and concrete strength, and the minimum concrete cover requirements of 7.7 shall be satisfied. See 18.16 for unbonded prestressing tendons.

R4.4.2 — If concretes with reinforcement are exposed to external sources of chlorides, the water-cementitious materials ratio, and specified compressive strength f_c' of 4.2.2 are the minimum requirements that must be considered. The designer should evaluate conditions in structures where chlorides may be applied. In such conditions, greater than the minimums required in 7.7 may be desirable. Use of slag meeting ASTM C 989[4.19] or fly ash meeting ASTM C 618 and increased levels of specified strength provide increased protection. Use of silica fume meeting ASTM C 1240[4.20] with an appropriate high-range water reducer, ASTM C 494,[4.21] Types F and G, or ASTM C 1017[4.22] can also provide additional protection.[4.23] Performance tests for chloride permeability by ASTM C 1202[4.24] (of concrete mixtures before use) will also provide additional assurance for chloride resistance.

4.4.3 — Where conditions exist that reduce the corrosion passive layer around the reinforcing steel or directly corrode the reinforcement, the reinforcing steel shall be protected.

R4.4.3 — The corrosive conditions that require protection depend on the chemicals used and the domestic and industrial wastes encountered. Refer to 4.5.2. The type of protection employed against chemical attack will also vary according to the kind and concentration of the chemical, frequency of contact, and physical conditions such as temperature, pressure, carbonation, mechanical wear or abrasion, and freezing-and-thawing cycles.

4.4.4 — When unbonded prestressing tendons are used in environmental structures, they shall be of the type that completely encapsulates the prestressing steel and its anchorages. Furthermore, all voids in sleeves or caps shall be completely filled with a corrosion-protective material.

R4.4.4 — For additional precautions against corrosion of unbonded tendons, see 18.16. Refer to R18.21.4 for optional additional protection of unbonded tendon anchorages.

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4.4.5 — Contact between dissimilar metals shall be avoided. Isolators shall be placed between different metals.

4.4.6 — When epoxy-coated reinforcement is used, precautions shall be taken to maintain the integrity of the coating.

4.5 — Chemical effects

4.5.1 — Concrete protection against chemicals

4.5.1.1 — Concrete subject to attack by chemical solutions or corrosive gases shall be protected in accordance with 4.5.1.2, 4.5.1.3, and 4.5.1.4.

4.5.1.2 — Proportion concrete with the appropriate type of cement; batch, mix, place, consolidate, finish, and cure the concrete to provide a liquid-tight and gas-tight structure.

4.5.1.3 — Concrete exposed to sulfate-containing solutions or soils shall meet the requirements of 4.3.

4.5.1.4 — Concrete exposed to chemical attack by sulfate or corrosive chemical solutions or gases shall be protected as follows:

(a) Concrete exposed to copper sulfate and/or ferric sulfate shall be made with sulfate-resistant cement or shall be given a protective coating or liner in accordance with 4.7.

(b) Concrete shall be protected against corrosive chemicals with a protective coating, in accordance with 4.7.

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R4.4.6 — Guidance is given in the CRSI Maintenance Guide, **"Field Handling Techniques for Epoxy-coated Rebar at the Job Site."**[4.25]

R4.5 — Chemical effects

R4.5.1.1 — Facilities and structures such as, but not limited to, the following are to be protected when exposed to chemical attack:

(a) Water treatment plants;

(b) Domestic and industrial wastewater treatment plants;

(c) Storage tanks and reservoirs;

(d) Water and wastewater pump stations;

(e) Conduits, sewers, manholes, and junction chambers;

(f) Hazardous materials containment structures.

R4.5.1.2 — Pozzolans such as fly ash and silica fume, when added to the concrete mixture, have been found to decrease the concrete's permeability and increase its resistance to chemical attack.

Refer to ACI 350.1[4.26] for liquid-tightness requirements.

R4.5.1.4 — Some of the most common chemicals that may be found in liquids contained by or in direct contact with environmental concrete structures are included in the following lists. Unless otherwise indicated, the maximum temperature of the chemicals is 120 °F when selecting a protection system. Protection of the concrete may be required where some of these materials contact concrete surfaces. Where concentrations are presented, they are indicated as percent by weight.

Group 1

These chemicals are not considered harmful to concrete, but are listed because, in some instances, treatment is desired to prevent the absorption of liquids into the concrete that may react with other chemicals in the future.

Activated silica, when not agitated;

Anhydrous ammonia (gas);

Aqua ammonia at up to 29.4 percent;

Bentonite;

Calcium carbonate;

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Calcium hydroxide;

Calcium oxide;

Carbon dioxide (gas);**

Chlorine, gas;

Chlorinated water;

Diatomaceous earth;

Dolomitic hydrated lime;

Dolomitic lime;

Hydrogen (gas);

Methanol;

Oxygen (gas);

Ozone (gas);

Polymer (emulsion);

Polymer (Mannich);

Polyphosphate (zinc orthophosphate);

Potassium hydroxide when 15 percent or less;*

Sodium bicarbonate;**

Sodium carbonate (soda ash);

Sodium hydroxide when less than 20 percent;

Sodium silicate;

Sulfur dioxide (gas);

Tetrasodium pyrophosphate;

Trisodium phosphate.

* Caution is recommended with respect to alkali-reactive aggregates. Refer to Sections 3.3.3/R3.3.3 of this standard.
** Carbonation may occur.

Group 2

These chemicals will stain concrete and, in some instances where appearance is a concern, treatment is required to prevent the staining.

Activated carbon (except when agitated, then place in Group 3);

Potassium permanganate.

Group 3

These chemicals are corrosive to concrete. Based on the rate of corrosion, the chemicals have been listed in one of three subgroups. Concrete exposed to any of the following

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chemicals needs to be given a protective lining or topping in accordance with 4.7. The concentrations shown are weight percent, and are the typical maximum concentration of the chemical delivered and used at the facility.

Group 3A — Slow corrosion of concrete

Acetic acid;

Ammonia silicofluoride;

Calcium hypochlorite;

Carbon dioxide (solution = carbonic acid);

Chlorine (solution) at 0.35 percent;

Chlorine dioxide (solution);

Cyanide;

Disodium phosphate;

Ferrous chloride at up to 35 percent;

Hydrogen sulfide;

Iodine;

Phosphoric acid at 85 percent;

Potassium hydroxide when more than 15 percent;

Sodium fluoride;

Sodium hexametaphosphate;

Sodium thiosulfate;

Sulfur dioxide at 1 percent (solution).

Group 3B — Corrosion of concrete

Activated carbon (when not agitated in Group 1);

Aluminum sulfate (alum);

Ammonium nitrate;

Ammonium sulfate;

Bromine;

Citric acid at 34 percent;

48.5 percent aluminum sulfate;

50 percent ferric sulfate;

45 percent ferric chloride;

Copper sulfate, will also stain;

Ferric chloride at up to 45 percent;

Ferric sulfate at up to 50 percent;

Ferrous sulfate at 19 percent;

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Hydrogen peroxide at 50 percent;

Manganese sulfate;

Potassium aluminum sulfate;

Potassium sulfate;

Sodium aluminate at 40 percent;

Sodium bisulfate;

Sodium bisulfite at 38 percent;

Sodium chloride (dry);

Sodium chloride (solution);

Sodium chlorite at 25 percent;

Sodium hydroxide when greater than 20 percent and up to 150 °F;

Sodium hypochlorite at up to 15 percent;

Sodium silicofluoride (dry and wet);

Sodium sulfate;

Sodium sulfite;

Zinc sulfate.

Group 3C — Rapid corrosion of concrete

Aluminum chloride (solution);

Hydrochloric acid at up to 37 percent and 150 °F;

Hydrofluosilicic acid at up to 30 percent;

Polyaluminum chloride;

Sulfuric acid up to 98 percent and 150 °F.

Table 2.5.2 of ACI 515.1R-79 (85)[4.27] provides additional information on the effect of chemicals on concrete. Portland Cement Association Concrete Information Bulletin on **"Effects of Substances on Concrete and Guide to Protective Treatments"**[4.28] is also a reference.

Special attention should be given to chemicals in solution under high temperature, pressure, or both, which may accelerate the corrosion process. In such cases, a liquid or gas under pressure will be forced through permeable or slightly cracked concrete, and will come in contact with reinforcing steel or embedded metals. These chemicals may also attack the concrete and, at increased temperatures, the attack may occur more rapidly.

Hydrogen sulfide gases are oxidized aerobically to sulfates, which in the presence of moisture and oxygen form sulfuric acid. The sulfuric acid then attacks the calcium hydroxide in the concrete to form calcium sulfate (gypsum), which usually has the effect of reducing the pH of the surface

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concrete, thereby reducing the protective alkaline environment around the reinforcing steel or metals, thus allowing the corrosion process to proceed. Refer to **"Sulfides in Wastewater Collection and Treatment Systems,"** ASCE Manual of Practice No. 69.[4.29]

Special cement mortars have been used in acid-resistant linings for tanks and basins. Specific design information should be obtained regarding the mortar's resistance to particular chemicals under varying environmental conditions such as temperature and concentrations of chemicals. For example, certain special acid-resistant mortars will rapidly deteriorate when exposed to caustic solutions having a pH of 7 or greater. Design and specifying information may be obtained from manufacturers and ACI publications such as ACI 515.1R-79(85) [4.27]

Ozone, hydrogen, and oxygen gases have been found to be relatively inert when in contact with concrete. These gases, however, are very harmful to reinforcing steel and embedded metals. Ozone has also been found to be reactive with many waterstops, sealants, coatings, and protection systems.

Carbonation may occur when carbon dioxide in the atmosphere reacts with hydrated portland cement in the concrete and reduces the concrete pH to approximately 9.0. When the pH of the pore fluid in the concrete is above a certain level, a passive layer forms on the surface of the reinforcement. If the pH drops too low, the passive layer breaks down, and reinforcement corrosion may occur. Refer to ACI 201.2R[4.17] and 222R.[4.18]

4.5.2 — Jointing materials, including waterstops, expansion joints, and sealants, shall be resistant to chemical attack for the design life of the facility. Materials shall be tested in accordance with ASTM C 920 and Federal Specification TT-S-00277E for sealants and ASTM D 570, ASTM D 746, ASTM D 1149, and CRD-C572 for PVC waterstops (See 4.8.2).

4.5.3 — Testing for chemical effects

R4.5.3 — Testing for chemical effects

Several recommended tests should be considered before and during the construction of environmental concrete structures:

4.5.3.1 — The composition and temperature of the liquid or gas and its pH shall be tested for aggressiveness to the concrete and to the protective barrier system.

R4.5.3.1 Chemical composition and temperature of the liquid or gas including pH determination.

4.5.3.2 — The adequacy of the protection against chemical effects shall be confirmed by tests. The tests shall determine the need for and the effectiveness of special cements, liners, coatings, and other protective measures.

R4.5.3.2 Concentration of dissolved carbon dioxide in the liquid stream.

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4.5.3.3 — Aggregates shall be tested for reactions to chemical attack in accordance with ASTM C 295.

4.6 — Protection against erosion

4.6.1 — Concrete shall be protected against erosion damage when subjected to cavitation or abrasion.

4.6.2 — For protection against cavitation erosion, at least one of the following shall be used:

(a) Reduce flow velocity and increase pressure by incorporating baffles or similar devices in the structure;

(b) Use structural shapes, surface finishes and tolerances characterized by values of the cavitation index at the condition of incipient cavitation;

(c) Supply air to the flow such that the air-to-water ratio near the solid boundary is approximately 8 percent by volume;

(d) Use erosion-resistant materials conforming to the requirements of 4.6.3.

4.6.3 — Where a structure will be subjected to abrasion erosion, aggregates shall meet requirements of ASTM C 33, concrete shall be tested in accordance with ASTM C 1138, and the concrete shall comply with the following additional requirements:

(a) Minimum f_c' = 5000 psi at 28 days;

(b) Maximum water-cementitious ratio = 0.40;

(c) Maximum air content = 6 percent; if not subject to freezing and thawing, the maximum air content = 3 percent;

(d) Minimum 610 lb of cementitious material per cubic yard of concrete;

(e) Hard, dense, clean aggregates.

Where additional protection is needed, use coatings and liners in accordance with 4.7.1.

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R4.5.3.3 Where concrete is exposed to a more severe chemical attack, such as acids, acid-resistant aggregates, or a coating or liner or both should be used. Refer to ACI 221R.[4.30] Petrographic examination of concrete samples (ASTM C 295[4.31]).

R4.6 — Protection against erosion

Erosion is defined as the progressive disintegration of a solid by abrasion or cavitation.

R4.6.1 — Cavitation erosion damage to concrete surfaces, as evidenced by small surface holes and pits, results from the collapse of bubbles or cavities in a liquid.

Abrasion erosion is defined as erosion caused by abrasion action. Abrasion erosion damage to concrete surfaces results from the wearing, grinding, or rubbing away by silt, sand, gravel, rocks, and other debris passing over the concrete surface of a structure.

R4.6.2 — The cavitation index is a dimensionless measure used in assessing the likelihood of cavitation damage. This index is a function of pressure and velocity. A detailed explanation of the cavitation index and the measures that can be used to minimize or eliminate cavitation erosion is provided in ACI 210R.[4.32]

To determine the required amount of air that is to be added to the flow, either finite element analyses or model tests may need to be performed. See Section 5.3 of ACI 210R.

R4.6.3 — For example, the abrasion erosion resistance of concrete is affected primarily by:

(a) Aggregate properties;

(b) Compressive strength;

(c) Surface finish and treatments;

(d) Curing.

Abrasion-resistant concrete should include the largest maximum size aggregate practical, the maximum amount of the hardest available coarse aggregate, and the lowest practical water-cementitious materials ratio. The abrasion-erosion resistance of concrete containing chert aggregates has been shown to be approximately twice that of concrete containing limestone aggregates (see Fig. 6.1 in ACI 210R). Given a quality, hard aggregate, any practice that produces a stronger paste structure will increase abrasion-erosion resistance. In some cases, where hard aggregates were not available, mixtures using high-range water reducer and

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silica fume have been used to develop very strong concrete; that is, concrete with a compressive strength of approximately 15,000 psi. This approach has also been used to overcome problems with unsatisfactory aggregates.[4.33] Apparently, at these high compressive strengths, the hardened cement paste assumes a greater role in resisting abrasion-erosion damage, and the aggregate quality becomes correspondingly less important.

4.6.4 — Structures exposed to cavitation erosion shall be constructed with high-strength, low water-cementitious material ratio concrete and shall receive surface finishes that are smooth with slight changes of slope in the direction of flow.

R4.6.4 — Surface imperfections have been determined to have caused cavitation damage at low flow velocities. Tolerances meeting ACI 117[4.34] may be satisfactory. Under certain conditions, however, it may be necessary to require more demanding tolerances. ACI 210R provides guidance on design for hydraulic structures subjected to erosion.

Proper selection of materials and of concrete proportions can increase resistance to cavitation erosion, but cannot eliminate it. The only way to eliminate cavitation erosion is to eliminate the factors that cause cavitation. A properly designed high-strength concrete with a low water-cementitious materials ratio has a greater resistance to cavitation erosion. The use of silica fume in concrete has been found to improve its resistance to erosion. Limiting the maximum size aggregate to 1-1/2 in. and using water-reducing admixtures have proven effective in increasing erosion resistance.

4.6.5 — When exposed to cavitation, the reinforcing bars closest to the concrete surface shall be placed parallel to the direction of flow.

R4.6.5 — Where potential for erosion is high, reinforcing bars should be positioned to provide the least resistance to flow should erosion reach the depth of the reinforcement.

4.6.6 — When exposed to cavitation, the adequacy of the protection against concrete erosion by cavitation or abrasion shall be confirmed by tests in accordance with ASTM C 1138.

R4.6.6 — When testing concrete in accordance with ASTM C 1138,[4.35] a concrete mixture should be considered acceptable if a loss of mass of 4 percent or less in 72 hours is not exceeded. A comprehensive treatment of this subject is included in ACI 210R.

4.7 — Coatings and liners

4.7.1 — When concrete is in contact with chemicals or corrosive gases that attack the cement mortar matrix or embedded reinforcing steel, coatings or liners shall be used. Where protective coatings and liners are used to prevent contact of chemical solutions or gases with concrete surfaces, they shall be impervious and shall exhibit good bond.

R4.7 — Coatings and liners

R4.7.1 — Two guides for determining when a particular substance attacks concrete and what acceptable protection treatments are available are ACI 515.1R and **"Effects of Substances on Concrete and Guide to Protective Treatments,"** Portland Cement Association Publication IS001T.[4.36]

Consideration should be given to attack by chemical solutions or gases and biologically induced corrosion from bacteria, fungi, and algae on the concrete elements in selecting compatible protective coatings and liners. In addition, each project should be considered individually because various materials and techniques used from time to time cause new problems of chemical attack. Detailed recommendations are given in ACI 515.1R.

Manufacturers of protective coatings and liners should be consulted for information on bond, anchorage, and the preparation of concrete surfaces, as well as the application of their products.

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It is recommended that form oils, form release agents, and curing components be checked for compatibility with surface coatings where surface coatings are specified.

Follow procedures outlined in ACI 515.1R and International Concrete Repair Institute (ICRI) Guideline No. 03732, **"Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings and Polymer Overlays."**[4.37]

4.7.2 — Surface coatings or liners shall not be used when in contact with ozone gas without testing for compatibility with ozone.

R4.7.2 — Special attention is called to PVC vulnerability in contact with ozone gas.

4.7.3 — Coatings shall be provided for the expected exposure. The effectiveness of the coatings shall be confirmed by tests. Coating thickness shall be measured using film thickness gages. Spark tests for liner joints shall be performed.

R4.7.3 — For additional information on coatings, refer to **"Concrete Structures Containing Hazardous Materials,"** ACI 350.2R.[4.38]

When cavitation or grit in the fluids of the process stream have been shown to cause damage to coatings or linings, testing in accordance with the following ASTM Standards should be accomplished: ASTM D 660,[4.39] D 661,[4.40] D 662,[4.41] and D 4214.[4.42] A loss in thickness of the coating or lining less than 50% over the estimated service life of the structure has been shown to be an acceptable limit for coatings or linings. ACI 210R, Section 9.2.8, refers to high-head erosion tests conducted on polyurethane and neoprene coatings. Testing has indicated problems with similar flexible coatings, when a portion of the coating is torn from the edge of the concrete surface and much of the remaining coating peeled away due to the hydraulic force. For a description of this testing, refer to **"Cavitation Resistance of Some Special Concretes,"** by Houghton, Borge, and Paxton.[4.43]

4.7.4 — Vapor transmission of liners and coatings

Where water vapor transmission (WVT) is dangerous, coatings and liners shall have a WVT less than 1×10^{-6} cm/s (34 g/h/m^2) when tested in accordance with ASTM E 96.

R4.7.4 — Vapor transmission of coatings and liners

While it is an advantage to have "breathable" coatings or liners, in certain structures, such barriers will permit the escape of gases that may be dangerous, especially when mixed with the air. Transmission of gases, other than water vapor, should be tested depending on the use intended.

4.7.5 — Selection of coatings and liners

4.7.5.1 — Coatings and liners shall be resistant to chemicals that are expected to be found in environmental engineering concrete structures, and in contact with them.

4.7.5.2 — Coatings and liners shall be able to bridge existing open cracks or joints in the substrate.

4.7.5.3 — Coating and liner materials shall meet all federal, state, and local standards when used in potable water applications.

R4.7.5 — Selection of coatings and liners

R4.7.5.2 — Cracks or joints may require special treatment before the application of a coating or liner.

R4.7.5.3 — Refer to ANSI/NSF 61[4.44] for potable water applications.

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4.8 — Joints

4.8.1 — General

Movement joints (expansion and contraction) and construction joints shall be designed to prevent cracking, spalling, and reinforcement corrosion. The number, spacings, and details of joints shall be designed taking full account of the physical properties and the ability of the filler, sealant, and waterstop materials to sustain cycles of deformations.

4.8.2 — Waterstops

Materials used for waterstops to stop the flow of liquids or gases shall be able to sustain movement deformations (elongation and contraction) without permanent deformation or failure and shall be resistant to freezing-and-thawing cycles, and temperature and chemical effects.

4.8.3 — Sealants

Joint sealants shall be provided along the exposed perimeter of the joints to exclude liquids or gases and to prevent solids from entering the joint and impairing the functioning of the joint. Sealants shall be designed to sustain the required pressures, temperatures, and movements and shall not debond or degrade under the expected chemical or gas attack and shall be resistant to the required pressures, temperatures, and movements.

4.8.4 — Ozone exposure

Sealants, joint fillers, and waterstops to be used shall be tested for compatibility with ozone. Contraction (control) joints that are formed by wedges or sawcuts shall be filled with sealant to protect the reinforcing steel.

4.8.5 — Shear keys

Where shear keys are used in movement joints, the joint shall be designed to avoid spalling and splitting of the concrete and resulting leakage.

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R4.8 — Joints

R4.8.1 — General

Refer to Chapter 6 and ACI 504R[4.45] for the design and details of movement and construction joints. Bulb-type waterstops are recommended for movement joints. The bulb size and construction of the waterstop influence the capability of the joint to sustain both out-of-plane and in-plane movements while maintaining liquid-tightness.

R4.8.2 — Waterstops

In ozone environments, it may be required to specify stainless steel waterstops. To allow for the expected movement, metal waterstops at expansion joints need to be crimped or bent.

R4.8.3 — Sealants

When selecting a sealant, consideration should be given to the sealant shape factor, surface preparation, and the contact bond strength between the sealant and the concrete or metal assembly.

Non-sag sealants are recommended for submerged service. Polyurethane sealants, single-component, and two-component are recommended. Polysulfide sealants are not recommended when the sealant is in contact with wastewater.

In addition to taste, odor, and toxicity concerns, the sealant for water treatment plants and reservoirs should be resistant to chlorinated water. Consideration should be given to the effects of prolonged exposure to chlorine at normal drinking water concentrations, as well as short-term exposure to chlorine at the high concentrations required for disinfection.

Joint sealants are not expected to function for the entire life of a structure. Owners should be advised of the need to repair, maintain, and reseal joints with proper joint sealants at proper intervals.

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Notes

CHAPTER 5 — CONCRETE QUALITY, MIXING, AND PLACING

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5.0 — Notation

f_c' = specified compressive strength of concrete, psi

f_{cr}' = required average compressive strength of concrete used as the basis for selection of concrete proportions, psi

s = standard deviation, psi

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The requirements for proportioning of concrete mixtures are based on the philosophy that concrete should provide both adequate durability (Chapter 4) and strength. The criteria for acceptance of concrete are based on the philosophy that the code is intended primarily to protect the safety of the public. Chapter 5 describes procedures by which concrete of adequate strength can be obtained, and provides procedures for checking the quality of the concrete during and after its placement in the work.

Chapter 5 also prescribes minimum criteria for mixing and placing concrete.

The purpose of 5.3, together with Chapter 4, is to establish the required mixture proportions, and not to constitute a basis for confirming the adequacy of concrete strength, which is covered in 5.5 (evaluation and acceptance of concrete).

5.1 — General

5.1.1 — Concrete shall be proportioned to provide an average compressive strength as prescribed in 5.3.2 and shall satisfy the durability criteria of Chapter 4. Concrete shall be produced to minimize frequency of strengths below f_c' as prescribed in 5.5.2.3. For concrete designed and constructed with the code, f_c' shall not be less than 4000 psi.

5.1.2 — Requirements for f_c' shall be based on tests of cylinders made and tested as prescribed in 5.5.2.

5.1.3 — Unless otherwise specified, f_c' shall be based on 28-day tests. If other than 28 days, test age for f_c' shall be as indicated in design drawings or specifications.

R5.1 — General

R5.1.1 — The basic premises governing the designation and evaluation of concrete strength are presented. It is emphasized that the average strength of concrete produced must always exceed the specified value of f_c' used in the structural design calculations. This is based on probabilistic concepts, and is intended to ensure that adequate concrete strength will be developed in the structure. The durability requirements prescribed in Chapter 4 must be satisfied in addition to attaining the average concrete strength in accordance with 5.3.2.

5.2 — Selection of concrete proportions

R5.2 — Selection of concrete proportions

Recommendations for selecting proportions for concrete are given in detail in **"Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete"** (ACI 211.1).[5.1] (Provides two methods for selecting and adjusting proportions for normalweight concrete: the estimated weight and absolute volume methods. Example calculations are shown for both methods. Proportioning of heavyweight concrete by the absolute volume method is presented in an appendix.)

Recommendations for lightweight concrete are given in **"Standard Practice for Selecting Proportions for Structural Lightweight Concrete"** (ACI 211.2).[5.2] (Provides a

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method of proportioning and adjusting structural grade concrete containing lightweight aggregates.)

5.2.1 — Proportions of materials for concrete shall be established to provide:

(a) Workability and consistency to permit concrete to be worked readily into forms and around reinforcement under conditions of placement to be employed, without segregation or excessive bleeding;

(b) Resistance to special exposures as required by Chapter 4;

(c) Conformance with strength test requirements of 5.5.

R5.2.1 — The selected water-cementitious materials ratio must be low enough, or the compressive strength high enough, to satisfy both the strength criteria and the special exposure requirements of Chapter 4 of ACI 350. ACI 350 includes provisions for especially severe exposures, such as acids or high temperatures, but is not concerned with aesthetic considerations such as surface finishes. Aesthetic considerations are beyond the scope of this code and should be covered specifically in the project specifications. Concrete ingredients and proportions should be selected to meet the minimum requirements stated in this code and the additional requirements of the contract documents.

5.2.2 — Where different materials are to be used for different portions of proposed work, each combination shall be evaluated.

5.2.3 — Concrete proportions, including water-cementitious materials ratio, shall be established on the basis of field experience, trial mixtures, or both, with materials to be employed, except as required by Chapter 4 of this code.

R5.2.3. — ACI 350 restricts the methods for selecting concrete mixture proportions to field experience or laboratory trial mixtures. Proportioning of concrete on the basis of the water-cementitious materials ratio as described in 5.4 of ACI 318 is not permitted by ACI 350.

5.3 — Proportioning on the basis of field experience, trial mixtures, or both

R5.3 — Proportioning on the basis of field experience, trial mixtures, or both

In selecting a suitable concrete mixture, there are three basic steps. The first is the determination of the standard deviation, and the second, the determination of the required average strength. The third step is the selection of mixture proportions required to produce that average strength, either by conventional trial mixture procedures or by a suitable experience record. Figure R5.3 is a flow chart outlining the mixture selection and documentation procedure. Figure R5.3 of ACI 318R is revised in this commentary to exclude the option of mixture proportioning based on the water-cementitious materials ratio.

The mixture selected must yield an average strength appreciably higher than the specified strength f_c'. The degree of mixture overdesign depends on the variability of the test results.

5.3.1 — Standard deviation

R5.3.1 — Standard deviation

When a concrete production facility has a suitable record of 30 consecutive tests of similar materials and conditions expected, the standard deviation is calculated from those results in accordance with the following formula

$$s = \left[\frac{\Sigma(X_i - \bar{X})^2}{(n-1)}\right]^{1/2}$$

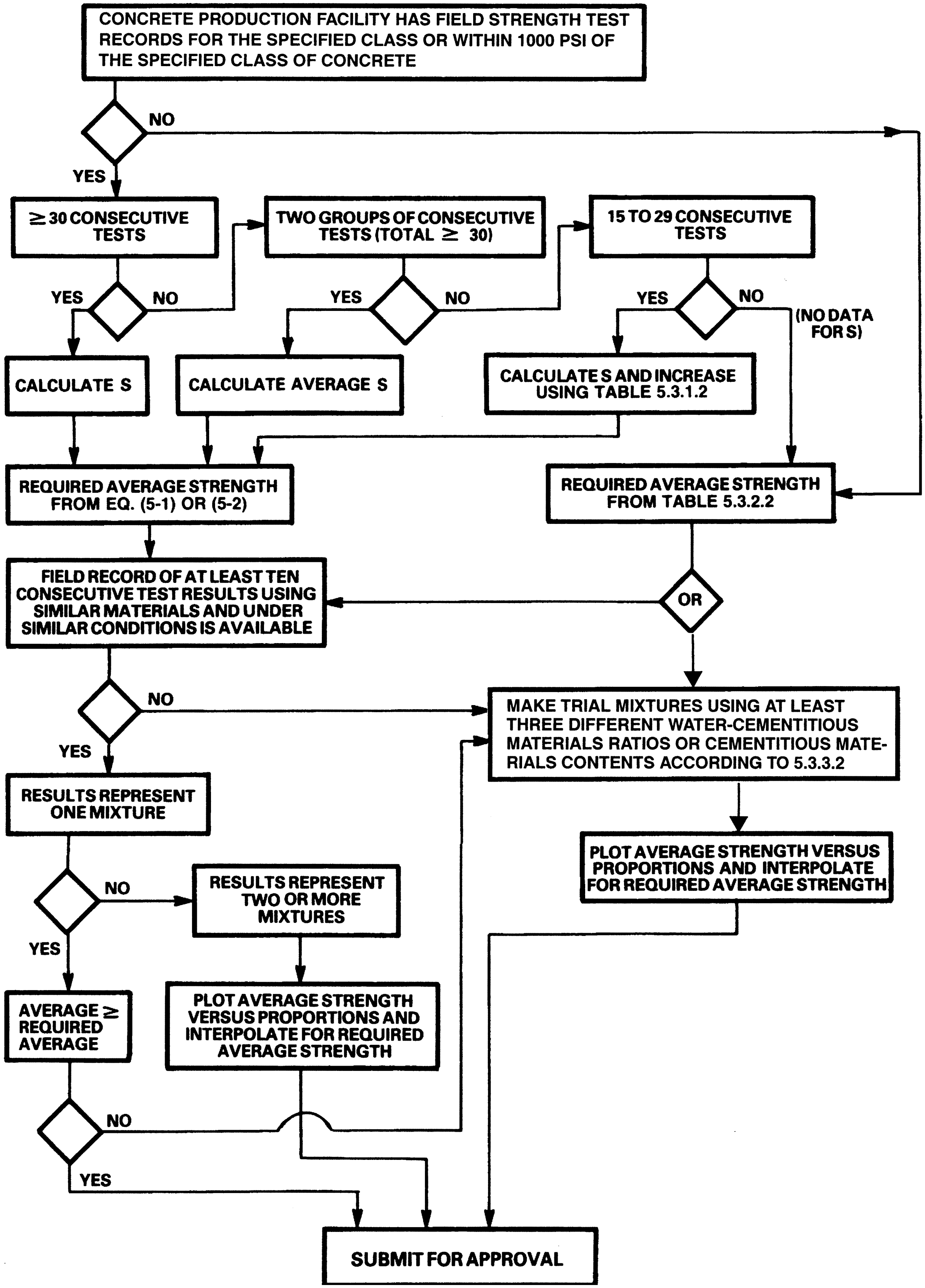

Fig. R5.3—Flow chart for selection and documentation of concrete proportions.

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where

s = standard deviation, psi;
X_i = individual strength tests as defined in 5.5.1.4;
$\overline{X}$ = average of n strength test results; and
n = number of consecutive strength tests.

The standard deviation is used to determine the average strength required in 5.3.2.1.

If two test records are used to obtain at least 30 tests, the standard deviation used shall be the statistical average of the values calculated from each test record in accordance with the following formula

$$\bar{s} = \left[\frac{(n_1 - 1)(s_1)^2 + (n_2 - 1)(s_2)^2}{(n_1 + n_2 - 2)}\right]^{1/2}$$

where

$\bar{s}$ = statistical average standard deviation where two test records are used to estimate the standard deviation;
s_1, s_2 = standard deviations calculated from two test records, 1 and 2, respectively; and
n_1, n_2 = number of tests in each test record, respectively.

If less than 30, but at least 15, tests are available, the calculated standard deviation is increased by the factor given in Table 5.3.1.2. This procedure results in a more conservative (increased) required average strength. The factors in Table 5.3.1.2 are based on the sampling distribution of the standard deviation and provide protection (equivalent to that from a record of 30 tests) against the possibility that the smaller sample underestimates the true or universe population standard deviation.

The standard deviation used in the calculation of required average strength must be developed under conditions "similar to those expected" [see 5.3.1.1(a)]. This requirement is important to ensure acceptable concrete.

Concrete for background tests to determine standard deviation is considered to be "similar" to that required if made with the same general types of ingredients under no more restrictive conditions of control over material quality and production methods than on the proposed work, and if its specified strength does not deviate more than 1000 psi from the f_c' required [see 5.3.1.1(b)]. A change in the type of concrete or a major increase in the strength level may increase the standard deviation. Such a situation might occur with a change in type of aggregate (that is, from natural aggregate to lightweight aggregate or vice versa) or a change from non-air-entrained concrete to air-entrained concrete. Also, there may be an increase in standard deviation when the average strength level is raised by a significant amount, although the increment of increase in standard deviation should be somewhat less than directly proportional to the strength increase. When there is reasonable

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5.3.1.1 — Where a concrete production facility has test records, a standard deviation shall be established. Test records from which a standard deviation is calculated:

(a) Shall represent materials, quality control procedures, and conditions similar to those expected and changes in materials and proportions within the test records shall not have been more restricted than those for proposed work;

(b) Shall represent concrete produced to meet a specified strength or strengths f_c' within 1000 psi of that specified for proposed work;

(c) Shall consist of at least 30 consecutive tests or two groups of consecutive tests totaling at least 30 tests as defined in 5.5.1.4, except as provided in 5.3.1.2;

(d) Shall consist of data that is not more than 12 months old.

5.3.1.2 — Where a concrete production facility does not have test records meeting requirements of 5.3.1.1, but does have a record based on 15 to 29 consecutive tests, a standard deviation shall be established as the product of the calculated standard deviation and modification factor of Table 5.3.1.2. To be acceptable, test record shall meet requirements (a) and (b) of 5.3.1.1,

TABLE 5.3.1.2 — MODIFICATION FACTOR FOR STANDARD DEVIATION

No. of tests*	Modification factor for standard deviation†
Less than 15	Use table 5.3.2.2
15	1.16
20	1.08
25	1.03
30 or more	1.00

*Interpolate for intermediate numbers of tests.

†Modified standard deviation to be used to determine required average strength f_{cr}' from 5.3.2.1.

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doubt, any estimated standard deviation used to calculate the required average strength should always be on the conservative (high) side.

Note that the code uses the standard deviation in pounds per square inch instead of the coefficient of variation in percent. The latter is equal to the former expressed as a percent of the average strength.

When a suitable record of test results is not available, the average strength must exceed the design strength by an amount that ranges from 1000 to 1400 psi, depending on the design strength. See Table 5.3.2.2.

Even when the average strength and standard deviation are of the levels assumed, there will be occasional tests that fail to meet the acceptance criteria prescribed in 5.5.2.3 (perhaps 1 test in 100).

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and represent only a single record of consecutive tests that span a period of not less than 45 calendar days.

5.3.2 — Required average strength

5.3.2.1 — Required average compressive strength f'_{cr} used as the basis for selection of concrete proportions shall be determined from Table 5.3.2.1 using a standard deviation calculated in accordance with 5.3.1.1 or 5.3.1.2.

TABLE 5.3.2.1 — REQUIRED AVERAGE COMPRESSIVE STRENGTH WHEN DATA ARE AVAILABLE TO ESTABLISH A STANDARD DEVIATION

Specified compressive strength f'_c, psi	Required average compressive strength f'_{cr}, psi
$f'_c \leq 5000$	Use the larger value computed from Eq. (5-1) and (5-2) $f'_{cr} = f'_c + 1.34s$ (5-1) $f'_{cr} = f'_c + 2.33s - 500$ (5-2)
Over 5000	Use the larger value computed from Eq. (5-1) and (5-3) $f'_{cr} = f'_c + 1.34s$ (5-1) $f'_{cr} = 0.90f'_c + 2.33s$ (5-3)

5.3.2.2 — When a concrete production facility does not have field strength test records current within 1 year for calculation of standard deviation meeting requirements of 5.3.1.1 or 5.3.1.2, required average strength f'_{cr} shall be determined from Table 5.3.2.2 and documentation of average strength shall be in accordance with requirements of 5.3.3.

TABLE 5.3.2.2 — REQUIRED AVERAGE COMPRESSIVE STRENGTH WHEN DATA ARE NOT AVAILABLE TO ESTABLISH A STANDARD DEVIATION

Specified compressive strength f_c', psi	Required average compressive strength f'_{cr}, psi
4000 to 5000	$f_c' + 1200$
Over 5000	$1.10f_c' + 700$

5.3.3 — Documentation of average strength

Documentation that proposed concrete proportions will produce an average compressive strength equal to or greater than required average compressive strength (See 5.3.2) shall consist of a field strength test record, several strength test records, or trial mixtures.

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R5.3.2 — Required average strength

R5.3.2.1 — Once the standard deviation has been determined, the required average compressive strength is obtained from the larger value computed from Eq. (5-1) and (5-2) for f'_c of 5000 psi or less, or the larger value computed from Eq. (5-1) and (5-3) for f'_c over 5000 psi. Equation (5-1) is based on a probability of 1-in-100 that the average of three consecutive tests may be below the specified compressive strength f'_c. Equation (5-2) is based on a similar probability that an individual test may be more than 500 psi below the specified compressive strength f'_c. Equation (5-3) is based on the same 1-in-100 probability that an individual test may be less than $0.90 f'_c$. These equations assume that the standard deviation used is equal to the population value appropriate for an infinite or very large number of tests. For this reason, use of standard deviations estimated from records of 100 or more tests is desirable. When 30 tests are available, the probability of failure will likely be somewhat greater than 1-in-100. The additional refinements required to achieve the 1-in-100 probability are not considered necessary, because of the uncertainty inherent in assuming that conditions operating when the test record was accumulated will be similar to conditions when the concrete will be produced.

Additionally, the change adopted in ACI 318-77 (requiring action to increase the average strength whenever either of the acceptance criteria of 5.5.2.3 is not met) is considered to provide significant additional protection against subsequent low tests.

R.5.3.3 — Documentation of average strength

Once the required average strength f'_{cr} is known, the next step is to select mixture proportions that will produce an average strength at least as great as the required average strength, and also meet special exposure requirements of Chapter 4. The documentation may consist of a strength test

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5.3.3.1 — When test records are used to demonstrate that proposed concrete proportions will produce the required average strength f'_{cr} (See 5.3.2), such records shall represent materials and conditions similar to those expected. Changes in materials, conditions, and proportions within the test records shall not have been more restricted than those for proposed work. For the purpose of documenting average strength potential, test records consisting of less than 30 but not less than 10 consecutive tests are acceptable provided test records encompass a period of time not less than 45 days. Required concrete proportions shall be permitted to be established by interpolation between the strengths and proportions of two or more test records each of which meets other requirements of this section.

5.3.3.2 — When an acceptable record of field test results is not available, concrete proportions established from trial mixtures meeting the following restrictions shall be permitted:

(a) Combination of materials shall be those for proposed work;

(b) Trial mixtures having proportions and consistencies required for proposed work shall be made using at least three different water-cementitious materials ratios or cementitious materials contents that will produce a range of strengths encompassing the required average strength f'_{cr};

(c) Trial mixtures shall be designed to produce a slump within ±0.75 in. of maximum permitted, and for air-entrained concrete, within ±0.5 percent of maximum allowable air content;

(d) For each water-cementitious materials ratio or cementitious materials content, at least three test cylinders for each test age shall be made and cured in accordance with "Method of Making and Curing Concrete Test Specimens in the Laboratory" (ASTM C 192). Cylinders shall be tested at 28 days or at test age designated for determination of f_c';

(e) From results of cylinder tests, a curve shall be plotted showing relationship between water-cementitious materials ratio or cementitious materials content and compressive strength at designated test age;

(f) Maximum water-cementitious materials ratio or minimum cementitious materials content for concrete to be used in proposed work shall be that shown by the curve to produce the average strength required by 5.3.2, unless a lower water-cementitious materials ratio or higher strength is required.

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record, several strength test records, or suitable laboratory trial mixtures. Generally, if a test record is used, it will be the same one that was used for computation of the standard deviation. If this test record shows either lower or higher average strength than the required average strength, however, different proportions may be necessary or desirable. In such instances, the average from a record of as few as 10 tests may be used, or the proportions may be established by interpolation between the strengths and proportions of two such records of consecutive tests. All test records for establishing proportions necessary to produce the average strength must meet the requirements of 5.3.3.1 for "similar materials and conditions."

For strengths over 5000 psi where the average strength documentation is based on laboratory trial mixtures, it may be appropriate to increase f'_{cr} calculated in Table 5.3.2.2 to allow for a reduction in strength from laboratory trials to actual concrete production.

The ACI 318-71 code required trial mixtures to be mixed at the maximum permitted slump and air content. Since 1977, the ACI 318 code has provided tolerances at the maximum permissible slump and air content. The code text makes it clear that these tolerances on slump and air content apply only to the trial mixtures and not to records of field tests or to later production of the concrete in the field.

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5.4 — Average strength reduction

As data become available during construction, it shall be permitted to reduce the amount by which f_{cr}' must exceed the specified value of f_c', provided:

(a) 30 or more test results are available and average of test results exceeds that required by 5.3.2.1, using a standard deviation calculated in accordance with 5.3.1.1; or

(b) 15 to 29 test results are available and average of test results exceeds that required by 5.3.2.1 using a standard deviation calculated in accordance with 5.3.1.2; and

(c) Special exposure requirements of Chapter 4 are met.

5.5 — Evaluation and acceptance of concrete

R5.5 — Evaluation and acceptance of concrete

Once the mixture proportions have been selected and the job started, the criteria for evaluation and acceptance of the concrete can be obtained from 5.5.

An effort has been made in the code to provide a clear-cut basis for judging the acceptability of the concrete, as well as to indicate a course of action to be followed when the results of strength tests are not satisfactory.

5.5.1 — Frequency of testing

5.5.1.1 — Samples for strength tests of each class of concrete placed each day shall be taken not less than once a day, nor less than once for each 100 yd^3 of concrete, nor less than once for each 5000 ft^2 of surface area for slabs or walls.

R5.5.1 — Frequency of testing

R5.5.1.1 — The following three criteria establish the required sampling frequency for each class of concrete:

(a) Once each day a given class is placed, nor less than

(b) Once for each 100 yd^3 of each class placed each day, nor less than

(c) Once for each 5000 ft^2 of slab or wall surface area placed each day.

In calculating surface area, only one side of the slab or wall should be considered. If the average wall or slab thickness is less than 6-1/2 in., criterion (c) will require more frequent sampling than once for each 100 yd^3 placed.

The change in sampling rate from the ACI 318 code is based on the need to ensure that the concrete placed in environmental concrete structures comply with the more stringent requirements of the ACI 350 Code. Generally, environmental engineering concrete structures will have a large volume of high-strength concrete that has a large amount of reinforcement. Increased difficulties with replacing defective concrete in liquid-containing structures are related to the large volume of concrete that will need to be replaced. It is very important that a different level of quality control be conducted.

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5.5.1.2 — On a given project, if total volume of concrete is such that frequency of testing required by 5.5.1.1 would provide less than five strength tests for a given class of concrete, tests shall be made from at least five randomly selected batches or from each batch if fewer than five batches are used.

5.5.1.3 — When total quantity of a given class of concrete is less than 50 yd^3, strength tests are not required when evidence of satisfactory strength is submitted to and approved by the building official.

5.5.1.4 — A strength test shall be the average of the strengths of two cylinders made from the same sample of concrete and tested at 28 days or at test age designated for determination of f_c'.

5.5.2 — Laboratory-cured specimens

5.5.2.1 — Samples for strength tests shall be taken in accordance with "Standard Practice for Sampling Freshly Mixed Concrete" (ASTM C 172).

5.5.2.2 — Cylinders for strength tests shall be molded and laboratory-cured in accordance with "Standard Practice for Making and Curing Concrete Test Specimens in the Field" (ASTM C 31) and tested in accordance with "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens" (ASTM C 39).

5.5.2.3 — Strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met:

(a) Every arithmetic average of any three consecutive strength tests equals or exceeds f_c'.

(b) No individual strength test (average of two cylinders) falls below f_c' by more than 500 psi when f_c' is 5000 psi or less; or by more than $0.10f_c'$ when f_c' is more than 5000 psi.

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R5.5.1.2 — Samples for strength tests must be taken on a strictly random basis if they are to properly measure the acceptability of the concrete. To be representative, the choice of times of sampling, or the batches of concrete to be sampled, must be made on the basis of chance alone, within the period of placement. Batches should not be sampled on the basis of appearance, convenience, or other possibly biased criteria, because the statistical analyses will lose their validity. Not more than one test (average of two cylinders made from a sample, 5.5.1.4) should be taken from a single batch, and water may not be added to the concrete after the sample is taken.

ASTM D 3665[5.3] describes procedures for random selection of the batches to be tested.

R5.5.2 — Laboratory-cured specimens

R5.5.2.3 — A single set of criteria is given for acceptability of strength and is applicable to all concrete used in structures designed in accordance with the code, regardless of design method used. The concrete strength is considered to be satisfactory as long as averages of any three consecutive strength tests remain above the specified f_c' and no individual strength test falls below the specified f_c' by more than 500 psi if f_c' is 5000 psi or less, or falls below f_c' by more than 10 percent if f_c' is over 5000 psi. Evaluation and acceptance of the concrete can be judged immediately as test results are received during the course of the work. Strength tests failing to meet these criteria will occur occasionally (probably about once in 100 tests) even though concrete strength and uniformity are satisfactory. Allowance should be made for such statistically expected variations in deciding whether the strength level being produced is adequate. In terms of the probability of failure, the criterion of minimum individual strength test result of 500 psi less than f_c' adapts itself readily to small numbers of tests. For

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example, if only five strength tests are made on a small job, it is apparent that, if any of the strength test results (average of two cylinders) is more than 500 psi below f'_c, the criterion is not met.

5.5.2.4 — If either of the requirements of 5.5.2.3 are not met, steps shall be taken to increase the average of subsequent strength test results. Requirements of 5.5.4 shall be observed if requirement of 5.5.2.3(b) is not met.

R5.5.2.4 — When concrete fails to meet either of the strength requirements of 5.5.2.3, steps must be taken to increase the average of the concrete test results. If sufficient concrete has been produced to accumulate at least 15 tests, these should be used to establish a new target average strength as described in 5.3.

If fewer than 15 tests have been made on the class of concrete in question, the new target level should be at least as great as the average level used in the initial selection of proportions. If the average of the available tests made on the project equals or exceeds the level used in the initial selection of proportions, a further increase in average level is required.

The steps taken to increase the average level of test results will depend on the particular circumstances, but could include one or more of the following:

(a) An increase in cementitious materials content;

(b) Changes in mixture proportions;

(c) Reductions in or better control of levels of slump supplied;

(d) A reduction in delivery time;

(e) Closer control of air content; or

(f) An improvement in the quality of the testing, including strict compliance with standard test procedures.

Such changes in operating and testing procedures, or changes in cementitious materials content, or slump should not require a formal resubmission under the procedures of 5.3; however, important changes in sources of cement, aggregates, or admixtures, should be accompanied by evidence that the average strength level will be improved.

Laboratories testing cylinders or cores to determine compliance with these requirements should be accredited for conformance to the requirement of ASTM C 1077[5.4] by a recognized agency such as the American Association for Laboratory Accreditation (A2LA), AASHTO Materials Reference Laboratory (AMRL), National Voluntary Laboratory Accreditation Program (NVLAP), Cement and Concrete Reference Laboratory (CCRL), or their equivalent.

5.5.3 — Field-cured specimens

5.5.3.1 — If required by the building official, results of strength tests of cylinders cured under field conditions shall be provided.

R5.5.3 — Field-cured specimens

R5.5.3.1 — Strength tests of cylinders cured under field conditions may be required to check the adequacy of curing and protection of concrete in the structure.

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5.5.3.2 — Field-cured cylinders shall be cured under field conditions in accordance with "Practice for Making and Curing Concrete Test Specimens in the Field" (ASTM C 31).

5.5.3.3 — Field-cured test cylinders shall be molded at the same time and from the same samples as laboratory-cured test cylinders.

5.5.3.4 — Procedures for protecting and curing concrete shall be improved when strength of field-cured cylinders at test age designated for determination of f_c' is less than 85 percent of that of companion laboratory-cured cylinders. The 85 percent limitation shall not apply if field-cured strength exceeds f_c' by more than 500 psi.

5.5.4 — Investigation of low-strength test results

5.5.4.1 — If any strength test (see 5.5.1.4) of laboratory-cured cylinders falls below specified value of f_c' by more than the values given in 5.5.2.3(b) or if tests of field-cured cylinders indicate deficiencies in protection and curing (see 5.5.3.4), steps shall be taken to assure that load-carrying capacity of the structure is not jeopardized.

5.5.4.2 — If the likelihood of low-strength concrete is confirmed and calculations indicate that load-carrying capacity is significantly reduced, tests of cores drilled from the area in question in accordance with "Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete" (ASTM C 42) shall be permitted. In such cases, three cores shall be taken for each strength test that falls below the values given in 5.5.2.3(b).

5.5.4.3 — Cores shall be prepared for transport and storage by wiping drilling water from their surfaces and placing the cores in watertight bags or containers immediately after drilling. Cores shall be tested no earlier than 48 hours and not later than 7 days after coring unless approved by the registered design professional.

5.5.4.4 — Concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to at least 85 percent of f_c' and if no single core is less than 75 percent of f_c'. Additional testing of cores extracted from locations represented by erratic core strength results shall be permitted.

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R5.5.3.4 — Positive guidance is provided in the code concerning the interpretation of tests of field-cured cylinders. Research has shown that cylinders protected and cured to simulate good field practice should test not less than about 85 percent of standard laboratory moist-cured cylinders. This percentage has been set merely as a rational basis for judging the adequacy of field curing. The comparison is made between the actual measured strengths of companion job-cured and laboratory-cured cylinders, not between job-cured cylinders and the specified value of f_c'. Results for the job-cured cylinders, however, are considered satisfactory if the job-cured cylinders exceed the specified f_c' by more than 500 psi, even though they fail to reach 85 percent of the strength of companion laboratory-cured cylinders.

R5.5.4 — Investigation of low-strength test results

Instructions are provided concerning the procedure to be followed when strength tests have failed to meet the specified acceptance criteria. For obvious reasons, these instructions cannot be dogmatic. The building official must apply judgment as to the true significance of low test results and whether they indicate need for concern. If further investigation is deemed necessary, such investigation may include nondestructive tests, or in extreme cases, strength tests of cores taken from the structure.

Nondestructive tests of the concrete in place, such as by probe penetration, impact hammer, ultrasonic pulse velocity or pull-out may be useful in determining whether or not a portion of the structure actually contains low-strength concrete. Such tests are of value primarily for comparisons within the same job rather than as quantitative measures of strength. For cores, if required, conservatively safe acceptance criteria are provided that should assure structural adequacy for virtually any type of construction.[5.5-5.8] Lower strength may, of course, be tolerated under many circumstances, but this again becomes a matter of judgment on the part of the building official and design engineer. When the core tests fail to provide assurance of structural adequacy, it may be practical, particularly in the case of floor or roof systems, for the building official to require a load test (Chapter 20). Short of load tests, if time and conditions permit, an effort may be made to improve the strength of the concrete in place by supplemental wet curing. Effectiveness of such a treatment must be verified by further strength evaluation using procedures previously discussed.

A core obtained through the use of a water-cooled bit results in a moisture gradient between the exterior and interior of the core

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5.5.4.5 — If criteria of 5.5.4.4 are not met and if the structural adequacy remains in doubt, the responsible authority shall be permitted to order a strength evaluation in accordance with Chapter 20 for the questionable portion of the structure, or take other appropriate action.

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being created during drilling. This adversely affects the core's compressive strength.[5.9] The restriction on the commencement of core testing provides a minimum time for the moisture gradient to dissipate.

Core tests having an average of 85 percent of the specified strength are entirely realistic. To expect core tests to be equal to f_c' is not realistic, because differences in the size of specimens, conditions of obtaining samples, and procedures for curing, do not permit equal values to be obtained.

The code, as stated, concerns itself with assuring structural safety, and the instructions in 5.5 are aimed at that objective. It is not the function of the code to assign responsibility for strength deficiencies, whether or not they are such as to require corrective measures.

Under the requirements of this section, cores taken to confirm structural adequacy will usually be taken at ages later than those specified for determination of f_c'.

5.6 — Preparation of equipment and place of deposit

5.6.1 — Preparation before concrete placement shall include the following:

(a) All equipment for mixing and transporting concrete shall be clean;

(b) All debris and ice shall be removed from spaces to be occupied by concrete;

(c) Forms shall be properly coated;

(d) Masonry filler units that will be in contact with concrete shall be well drenched;

(e) Reinforcement shall be thoroughly clean of ice or other deleterious coatings;

(f) Water shall be removed from place of deposit before concrete is placed unless a tremie is to be used or unless otherwise permitted by the building official;

(g) All laitance and other unsound material shall be removed before additional concrete is placed against hardened concrete.

R5.6 — Preparation of equipment and place of deposit

Recommendations for mixing, handling and transporting, and placing concrete are given in detail in **"Guide for Measuring, Mixing, Transporting, and Placing Concrete"** reported by ACI Committee 304.[5.10] (Presents methods and procedures for control, handling, and storage of materials, measurement, batching tolerances, mixing, methods of placing, transporting, and forms.)

Attention is directed to the need for using clean equipment and for cleaning forms and reinforcement thoroughly before beginning to deposit concrete. In particular, sawdust, nails, wood pieces, and other debris that may collect inside the forms must be removed. Reinforcement must be thoroughly cleaned of ice, dirt, loose rust, mill scale, or other coatings. Water should be removed from the forms.

5.7 — Mixing

5.7.1 — All concrete shall be mixed until there is a uniform distribution of materials and shall be discharged completely before mixer is recharged.

5.7.2 — Ready-mixed concrete shall be mixed and delivered in accordance with requirements of "Specification for Ready-Mixed Concrete" (ASTM C 94) or "Specification for Concrete Made by Volumetric Batching and Continuous Mixing" (ASTM C 685).

R5.7 — Mixing

Concrete of uniform and satisfactory quality requires the materials to be thoroughly mixed until uniform in appearance and all ingredients are distributed. Samples taken from different portions of a batch should have essentially the same unit weight, air content, slump, and coarse aggregate content. Test methods for uniformity of mixing are given in ASTM C 94. The necessary time of mixing will depend on many factors including batch size, stiffness of the batch,

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5.7.3 — Job-mixed concrete shall be mixed in accordance with the following:

(a) Mixing shall be done in a batch mixer of approved type;

(b) Mixer shall be rotated at a speed recommended by the manufacturer;

(c) Mixing shall be continued for at least 1-1/2 minutes after all materials are in the drum, unless a shorter time is shown to be satisfactory by the mixing uniformity tests of "Specification for Ready-Mixed Concrete" (ASTM C 94);

(d) Materials handling, batching, and mixing shall conform to applicable provisions of "Specification for Ready-Mixed Concrete" (ASTM C 94);

(e) A detailed record shall be kept to identify:

(1) number of batches produced;

(2) proportions of materials used;

(3) approximate location of final deposit in structure;

(4) time and date of mixing and placing.

5.8 — Conveying

5.8.1 — Concrete shall be conveyed from mixer to place of final deposit by methods that will prevent separation or loss of materials.

5.8.2 — Conveying equipment shall be capable of providing a supply of concrete at site of placement without separation of ingredients and without interruptions sufficient to permit loss of plasticity between successive increments.

5.9 — Depositing

5.9.1 — Concrete shall be deposited as nearly as practical in its final position to avoid segregation due to rehandling or flowing.

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size and grading of the aggregate, and the efficiency of the mixer. Excessively long mixing times should be avoided to guard against grinding of the aggregates.

R5.8 — Conveying

Each step in the handling and transporting of concrete needs to be carefully controlled to maintain uniformity within a batch and from batch to batch. It is essential to avoid segregation of the coarse aggregate from the mortar or of water from the other ingredients.

The code requires the equipment for handling and transporting concrete to be capable of supplying concrete to the place of deposit continuously and reliably under all conditions and for all methods of placement. The provisions of 5.8 apply to all placement methods, including pumps, belt conveyors, pneumatic systems, wheelbarrows, buggies, crane buckets, and tremies.

Serious loss in strength can result when concrete is pumped through pipe made of aluminum or aluminum alloy.[5.11] Hydrogen gas generated by the reaction between the cement alkalies and the aluminum eroded from the interior of the pipe surface has been shown to cause strength reduction as much as 50 percent. Hence, equipment made of aluminum or aluminum alloys should not be used for pump lines, tremies, or chutes other than short chutes such as those used to convey concrete from a truck mixer.

R5.9 — Depositing

Rehandling concrete can cause segregation of the materials. Hence, the code cautions against this practice. Retempering of partially set concrete with the addition of water should not be permitted, unless authorized. This does not preclude

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5.9.2 — Concreting shall be carried on at such a rate that concrete is at all times plastic and flows readily into spaces between reinforcement.

5.9.3 — Concrete that has partially hardened or been contaminated by foreign materials shall not be deposited in the structure.

5.9.4 — Retempered concrete or concrete that has been remixed after initial set shall not be used unless approved by the engineer.

5.9.5 — After concreting is started, it shall be carried on as a continuous operation until placing of a panel or section, as defined by its boundaries or predetermined joints, is completed except as permitted or prohibited by 6.4.

5.9.6 — Top surfaces of vertically formed lifts shall be generally level.

5.9.7 — When construction joints are required, joints shall be made in accordance with 6.4.

5.9.8 — All concrete shall be thoroughly consolidated by suitable means during placement and shall be thoroughly worked around reinforcement and embedded fixtures and into corners of form.

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the practice (recognized in ASTM C 94) of adding water to mixed concrete to bring it up to the specified slump range so long as prescribed limits on the maximum mixing time and water-cementitious materials ratio are not violated.

Section 5.10.4 of the ACI 318-71 code contained a requirement that "where conditions make consolidation difficult or where reinforcement is congested, batches of mortar containing the same proportions of cement, sand, and water as used in the concrete, shall first be deposited in the forms to a depth of at least 1 in." That requirement was deleted from the ACI 318-77 code because the conditions for which it was applicable could not be defined precisely enough to justify its inclusion as a code requirement. The practice, however, has merit, and should be incorporated in job specifications where appropriate, with the specific enforcement the responsibility of the job inspector rather than the building official. The use of mortar batches aids in preventing honeycomb and poor bonding of the concrete with the reinforcement. The mortar should be placed immediately before depositing the concrete, and must be plastic (neither stiff nor fluid) when the concrete is placed.

Recommendations for consolidation of concrete are given in detail in **"Guide for Consolidation of Concrete"** reported by ACI Committee 309.[5.12] (Presents current information on the mechanism of consolidation and gives recommendations on equipment characteristics and procedures for various classes of concrete.)

5.10 — Curing

5.10.1 — Concrete (other than high-early-strength) shall be maintained above 50 °F and in a moist condition for at least the first 7 days after placement, except when cured in accordance with 5.10.3.

5.10.2 — High-early-strength concrete shall be maintained above 50 °F and in a moist condition for at least the first 3 days, except when cured in accordance with 5.10.3.

5.10.3 — Accelerated curing

5.10.3.1 — Curing by high-pressure steam, steam at atmospheric pressure, heat and moisture, or other accepted processes, shall be permitted to accelerate strength gain and reduce time of curing.

5.10.3.2 — Accelerated curing shall provide a compressive strength of the concrete at the load stage considered at least equal to required design strength at that load stage.

5.10.3.3 — Curing process shall be such as to produce concrete with a durability at least equivalent to the curing method of 5.10.1 or 5.10.2.

R5.10 — Curing

Recommendations for curing concrete are given in detail in **"Standard Practice for Curing Concrete"** reported by ACI Committee 308.[5.13] (Presents basic principles of proper curing and describes the various methods, procedures, and materials for curing of concrete.)

R5.10.3 — Accelerated curing

The provisions of this section apply whenever an accelerated curing method is used, whether for precast or cast-in-place elements. The compressive strength of steam-cured concrete is not as high as that of similar concrete continuously cured under moist conditions at moderate temperatures. Also, the elastic modulus E_c of steam-cured specimens may vary from that of specimens moist-cured at normal temperatures. When steam-curing is to be used, it is advisable to base the concrete mixture proportions on steam-cured test cylinders.

Accelerated curing procedures require careful attention to obtain uniform and satisfactory results. It is essential that moisture loss during the curing process be prevented.

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5.10.4 — When required by the engineer or architect, supplementary strength tests in accordance with 5.5.3 shall be performed to assure that curing is satisfactory.

5.11 — Cold weather requirements

5.11.1 — Adequate equipment shall be provided for heating concrete materials and protecting concrete during freezing or near-freezing weather.

5.11.2 — All concrete materials and all reinforcement, forms, fillers, and ground with which concrete is to come in contact shall be free from frost.

5.11.3 — Frozen materials or materials containing ice shall not be used.

5.12 — Hot weather requirements

During hot weather, proper attention shall be given to ingredients, production methods, handling, placing, protection, and curing to prevent excessive concrete temperatures or water evaporation that could impair required strength or serviceability of the member or structure.

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R5.10.4 — In addition to requiring a minimum curing temperature and time for normal- and high-early-strength concrete, the code provides a specific criterion in 5.5.3 for judging the adequacy of field curing. At the test age for which the strength is specified (usually 28 days), field-cured cylinders should produce strength not less than 85 percent of that of the standard, laboratory-cured cylinders. For a reasonably valid comparison to be made, field-cured cylinders and companion laboratory-cured cylinders must come from the same sample. Field-cured cylinders must be cured under conditions identical to those of the structure. If the structure is protected from the elements, the cylinder should be protected similarly.

That is, cylinders related to members not directly exposed to weather should be cured adjacent to those members and provided with the same degree of protection and method of curing.

Obviously, the field cylinders should not be treated more favorably than the elements they represent. (See code and commentary, 5.5.3 for additional information.)

If the field-cured cylinders do not provide satisfactory strength by this comparison, measures should be taken to improve the curing of the structure. If the tests indicate a possible serious deficiency in strength of concrete in the structure, core tests may be required, with or without supplemental wet curing, to check the structural adequacy, as provided in 5.5.4.

R5.11 — Cold weather requirements

Recommendations for cold weather concreting are given in detail in **"Cold Weather Concreting"** reported by ACI Committee 306.[5.14] (Presents requirements and methods for producing satisfactory concrete during cold weather.)

R5.12 — Hot weather requirements

Recommendations for hot weather concreting are given in detail in **"Hot Weather Concreting"** reported by ACI Committee 305.[5.15] (Defines the hot weather factors that affect concrete properties and construction practices and recommends measures to eliminate or minimize the undesirable effects.) ACI 306.1[5.16] is a specification for cold weather concreting.

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Notes

CHAPTER 6 — FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION AND MOVEMENT JOINTS

CODE

6.1 — Design of formwork

6.1.1 — Forms shall result in a final structure that conforms to shapes, lines, and dimensions of the members as required by the design drawings and specifications.

6.1.2 — Forms shall be substantial and sufficiently tight to prevent leakage of mortar.

6.1.3 — Forms shall be properly braced or tied together to maintain position and shape.

6.1.4 — Forms and their supports shall be designed so as not to damage previously placed structure.

6.1.5 — Design of formwork shall include consideration of the following factors:

(a) Rate and method of placing concrete;

(b) Construction loads, including vertical, horizontal, and impact loads;

(c) Special form requirements for construction of shells, folded plates, domes, architectural concrete, or similar types of elements.

6.1.6 — Forms for prestressed concrete members shall be designed and constructed to permit movement of the member without damage during application of prestressing force.

6.1.7 — Form tie assemblies and systems in liquid-containment structures shall be suitable for providing a liquid-tight structure.

6.1.7.1 — Form tie assemblies for liquid-containment structures shall leave no metal or other material except concrete within 1-1/2 in. of the formed surface.

6.1.8 — Form surfaces that will be in contact with concrete shall be coated with an effective bond-breaking form coating.

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R6.1 — Design of formwork

Only minimum performance requirements for formwork, necessary to provide for public health and safety, are prescribed in Chapter 6. Formwork for concrete, including proper design, construction, and removal, demands sound judgment and planning to achieve adequate forms that are both economical and safe. Detailed information on formwork for concrete is given in: **"Guide to Formwork for Concrete,"** reported by Committee 347.[6.1] (Provides recommendations for design, construction, and materials for formwork, forms for special structures, and formwork for special methods of construction. Directed primarily to contractors, the suggested criteria will aid engineers and architects in preparing job specifications for the contractors.)

Formwork for Concrete[6.2] prepared under the direction of ACI Committee 347. (A how-to-do-it handbook for contractors, engineers, and architects following the guidelines established in ACI 347R-88. Planning, building, and using formwork are discussed, including tables, diagrams, and formulas for form design loads.)

R6.1.7 — When portions of single-rod ties are to remain in a liquid-containing structure, the portions that are to remain should be provided with an integral waterstop at midpoint.

The assembly should provide cone-shaped depressions in the forms at the surface, at least 1 in. in diameter and 1-1/2 in. deep, to allow filling and patching.

Through ties that are to be entirely removed from the structure should be tapered over the portion that passes through the concrete. The large end of tapered ties should be on the liquid side of the wall. The contractor should be required to demonstrate the methods and materials to be used to fill the void thus formed.

R6.1.8 — Bond-breaking form coatings should be used in accordance with 4.4 of ACI 347. Refer to R4.7.1. of this commentary for compatibility of form-release agents and coatings.

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6.2 — Removal of forms, shores, and reshoring

6.2.1 — Removal of forms

Forms shall be removed in such a manner as not to impair safety and serviceability of the structure. Concrete to be exposed by form removal shall have sufficient strength not to be damaged by removal operation.

6.2.2 — Removal of shores and reshoring

The provisions of 6.2.2.1 through 6.2.2.3 shall apply to slabs and beams except where cast on the ground.

6.2.2.1 — Before starting construction, the contractor shall develop a procedure and schedule for removal of shores and installation of reshores and for calculating the loads transferred to the structure during the process.

(a) The structural analysis and concrete strength data used in planning and implementing form removal and shoring shall be furnished by the contractor to the building official when so requested;

(b) No construction loads shall be supported on, nor any shoring removed from, any part of the structure under construction except when that portion of the structure in combination with remaining forming and shoring system has sufficient strength to support safely its weight and loads placed thereon;

(c) Sufficient strength shall be demonstrated by structural analysis considering proposed loads, strength of forming and shoring system, and concrete strength data. Concrete strength data shall be based on tests of field-cured cylinders or, when approved by the building official, on other procedures to evaluate concrete strength.

6.2.2.2 — No construction loads exceeding the combination of superimposed dead load plus specified live load shall be supported on any unshored portion of the structure under construction, unless analysis indicates adequate strength to support such additional loads.

6.2.2.3 — Form supports for prestressed concrete members shall not be removed until sufficient prestressing has been applied to enable prestressed members to carry their dead load and anticipated construction loads.

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R6.2 — Removal of forms, shores, and reshoring

In determining the time for removal of forms, consideration should be given to the construction loads and to the possibilities of deflections.[6.3] The construction loads are frequently at least as great as the specified live loads. At early ages, a structure may be adequate to support the applied loads, but may deflect sufficiently to cause permanent damage.

Evaluation of concrete strength during construction may be demonstrated by field-cured test cylinders or other procedures approved by the building official such as:

(a) Tests of cast-in-place cylinders in accordance with "Standard Test Method for Compressive Strength of Concrete Cylinders Cast-in-Place in Cylindrical Molds" (ASTM C 873[6.4]). (This method is limited to use in slabs where the depth of concrete is from 5 to 12 in.);

(b) Penetration resistance in accordance with "Standard Test Method for Penetration Resistance of Hardened Concrete" (ASTM C 803[6.5]);

(c) Pullout strength in accordance with "Standard Test Method for Pullout Strength of Hardened Concrete" (ASTM C 900[6.6]);

(d) Maturity factor measurements and correlation in accordance with ASTM C 1074.[6.7]

Procedures (b), (c), and (d) require sufficient data, using job materials, to demonstrate correlation of measurements on the structure with compressive strength of molded cylinders or drilled cores.

Where the structure is adequately supported on shores, the side forms of beams, girders, columns, walls, and similar vertical forms, may generally be removed after 12 hours of cumulative curing time, provided the side forms support no loads other than the lateral pressure of the plastic concrete. "Cumulative curing time" represents the sum of time intervals, not necessarily consecutive, during which the temperature of the air surrounding the concrete is above 50 °F. The 12-hour cumulative curing time is based on regular cements and ordinary conditions; the use of special cements or unusual conditions may require adjustment of the given limits. For example, concrete made with Type II or V (ASTM C 150) or ASTM C 595 cements, concrete containing retarding admixtures, and concrete to which ice was added during mixing (to lower the temperature of fresh concrete) may not have sufficient strength in 12 hours and should be investigated before removal of formwork.

The removal of formwork for multistory construction should be a part of a planned procedure considering the temporary support of the whole structure as well as that of each individual member. Such a procedure should be worked out before construction and should be based on a structural analysis taking into account the following items, as a minimum:

(a) The structural system that exists at the various stages of construction and the construction loads corresponding to those stages;

(b) The strength of the concrete at the various ages during construction;

(c) The influence of deformations of the structure and shoring system on the distribution of dead loads and construction loads during the various stages of construction;

(d) The strength and spacing of shores or shoring systems used, as well as the method of shoring, bracing, shore removal, and reshoring including the minimum time intervals between the various operations;

(e) Any other loading or condition that affects the safety or serviceability of the structure during construction.

For multistory construction, the strength of the concrete during the various stages of construction should be substantiated by field-cured test specimens or other approved methods.

6.3 — Conduits and pipes embedded in concrete

6.3.1 — Conduits, pipes, and sleeves of any material not harmful to concrete and within limitations of 6.3 shall be permitted to be embedded in concrete with approval of the engineer, provided they are not considered to replace structurally the displaced concrete, except as provided in 6.3.6.

6.3.2 — Conduits and pipes of aluminum shall not be embedded in structural concrete.

6.3.3 — Conduits, pipes, and sleeves passing through a slab, wall, or beam shall not impair significantly the strength of the construction.

R6.3 — Conduits and pipes embedded in concrete

R6.3.1 — Conduits, pipes, and sleeves not harmful to concrete can be embedded within the concrete, but the work should be done in such a manner that the structure will not be endangered. Empirical rules are given in 6.3 for safe installations under common conditions; for other than common conditions, special designs should be made. Many general building codes have adopted ANSI/ASME piping codes B 31.1 for power piping[6.8] and B 31.3 for chemical and petroleum piping.[6.9] The specifier should be sure that the appropriate piping codes are used in the design and testing of the system. The contractor should not be permitted to install conduits, pipes, ducts, or sleeves that are not shown on the plans or not approved by the engineer or architect.

For the integrity of the structure, it is important that all conduit and pipe fittings within the concrete be carefully assembled as shown on the plans or called for in the job specifications.

R6.3.2 — The code prohibits the use of aluminum in structural concrete. Aluminum reacts with concrete and, in the presence of chloride ions, can also react electrolytically with steel, causing cracking and/or spalling of the concrete. Aluminum electrical conduits present a special problem because stray electrical current accelerates the adverse reaction.

6.3.4 — Conduits and pipes, with their fittings, embedded within a column shall not displace more than 4 percent of the area of cross section on which strength is calculated or which is required for fire protection.

6.3.5 — Except when drawings for conduits and pipes are approved by the structural engineer, conduits and pipes embedded within a slab, wall, or beam (other than those merely passing through) shall satisfy the following:

6.3.5.1 — They shall not be larger in outside dimension than 1/3 the overall thickness of slab, wall, or beam in which they are embedded.

6.3.5.2 — They shall not be spaced closer than three diameters or widths on center.

6.3.5.3 — They shall not impair significantly the strength of the construction.

6.3.6 — Conduits, pipes, and sleeves shall be permitted to be considered as replacing structurally in compression the displaced concrete provided in 6.3.6.1 through 6.3.6.3:

6.3.6.1 — They are not exposed to rusting or other deterioration.

6.3.6.2 — They are of uncoated or galvanized iron or steel not thinner than standard Schedule 40 steel pipe.

6.3.6.3 — They have a nominal inside diameter not over 2 in. and are spaced not less than three diameters on centers.

6.3.7 — Pipes and fittings shall be designed to resist effects of the material, pressure, and temperature to which they will be subjected.

6.3.8 — No liquid, gas, or vapor, except water not exceeding 90 °F nor 50 psi pressure, shall be placed in the pipes until the concrete has attained its design strength.

6.3.9 — In solid slabs, piping, unless it is for radiant heating or snow melting, shall be placed between top and bottom reinforcement.

6.3.10 — Concrete cover for pipes, conduits, and fittings shall not be less than 2 in. for concrete exposed to earth, contained liquids, or weather, nor 3/4 in. for concrete not exposed to contained liquids, weather, or in contact with ground.

6.3.11 — Reinforcement with an area not less than 0.002 times area of concrete section nor less than the area required by 7.12 of this code shall be provided normal to piping.

COMMENTARY

R6.3.6 — There have been some reported cases of extensive deterioration of concrete surrounding embedded galvanized conduits and pipes exposed to corrosive liquids or gases. The loss of the self-sacrificing galvanized coating results in the release of gases, creating a void around the conduit or pipe, thus allowing the migration of moisture or gases to the bare steel and initiating the corrosion process. In the corrosion process of zinc, there is the release of hydrogen gas, which may also cause "Hydrogen Embrittlement" of certain steels.[6.10,6.11]

R6.3.7 — The ACI 318-83 code limited the maximum pressure in embedded pipe to 200 psi, which was considered too restrictive. Nevertheless, the effects of such pressures and the expansion of embedded pipe should be considered in the design of the concrete member.

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6.3.12 — Piping and conduit shall be so fabricated and installed that cutting, bending, or displacement of reinforcement from its proper location will not be required.

6.3.13 — Pipes passing through walls of a liquid-containing structure shall include an integral waterstop.

6.4 — Construction joints

6.4.1 — Surface of concrete construction joints shall be cleaned and laitance removed.

6.4.2 — Immediately before new concrete is placed, all construction joints shall be wetted and standing water removed.

6.4.3 — Construction joints shall be so made and located as not to impair the strength of the structure. Provision shall be made for transfer of shear and other forces through construction joints. See 11.7.9.

6.4.4 — Construction joints in floors shall be located within the middle third of spans of slabs, beams, and girders. Joints in girders shall be offset a minimum distance of two times the width of intersecting beams.

6.4.5 — Beams, girders, or slabs supported by columns or walls shall not be cast or erected until concrete in the vertical support members is no longer plastic.

6.4.6 — Beams, girders, haunches, drop panels, and capitals shall be placed monolithically as part of a slab system, unless otherwise shown in contract documents.

6.4.7 — Where intended to be watertight, construction joints in conventionally reinforced liquid-containing structures shall have an integral waterstop.

COMMENTARY

R6.4 — Construction joints

For the integrity of the structure, it is important that all construction joints be carefully defined in construction documents and constructed as required. Any deviations therefrom should be approved by the engineer or architect.

R6.4.1 — The surfaces of concrete at all construction joints should be prepared as called for in ACI 301.

R6.4.2 — The requirements of the ACI 318-77 code for the use of neat cement on vertical joints have been removed, because it is rarely practical and can be detrimental where deep forms and steel congestion prevent proper access. Often wet blasting and other procedures are more appropriate. Because the code sets only minimum standards, the engineer may have to specify special procedures if conditions warrant. The degree to which mortar batches are needed at the start of concrete placement depend on concrete proportions, congestion of steel, vibrator access, and other factors.

R6.4.3 — Construction joints should be located where they will cause the least weakness in the structure. When shear due to gravity load is not significant, as is usually the case in the middle of the span of flexural members, a simple vertical joint may be adequate. Lateral force design may require special design treatment of construction joints. Shear keys, intermittent shear keys, diagonal dowels, or the shear transfer method of 11.7 may be used whenever a force transfer is required.

R6.4.5 — Delay in placing concrete in members supported by columns and walls is necessary to prevent cracking at the interface of the slab and supporting member, caused by bleeding and settlement of plastic concrete in the supporting member.

R6.4.6 — Separate placement of slabs and beams, haunches, and similar elements is permitted when shown on the drawings and where provision has been made to transfer forces as required in 6.4.3.

R6.4.7 — Waterstops may be rubber, plastic, or metal. Information on the design and detailing of construction joints is included in ACI 350.4R.

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6.4.8 — For environmental engineering concrete structures, the elapsed time between casting adjoining units shall be at least 48 hours.

6.5 — Movement joints

6.5.1 — The design shall consider and provide for volume changes in a manner that will minimize damage to the structure.

6.5.2 — Expansion joints, when used, shall include a compressible preformed joint filler, a joint sealant, and where intended to be watertight, a waterstop.

6.5.3 — Contraction joints, when used, shall be permitted to be partial or full, depending on the reinforcing detail, and shall include a groove or recess at the surface for the placement of joint sealant.

COMMENTARY

R6.4.8 — If the elapsed time between adjoining placements is very short, there will not be adequate dissipation of shrinkage and heat of hydration effects in the first placement.

R6.5 — Movement joints

This chapter addresses requirements for movement joints (expansion and contraction joints) in addition to requirements for construction joints.

R6.5.1 — Volume changes in concrete are generally caused by expansion or contraction in response to change in temperature, moisture content, or both. Movement joints (expansion joints and contraction joints) may be used to accommodate the resultant stresses and strains, particularly at changes in mass or configuration. Information on the design and detailing of movement joints is included in ACI 350.4R and 224.2R.

R6.5.3 — Full contraction joints have all reinforcement stopped at the joint. Partial contraction joints have a portion of the normal reinforcement passing through the joint up to a maximum of 50 percent.

CHAPTER 7 — DETAILS OF REINFORCEMENT

CODE

7.0 — Notation

d = distance from extreme compression fiber to centroid of tension reinforcement, in.
d_b = nominal diameter of bar, wire, or prestressing strand, in.
f'_{ci} = compressive strength of concrete at time of initial prestress, psi
f_y = specified yield strength of nonprestressed reinforcement, psi
ℓ_d = development length, in. See Chapter 12

COMMENTARY

Recommended methods and standards for preparing design drawings, typical details, and drawings for the fabrication and placing of reinforcing steel in reinforced concrete structures are given in ***ACI Detailing Manual***, reported by ACI Committee 315.[7.1]

All provisions in the code relating to bar, wire, or strand diameter (and area) are based on the nominal dimensions of the reinforcement as given in the appropriate ASTM specification. Nominal dimensions are equivalent to those of a circular area having the same weight per foot as the ASTM designated bar, wire, or strand sizes. Cross-sectional area of reinforcement is based on nominal dimensions.

7.1 — Standard hooks

The term standard hook as used in this code shall mean one of the following:

7.1.1 — 180-deg bend plus $4d_b$ extension, but not less than 2-1/2 in. at free end of bar.

7.1.2 — 90-deg bend plus $12d_b$ extension at free end of bar.

7.1.3 — For stirrup and tie hooks*

(a) No. 5 bar and smaller, 90-deg bend plus $6d_b$ extension at free end of bar; or

(b) No. 6, No. 7, and No. 8 bar, 90-deg bend plus $12d_b$ extension at free end of bar; or

(c) No. 8 bar and smaller, 135-deg bend plus $6d_b$ extension at free end of bar.

7.1.4 — Seismic hooks as defined in 21.1.

R7.1 — Standard hooks

R7.1.3 — Standard stirrup and tie hooks are limited to No. 8 bars and smaller, and the 90-deg hook with $6d_b$ extension is further limited to No. 5 bars and smaller, in both cases as the result of research showing that larger bar sizes with 90-deg hooks and $6d_b$ extensions tend to "pop out" under high load.

7.2 — Minimum bend diameters

7.2.1 — Diameter of bend measured on the inside of the bar, other than for stirrups and ties in sizes No. 3 through No. 5, shall not be less than the values in Table 7.2.

7.2.2 — Inside diameter of bend for stirrups and ties shall not be less than $4d_b$ for No. 5 bar and smaller. For bars larger than No. 5, diameter of bend shall be in accordance with Table 7.2.

*For closed ties and continuously wound ties defined as hoops in Chapter 21, a 135-deg bend plus an extension of at least $6d_b$, but not less than 3 in. (See definition of "hoop" in 21.1.)

R7.2 — Minimum bend diameters

Standard bends in reinforcing bars are described in terms of the inside diameter of bend because this is easier to measure than the radius of bend. The primary factors affecting the minimum bend diameter are feasibility of bending without breakage and avoidance of crushing the concrete inside the bend.

R7.2.2 — The minimum $4d_b$ bend for the bar sizes commonly used for stirrups and ties is based on accepted industry practice in the United States. Use of a stirrup bar size not greater than No. 5 for either the 90-deg or 135-deg standard stirrup hook will permit multiple bending on standard stirrup bending equipment.

CODE

TABLE 7.2—MINIMUM DIAMETERS OF BEND

Bar size	Minimum diameter
No. 3 through No. 8	$6d_b$
No. 9, No. 10, and No. 11	$8d_b$
No. 14 and No. 18	$10d_b$

7.2.3 — Inside diameter of bend in welded wire fabric (plain or deformed) for stirrups and ties shall not be less than $4d_b$ for deformed wire larger than D6 and $2d_b$ for all other wires. Bends with inside diameter of less than $8d_b$ shall not be less than $4d_b$ from nearest welded intersection.

7.3 — Bending

7.3.1 — All reinforcement shall be bent cold, unless otherwise permitted by the engineer.

7.3.2 — Reinforcement partially embedded in concrete shall not be field bent, except as shown on the design drawings or permitted by the engineer.

7.4 — Surface conditions of reinforcement

7.4.1 — At the time concrete is placed, reinforcement shall be free from mud, oil, or other nonmetallic coatings that decrease bond. Epoxy coating of steel reinforcement in accordance with standards referenced in 3.5.3.7 and 3.5.3.8 shall be permitted.

COMMENTARY

R7.2.3 — Welded wire fabric, of plain or deformed wire, can be used for stirrups and ties. The wire at welded intersections does not have the same uniform ductility and bendability as in areas that were not heated. These effects of the welding temperature are usually dissipated in a distance of approximately four wire diameters. Minimum bend diameters permitted are in most cases the same as those required in the ASTM bend tests for wire material (ASTM A 82 and A 496).

R7.3 — Bending

R7.3.1 — The engineer may be the design engineer or architect or the engineer or architect employed by the owner to perform inspection. For unusual bends with inside diameters less than ASTM bend test requirements, special fabrication may be required.

R7.3.2 — Construction conditions may make it necessary to bend bars that have been embedded in concrete. Such field bending should not be done without authorization of the engineer. The engineer should determine whether the bars should be bent cold or if heating should be used. Bends should be gradual and must be straightened as required.

Tests[7.2,7.3] have shown that A 615 Grade 40 and Grade 60 reinforcing bars can be cold bent and straightened up to 90 deg at or near the minimum diameter specified in 7.2. If cracking or breakage is encountered, heating to a maximum temperature of 1500 °F may avoid this condition for the remainder of the bars. Bars that fracture during bending or straightening can be spliced outside the bend region.

Heating should be performed in a manner that will avoid damage to the concrete. If the bend area is within approximately 6 in. of the concrete, some protective insulation may need to be applied. Heating of the bar should be controlled by temperature-indicating crayons or other suitable means. The heated bars should not be artificially cooled (with water or forced air) until after cooling to at least 600 °F.

R7.4 — Surface conditions of reinforcement

Specific limits on rust are based on tests,[7.4] plus a review of earlier tests and recommendations. Reference 7.4 provides guidance with regard to the effects of rust and mill scale on bond characteristics of deformed reinforcing bars. Research has shown that a normal amount of rust increases bond. Normal rough handling generally removes rust that is loose enough to injure the bond between the concrete and reinforcement.

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7.4.2 — Except for prestressing steel, steel reinforcement with rust, mill scale, or a combination of both shall be considered satisfactory, provided the minimum dimensions (including height of deformations) and weight of a hand-wire-brushed test specimen comply with applicable ASTM specifications referenced in 3.5.

7.4.3 — Prestressing steel shall be clean and free of oil, dirt, scale, pitting, and excessive rust. A light coating of rust shall be permitted.

7.5 — Placing reinforcement

7.5.1 — Reinforcement, including tendons, and post-tensioning ducts shall be accurately placed and adequately supported before concrete is placed, and shall be secured against displacement within tolerances permitted in 7.5.2.

7.5.2 — Unless otherwise specified by the licensed design professional, reinforcement, including tendons, and post-tensioning ducts shall be placed within the tolerances in 7.5.2.1 and 7.5.2.2.

7.5.2.1 — Tolerance for depth ***d***, and minimum concrete cover in flexural members, walls, and compression members shall be as follows:

	Tolerance on ***d***	Tolerance on minimum concrete cover
$d \leq 8$ in.	±3/8 in.	–3/8 in.
$d > 8$ in.	±1/2 in.	–1/2 in.

except that tolerance for the clear distance to formed soffits shall be minus 1/4 in. and tolerance for cover

COMMENTARY

R7.4.3 — Guidance for evaluating the degree of rusting on strand is given in Reference 7.5.

R7.5 — Placing reinforcement

R7.5.1 — Reinforcement including tendons and post-tensioning ducts should be adequately supported in the forms to prevent displacement by concrete placement or workers. Beam stirrups should be supported on the bottom form of the beam by positive supports such as continuous longitudinal beam bolsters. If only the longitudinal beam bottom reinforcement is supported, construction traffic can dislodge the stirrups as well as any prestressing tendons tied to the stirrups.

R7.5.2 — Generally accepted practice, as reflected in "**Standard Specifications for Tolerances for Concrete Construction and Materials**," reported by ACI Committee 117[7.6] has established tolerances on total depth (formwork or finish) and fabrication of truss bent reinforcing bars and closed ties, stirrups, and spirals. The engineer should specify more restrictive tolerances than those permitted by the code when necessary to minimize the accumulation of tolerances resulting in excessive reduction in effective depth or cover.

More restrictive tolerances have been placed on minimum clear distance to formed soffits because of its importance for durability and fire protection, and because bars are usually supported in such a manner that the specified tolerance is practical.

More restrictive tolerances than those required by the code may be desirable for prestressed concrete to achieve camber control within limits acceptable to the designer or owner. In such cases, the engineer should specify the necessary tolerances. Recommendations are given in Reference 7.7.

R7.5.2.1 — The code specifies a tolerance on depth ***d***, an essential component of strength of the member. Because reinforcing steel is placed with respect to edges of members and formwork surfaces, the depth ***d*** is not always conveniently measured in the field. Engineers should specify tolerances for bar placement, cover, and member size. See ACI 117.[7.6]

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shall not exceed minus 1/3 the minimum concrete cover required in the contract documents.

7.5.2.2 — Tolerance for longitudinal location of bends and ends of reinforcement shall be ±2 in., except the tolerance shall be at ±1/2 in. at the discontinuous ends of brackets and corbels, and ±1 in. at the discontinuous ends of other members. The tolerance for minimum concrete cover of 7.5.2.1 shall also apply at discontinuous ends of members.

7.5.3 — Welded wire fabric (with wire size not greater than W5 or D5) used in slabs not exceeding 10 ft in span shall be permitted to be curved from a point near the top of slab over the support to a point near the bottom of slab at midspan, provided such reinforcement is either continuous over, or securely anchored at support.

7.5.4 — Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the engineer.

7.6 — Spacing limits for reinforcement

7.6.1 — The minimum clear spacing between parallel bars in a layer shall be d_b, but not less than 1 in. See also 3.3.2.

7.6.2 — Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above bars in the bottom layer with clear distance between layers not less than 1 in.

7.6.3 — In spirally reinforced or tied reinforced compression members, clear distance between longitudinal bars shall be not less than $1.5d_b$ nor less than 1-1/2 in. See also 3.3.2.

7.6.4 — Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.

7.6.5 — In walls and slabs other than concrete joist construction, primary flexural reinforcement shall not be spaced farther apart than two times the wall or slab thickness, nor farther apart than 12 in.

7.6.6 — Bundled bars

7.6.6.1 — Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four in any one bundle.

7.6.6.2 — Bundled bars shall be enclosed within stirrups or ties.

COMMENTARY

R7.5.4 — "Tack" welding (welding crossing bars) can seriously weaken a bar at the point welded by creating a metallurgical notch effect. This operation can be performed safely only when the material welded and welding operations are under continuous competent control, as in the manufacture of welded wire fabric.

R7.6 — Spacing limits for reinforcement

Although the minimum bar spacings are unchanged in this code, the development lengths given in Chapter 12 became a function of the bar spacings since the ACI 318-89 code. As a result, it may be desirable to use larger than minimum bar spacings in some cases. The minimum limits were originally established to permit concrete to flow readily into spaces between bars and between bars and forms without honeycomb, and to ensure against concentration of bars on a line that may cause shear or shrinkage cracking. Use of "nominal" bar diameter to define minimum spacing permits a uniform criterion for all bar sizes.

R7.6.5 — A limitation has been placed on the maximum spacing of reinforcement for crack control in liquid-retaining structures.

R7.6.6 — Bundled bars

Bond research[7.8] showed that bar cutoffs within bundles should be staggered. Bundled bars should be tied, wired, or otherwise fastened together to ensure remaining in position whether vertical or horizontal.

A limitation that bars larger than No. 11 not be bundled in beams or girders is a practical limit for application to

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7.6.6.3 — Bars larger than No. 11 shall not be bundled in beams.

7.6.6.4 — Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least $40d_b$ stagger.

7.6.6.5 — Where spacing limitations and minimum concrete cover are based on bar diameter d_b, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

7.6.7 — Tendons and ducts

7.6.7.1 — Center-to-center spacing of pretensioning tendons at each end of a member shall be not less than $4d_b$ for strands, or $5d_b$ for wire, except that if concrete strength at transfer of prestress, f'_{ci} is 4000 psi or more, minimum center-to-center spacing of strands shall be 1-3/4 in. for strands of 1/2 in. nominal diameter or smaller and 2 in. for strands of 0.6 in. nominal diameter. See also 3.3.2. Closer vertical spacing and bundling of tendons shall be permitted in the middle portion of a span.

7.6.7.2 — Bundling of post-tensioning ducts shall be permitted if shown that concrete can be satisfactorily placed and if provision is made to prevent the prestressing steel, when tensioned, from breaking through the duct.

7.7 — Concrete protection for reinforcement

7.7.1 — Cast-in-place concrete (nonprestressed)

The following minimum concrete cover shall be provided for reinforcement, but shall not be less than required by 7.7.6:

	Minimum cover, in.
(a) Concrete cast against and permanently exposed to earth	3
(b) Concrete exposed to earth, liquid, weather, or bearing on work mat or slabs supporting earth cover:	
Slabs and joists	2
Beams and columns:	
Stirrups, spirals, and ties	2

COMMENTARY

building size members. (The "**Standard Specifications for Highway Bridges**"[7.9] permits two-bar bundles for No. 14 and No. 18 bars in bridge girders.) Conformance to the crack control requirements of 10.6 will effectively preclude bundling of bars larger than No. 11 as tensile reinforcement.

The code phrasing "bundled in contact to act as a unit," is intended to preclude bundling more than two bars in the same plane. Typical bundle shapes are triangular, square, or L-shaped patterns for three- or four-bar bundles. As a practical caution, bundles more than one bar deep in the plane of bending should not be hooked or bent as a unit. Where end hooks are required, it is preferable to stagger the individual bar hooks within a bundle.

R7.6.7 — Tendons and ducts

R7.6.7.1 — The allowed decreased spacing in this section for transfer strengths of 4000 psi or greater is based on Reference 7.10 and 7.11.

R7.6.7.2 — When ducts for prestressing steel in a beam are arranged closely together vertically, provision should be made to prevent the prestressing steel from breaking through the duct when tensioned. Horizontal disposition of ducts should allow proper placement of concrete. A clear spacing of 1-1/3 times the size of the coarse aggregate, but not less than 1 in., has proven satisfactory. Where concentration of tendons or ducts tends to create a weakened plane in the concrete cover, reinforcement should be provided to control cracking.

R7.7 — Concrete protection for reinforcement

Concrete cover as protection of reinforcement against weather and other effects is measured from the concrete surface to the outermost surface of the steel to which the cover requirement applies. Where minimum cover is prescribed for a class of structural member, it is measured to the outer edge of stirrups, ties, or spirals if transverse reinforcement encloses main bars; to the outermost layer of bars if more than one layer is used without stirrups or ties; or to the metal end fitting or duct on post-tensioned prestressing steel.

The condition "concrete surfaces exposed to earth or weather" refers to direct exposure to moisture changes and not just to temperature changes. Slab or thin shell soffits are not usually considered directly "exposed" unless subject to alternate wetting and drying, including that due to condensation conditions or direct leakage from exposed top surface, runoff, or similar effects. Soffits of slabs above

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Primary reinforcement 2-1/2

Walls 2

Footings and base slabs:
Formed surfaces..2
Top of footings and base slabs 2

Shells, folded plate members.......................1-1/2

(c) Conditions not covered in 7.7.1(a) and (b):
Slabs and joists:
No. 11 bars and smaller 3/4
No. 14 and No. 18 bars.............................1-1/2

Beams and columns:
Stirrups, spirals, and ties 1-1/2
Primary reinforcement 2

Walls:
No. 11 bars and smaller 3/4
No. 14 and No. 18 bars.............................1-1/2

Shells, folded plate members:
No. 5 bars, W31 or D31 wire and smaller.... 1/2
No. 6 bars and larger..................................3/4

7.7.2 — Precast concrete (manufactured under plant control conditions)

The following minimum concrete cover shall be provided for prestressed and nonprestressed reinforcement, ducts, and end fittings, but shall not be less than required by 7.7.6:

Minimum cover, in.

(a) Concrete exposed to earth, liquid, weather, or bearing on work mat, or slabs supporting earth cover:
Slabs and joists...1-1/2

Beams and columns:
Stirrups, spirals, and ties 1-1/2
Primary reinforcement 2

Walls 1-1/2

Shells, folded plate members............................. 1

(b) Conditions not covered in 7.7.2(a):
Slabs and joists:
No. 11 bars, prestressing tendons 1-1/2 in. diameter and smaller..................................3/4
No. 14 and No. 18 bars, prestressing tendons larger than 1-1/2 in. diameter 1-1/2

Beams and columns:
Stirrups, spirals, and ties 1
Primary reinforcement 1-1/2

COMMENTARY

liquid contents in closed structures (openings in slab less than 25 percent of total area) are considered to be directly "exposed." Where the potential for hydrogen sulfide generation is present, the soffit should also be protected with an acid-resistant liner.

Alternative methods of protecting the reinforcement from weather may be provided if they are equivalent to the additional concrete cover required by the code. When approved by the building official under the provisions of 1.4, reinforcement with alternative protection from the weather may have concrete cover not less than the cover required for reinforcement not exposed to weather.

The development length given in Chapter 12 is now a function of the bar cover. As a result, it may be desirable to use larger than minimum cover in some cases.

R7.7.2 — Precast concrete (manufactured under plant control conditions)

The lesser cover thicknesses for precast construction reflect the greater convenience of control for proportioning, placing, and curing inherent in precasting. The term "manufactured under plant control conditions" does not specifically imply that precast members should be manufactured in a plant. Structural elements precast at the job site will also qualify under this section if the control of form dimensions, placing of reinforcement, quality control of concrete, and curing procedure are equal to that normally expected in a plant.

Concrete cover to a pretensioned strand as described in this section is intended to provide minimum protection against weather and other effects. Such cover may not be sufficient to transfer or develop the stress in the strand, and it may be necessary to increase the cover accordingly.

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Walls:
No. 11 bars, prestressing tendons 1-1/2 in. diameter and smaller 3/4
No. 14 and No. 18 bars, prestressing tendons larger than 1-1/2 in. diameter.................. 1-1/2

Shells, folded plate members:
No. 5 bars, W31 or D31 wire and smaller 3/4
No. 6 bars and larger 1

7.7.3 — Cast-in-place concrete (prestressed)

7.7.3.1 — The following minimum concrete cover shall be provided for prestressed and nonprestressed reinforcement, ducts, and end fittings, but shall not be less than required by 7.7.6:

Minimum cover, in.

(a) Concrete cast against and permanently exposed to earth 3

(b) Concrete exposed to earth, liquid, weather, or bearing on work mat, or slabs supporting earth cover:

Slabs and joists 1-1/2

Beams and columns:
Stirrups, spirals, and ties......................... 1-1/2
Primary reinforcement 2

Walls.. 1-1/2

Shells, folded plate members 1

(c) Conditions not covered in 7.7.3.1(a) and (b):
Slabs and joists:
No. 11 bars and smaller..............................3/4
No. 14 and No. 18 bars........................... 1-1/2

Beams and columns:
Stirrups, spirals, and ties................................1
Primary reinforcement 1-1/2

Walls:
No. 11 bars and smaller..............................3/4
No. 14 and No. 18 bars........................... 1-1/2

Shells, folded plate members
No. 5 bars, W31 and D31 wire and smaller 3/4
No. 6 bars and larger 1

7.7.3.2 — For prestressed concrete members exposed to earth, weather, or corrosive environments, and in which permissible tensile stress of 18.4.2(c) is exceeded, minimum cover shall be increased 50 percent.

7.7.3.3 — For prestressed concrete members manufactured under plant control conditions, minimum concrete cover for nonprestressed reinforcement shall be as required in 7.7.2.

COMMENTARY

R7.7.3 — Cast-in-place concrete (prestressed)

The lower required cover concrete for prestressed concrete construction reflects the lower amount of cracking inherent in prestressed concrete relative to conventionally reinforced concrete.

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COMMENTARY

7.7.4 — Bundled bars

For bundled bars, minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 2 in.; except for concrete cast against and permanently exposed to earth, where minimum cover shall be 3 in.

7.7.5 — Future extensions

Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

7.7.6 — Fire protection

When the general building code (of which this code forms a part) requires a thickness of cover for fire protection greater than the minimum concrete cover specified in 7.7, such greater thicknesses shall be used.

7.8 — Special reinforcement details for columns

R7.8 — Special reinforcement details for columns

7.8.1 — Offset bars

Offset bent longitudinal bars shall conform to the following:

7.8.1.1 — Slope of inclined portion of an offset bar with axis of column shall not exceed 1 in 6.

7.8.1.2 — Portions of bar above and below an offset shall be parallel to axis of column.

7.8.1.3 — Horizontal support at offset bends shall be provided by lateral ties, spirals, or parts of the floor construction. Horizontal support provided shall be designed to resist 1-1/2 times the horizontal component of the computed force in the inclined portion of an offset bar. Lateral ties or spirals, if used, shall be placed not more than 6 in. from points of bend.

7.8.1.4 — Offset bars shall be bent before placement in the forms. See 7.3.

7.8.1.5 — Where a column face is offset 3 in. or greater, longitudinal bars shall not be offset bent. Separate dowels, lap spliced with the longitudinal bars adjacent to the offset column faces, shall be provided. Lap splices shall conform to 12.17.

7.8.2 — Steel cores

Load transfer in structural steel cores of composite compression members shall be provided by the following:

7.8.2.1 — Ends of structural steel cores shall be accurately finished to bear at end bearing splices, with positive provision for alignment of one core above the other in concentric contact.

R7.8.2 — Steel cores

The 50 percent limit on transfer of compressive load by end bearing on ends of structural steel cores is intended to provide some tensile capacity at such splices (up to 50 percent), because the remainder of the total compressive stress in the steel core is to be transmitted by dowels, splice plates, welds, etc. This provision should ensure that splices in composite compression members meet essentially the same

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7.8.2.2 — At end bearing splices, bearing shall be considered effective to transfer not more than 50 percent of the total compressive stress in the steel core.

7.8.2.3 — Transfer of stress between column base and footing shall be designed in accordance with 15.8.

7.8.2.4 — Base of structural steel section shall be designed to transfer the total load from the entire composite member to the footing; or the base shall be designed to transfer the load from the steel core only, provided ample concrete section is available for transfer of the portion of the total load carried by the reinforced concrete section to the footing by compression in the concrete and by reinforcement.

COMMENTARY

tensile capacity as required for conventionally reinforced concrete compression members.

7.9 — Connections

7.9.1 — At connections of principal framing elements (such as beams and columns), enclosure shall be provided for splices of continuing reinforcement and for anchorage of reinforcement terminating in such connections.

7.9.2 — Enclosure at connections shall consist of external concrete or internal closed ties, spirals, or stirrups.

R7.9 — Connections

Confinement is essential at connections to ensure that the flexural capacity of the members can be developed without deterioration of the joint under repeated loadings.[7.12,7.13]

7.10 — Lateral reinforcement for compression members

7.10.1 — Lateral reinforcement for compression members shall conform to the provisions of 7.10.4 and 7.10.5 and, where shear or torsion reinforcement is required, shall also conform to provisions of Chapter 11.

7.10.2 — Lateral reinforcement requirements for composite compression members shall conform to 10.16. Lateral reinforcement requirements for tendons shall conform to 18.11.

7.10.3 — It shall be permitted to waive the lateral reinforcement requirements of 7.10, 10.16, and 18.11 where tests and structural analysis show adequate strength and feasibility of construction.

R7.10 — Lateral reinforcement for compression members

R7.10.3 — Precast columns with cover less than 1-1/2 in., prestressed columns without longitudinal bars, columns smaller than minimum dimensions prescribed in the ACI 350-01 edition, columns of concrete with small size coarse aggregate, wall-like columns, and other special cases may require special designs for lateral reinforcement. Plain or deformed wire, W4, D4, or larger, may be used for ties or spirals. If such special columns are considered as spiral columns for load strength in design, the ratio of spiral reinforcement ρ_s is to conform to 10.9.3.

7.10.4 — Spirals

Spiral reinforcement for compression members shall conform to 10.9.3 and to the following:

7.10.4.1 — Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled

R7.10.4 — Spirals

For practical considerations in cast-in-place construction, the minimum diameter of spiral reinforcement is 3/8 in. (3/8 in., No. 3 bar, or W11 or D11 wire). This is the smallest size that can be used in a column with 1-1/2 in. or more cover and having concrete strengths of 3000 psi or more if the minimum clear spacing for placing concrete is to be maintained.

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to permit handling and placing without distortion from designed dimensions.

7.10.4.2 — For cast-in-place construction, size of spirals shall not be less than 3/8 in. diameter.

7.10.4.3 — Clear spacing between spirals shall not exceed 3 in., nor be less than 1 in. See also 3.3.2.

7.10.4.4 — Anchorage of spiral reinforcement shall be provided by 1-1/2 extra turns of spiral bar or wire at each end of a spiral unit.

7.10.4.5 — Spiral reinforcement shall be spliced, if needed, by any of the following methods:

(a) Lap splices not less than the larger of 12 in. and the length indicated in one of (1) through (5) of the following:

(1) Deformed uncoated bar or wire.......... **$48d_b$**

(2) Plain uncoated bar or wire .... **$72d_b$**

(3) Epoxy-coated deformed bar or wire .... **$72d_b$**

(4) Plain uncoated bar or wire with a standard stirrup or tie hook in accordance with 7.1.3 at ends of lapped spiral reinforcement. The hooks shall be embedded within the core confined by the spiral reinforcement ... **$48d_b$**

(5) Epoxy-coated deformed bar or wire with a standard stirrup or tie hook in accordance with 7.1.3 at ends of lapped spiral reinforcement. The hooks shall be embedded within the core confined by the spiral reinforcement .... **$48d_b$**

(b) Full mechanical or welded splices in accordance with 12.14.3.

7.10.4.6 — Spirals shall extend from top of footing or slab in any story to level of lowest horizontal reinforcement in members supported above.

7.10.4.7 — Where beams or brackets do not frame into all sides of a column, ties shall extend above termination of spiral to bottom of slab or drop panel.

7.10.4.8 — In columns with capitals, spirals shall extend to a level at which the diameter or width of capital is two times that of the column.

7.10.4.9 — Spirals shall be held firmly in place and true to line.

7.10.5 — Ties

Tie reinforcement for compression members shall conform to the following:

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Standard spiral sizes are 3/8, 1/2, and 5/8 in. diameter for hot rolled or cold drawn material, plain or deformed.

The code allows spirals to be terminated at the level of lowest horizontal reinforcement framing into the column. If one or more sides of the column are not enclosed by beams or brackets, however, ties are required from the termination of the spiral to the bottom of the slab or drop panel. If beams or brackets enclose all sides of the column but are of different depths, the ties should extend from the spiral to the level of the horizontal reinforcement of the shallowest beam or bracket framing into the column. These additional ties are to enclose the longitudinal column reinforcement and the portion of bars from beams bent into the column for anchorage. See also 7.9.

Spirals should be held firmly in place, at proper pitch and alignment, to prevent displacement during concrete placement. The ACI 318 code has traditionally required spacers to hold the fabricated spiral cage in place, but was changed in 1989 to allow alternate methods of installation. When spacers are used, the following may be used for guidance: for spiral bar or wire smaller than 5/8 in. diameter, a minimum of two spacers should be used for spirals less than 20 in. in diameter, three spacers for spirals 20 to 30 in. in diameter, and four spacers for spirals greater than 30 in. in diameter. For spiral bar or wire 5/8 in. diameter or larger, a minimum of three spacers should be used for spirals 24 in. or less in diameter, and four spacers for spirals greater than 24 in. in diameter. The project specifications or subcontract agreements should be clearly written to cover the supply of spacers or field tying of the spiral reinforcement. In the current code, splice requirements were modified for epoxy-coated and plain spirals and to allow mechanical splices.

R7.10.5 — Ties

All longitudinal bars in compression should be enclosed within lateral ties. Where longitudinal bars are arranged in a

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7.10.5.1 — All nonprestressed bars shall be enclosed by lateral ties, at least No. 3 in size for longitudinal bars No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, and bundled longitudinal bars. Deformed wire or welded wire fabric of equivalent area shall be permitted.

COMMENTARY

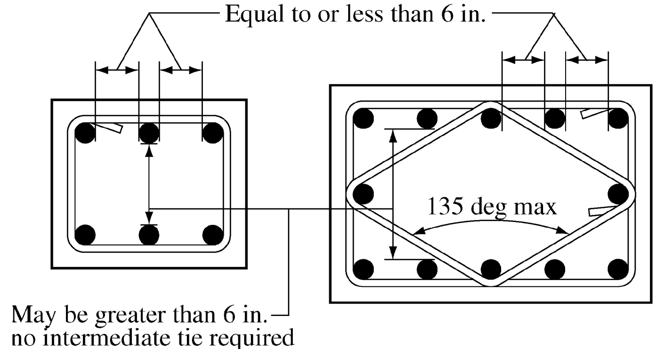

Fig. R7.10.5—Sketch to clarify measurements between laterally supported column bars.

circular pattern, only one circular tie per specified spacing is required. This requirement can be satisfied by a continuous circular tie (helix) at larger pitch than required for spirals under 10.9.3, the maximum pitch being equal to the required tie spacing.

The ACI 318-56 code required "lateral support equivalent to that provided by a 90-deg corner of a tie," for every vertical bar. Tie requirements were liberalized in 1963 by increasing the permissible included angle from 90 to 135 deg and exempting bars that are located within 6 in. clear on each side along the tie from adequately tied bars (see Fig. R7.10.5). Limited tests[7.14] on full-size, axially-loaded, tied columns containing full-length bars (without splices) showed no appreciable difference between ultimate strengths of columns with full tie requirements and no ties at all.

Because spliced bars and bundled bars were not included in the tests of Reference 7.14, it is prudent to provide a set of ties at each end of lap spliced bars, above and below end-bearing splices, and at minimum spacings immediately below sloping regions of offset bent bars.

Standard tie hooks are intended for use with deformed bars only, and should be staggered where possible. See also 7.9.

Continuously wound bars or wires can be used as ties provided their pitch and area are at least equivalent to the area and spacing of separate ties. Anchorage at the end of a continuously wound bar or wire should be by a standard hook as for separate bars or by one additional turn of the tie pattern. A circular continuously wound bar or wire is considered a spiral if it conforms to 7.10.4, otherwise it is considered a tie.

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7.10.5.2 — Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie bar or wire diameters, or least dimension of the compression member.

7.10.5.3 — Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 deg and no bar shall be farther than 6 in. clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie shall be permitted.

7.10.5.4 — Ties shall be located vertically not more than one-half a tie spacing above the top of footing or slab in any story, and shall be spaced as provided herein to not more than one-half a tie spacing below the lowest horizontal reinforcement in slab or drop panel above.

7.10.5.5 — Where beams or brackets frame from four directions into a column, termination of ties not more than 3 in. below lowest reinforcement in shallowest of such beams or brackets shall be permitted.

7.10.5.6 — Where anchor bolts are placed in the top of columns or pedestals, the bolts shall be enclosed by lateral reinforcement that also surrounds at least four vertical bars of the column or pedestal. The lateral reinforcement shall be distributed within 5 in. of the top of the column or pedestal, and shall consist of at least two No. 4 or three No. 3 bars.

7.11 — Lateral reinforcement for flexural members

7.11.1 — Compression reinforcement in beams shall be enclosed by ties or stirrups satisfying the size and spacing limitations in 7.10.5 or by welded wire fabric of equivalent area. Such ties or stirrups shall be provided throughout the distance where compression reinforcement is required.

7.11.2 — Lateral reinforcement for flexural framing members subject to stress reversals or to torsion at supports shall consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement.

7.11.3 — Closed ties or stirrups shall be formed in one piece by overlapping standard stirrup or tie end hooks around a longitudinal bar, or formed in one or two pieces lap spliced with a Class B splice (lap of $1.3\ell_d$) or anchored in accordance with 12.13.

7.12 — Shrinkage and temperature reinforcement

7.12.1 — Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement shall be provided

COMMENTARY

R7.10.5.5 — With the ACI 318-83 code, the wording of this section was modified to clarify that ties may be terminated only when elements frame into all four sides of square and rectangular columns; for round or polygonal columns, such elements frame into the column from four directions.

R7.10.5.6 — Confinement of anchor bolts that are placed in the top of columns or pedestals improves load transfer from the anchor bolts to the column or pier for situations where the concrete cracks in the vicinity of the bolts. Such cracking can occur due to unanticipated forces caused by temperature, restrained shrinkage, and similar effects.

R7.11 — Lateral reinforcement for flexural members

R7.11.1 — Compression reinforcement in beams and girders should be enclosed to prevent buckling; similar requirements for such enclosure have remained essentially unchanged through several editions of the code, except for minor clarification.

R7.12 — Shrinkage and temperature reinforcement

R7.12.1 — Shrinkage and temperature reinforcement is required normal to the flexural reinforcement to minimize

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in structural slabs and walls where the flexural reinforcement extends in one direction only.

7.12.1.1 — Shrinkage and temperature reinforcement shall be provided in accordance with either 7.12.2 or 7.12.3.

7.12.1.2 — Where shrinkage and temperature movements are significantly restrained, the requirements of 8.2.4 and 9.2.3 shall be considered.

7.12.2 — Deformed reinforcement conforming to 3.5.3 used for shrinkage and temperature reinforcement shall be provided in accordance with the following:

7.12.2.1 — For members subjected to environmental exposure conditions or required to be liquid-tight, the area of shrinkage and temperature reinforcement shall provide at least the ratios of reinforcement area to gross concrete area shown in Table 7.12.2.1:

Concrete sections that are at least 24 in. may have the minimum shrinkage and temperature reinforcement based on a 12 in. concrete layer at each face. The reinforcement in the bottom of base slabs in contact with soil may be reduced to 50 percent of that required in Table 7.12.2.1.

TABLE 7.12.2.1—MINIMUM SHRINKAGE AND TEMPERATURE REINFORCEMENT

Length between movement joints, ft	Minimum shrinkage and temperature reinforcement ratio	
	Grade 40	Grade 60
Less than 20	0.0030	0.0030
20 to less than 30	0.0040	0.0030
30 to less than 40	0.0050	0.0040
40 and greater	0.0060*	0.0050*

*Maximum shrinkage and temperature reinforcement where movement joints are not provided.

Note: This table applies to spacing between expansion joints and full contraction joints. When used with partial contraction joints, the minimum reinforcement ratio shall be determined by multiplying the actual length between partial contraction joints by 1.5.

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cracking and to tie the structure together to ensure it is acting as assumed in the design. Where restraint is present to develop shrinkage and temperature stresses in the same direction as flexural stresses, the section may need to be checked for sufficient reinforcement for each kind of stress.

R7.12.1.2 — The area of shrinkage and temperature reinforcement required by 7.12 has been satisfactory where shrinkage and temperature movements are permitted to occur. For cases where structural walls or large columns provide significant restraints to shrinkage and temperature movements, it may be necessary to increase the amount of reinforcement normal to the flexural reinforcement in 7.12.1.2 (see Reference 7.15). Top and bottom reinforcement are both effective in controlling cracks. Control strips during the construction period, which permit initial shrinkage to occur without causing an increase in stresses, are also effective in reducing cracks caused by restraints.

R7.12.2 — The amounts given for deformed bars and welded wire fabric are empirical but have been used satisfactorily for many years. Splices and end anchorages of shrinkage and temperature reinforcement must be designed for the full specified yield strength in accordance with 12.1, 12.15, 12.18, and 12.19.

R7.12.2.1 — The required amount of shrinkage and temperature reinforcement is a function of the distance between the movement joints that will minimize cracking perpendicular to the reinforcement. In addition, the amount of shrinkage and temperature reinforcement is a function of the particular concrete mixture and other properties, the amount of aggregate, the member thickness, its reinforcement, and the environmental conditions of the site. These factors have been considered in applying the analysis method developed by Vetter[7.16] to environmental engineering concrete structures, and the recommendations contained in the remainder of this section are based on that work.[7.17]

When shrinkage-compensating concrete is used per manufacturer's recommendations, no less than 0.3 percent reinforcement should be provided.

Where positive means are taken to substantially reduce restraint, the amount of temperature and shrinkage reinforcement and the distance between movement joints may be adjusted accordingly.

Consideration may be given to reducing the amount of shrinkage and temperature reinforcement shown in Table 7.12.2.1 when details are developed in accordance with ACI 223 recommendations.

Where movement joints are not provided, shrinkage and temperature reinforcement need not exceed the values listed in Table 7.12.2.1 for greater than 40 ft joint spacing.

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7.12.2.2 — Shrinkage and temperature reinforcement shall not be spaced farther apart than 12 in. and the minimum bar size shall be No. 4. No less than 1/3 of the required area of shrinkage and temperature steel shall be distributed at any one face.

7.12.2.3 — At all sections where required, reinforcement for shrinkage and temperature stresses shall develop the specified yield strength f_y in tension in accordance with Chapter 12.

7.12.3 — Prestressing steel conforming to 3.5.5 used for shrinkage and temperature reinforcement shall be provided in accordance with the following:

7.12.3.1 — Tendons shall be proportioned to provide a minimum average compressive stress in accordance with 18.3.

7.12.3.2 — Spacing of tendons shall not exceed 6 ft.

7.12.3.3 — When spacing of tendons exceeds 54 in., additional bonded shrinkage and temperature reinforcement conforming to 7.12.2 shall be provided between the tendons at slab edges extending from the slab edge for a distance equal to the tendon spacing.

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R7.12.2.2 — The shrinkage and temperature reinforcement is normally divided equally between both concrete faces. Where special conditions exist that significantly change the rate of drying or cooling on one face of the member, shrinkage and temperature reinforcement may be adjusted accordingly.

R7.12.3 — Prestressed reinforcement requirements have been selected to provide an effective force on the slab approximately equal to the yield strength force for nonprestressed shrinkage and temperature reinforcement. The amount of prestressing indicated on the gross concrete area has been successfully used on a large number of environmental engineering concrete structures. When the spacing of tendons used for shrinkage and temperature reinforcement exceeds 54 in., additional bonded reinforcement should be provided at slab edges where the prestressing forces are applied. Adequately reinforce the area between the slab edge and the point where compressive stresses behind individual anchorages have spread sufficiently that the slab is uniformly in compression. Application of the provisions of 7.12.3 to monolithic cast-in-place post-tensioned beam and slab construction is illustrated in Fig. R7.12.3.

7.13 — Requirements for structural integrity

7.13.1 — In the detailing of reinforcement and connections, members of a structure shall be effectively tied together to improve integrity of the overall structure.

R7.13 — Requirements for structural integrity

Experience has shown that the overall integrity of a structure can be substantially enhanced by minor changes in detailing of reinforcement. It is the intent of this section of the code to improve the redundancy and ductility in structures so that in the event of damage to a major supporting element or an abnormal loading event, the resulting damage may be confined to a relatively small area and the structure will have a better chance to maintain overall stability.

R7.13.1 — It is important to consider the effect of discontinuities, caused by expansion joints in base slabs or walls, which could result in an unstable structure.

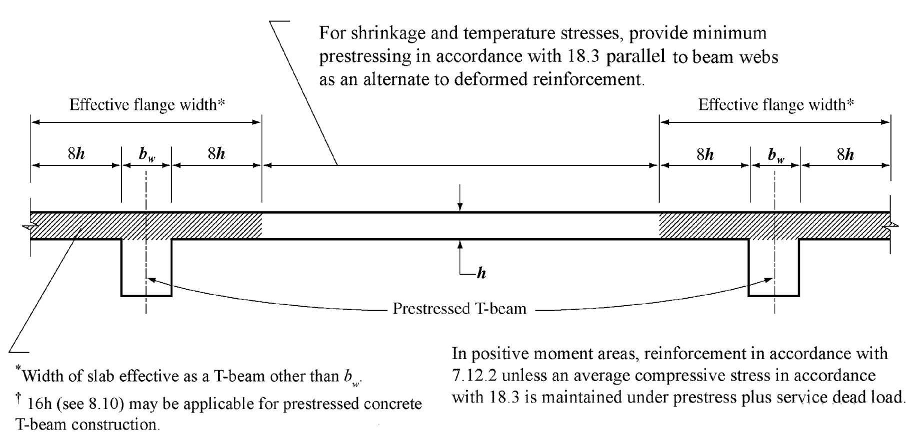

Fig. R7.12.3—Prestressing used for shrinkage and temperature.

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7.13.2 — For cast-in-place construction, the following shall constitute minimum requirements:

7.13.2.1 — In joist construction, at least one bottom bar shall be continuous or shall be spliced with a Class A tension splice or a mechanical or welded splice satisfying 12.14.3 and at noncontinuous supports shall be terminated with a standard hook.

7.13.2.2 — Beams along the perimeter of the structure shall have continuos reinforcement consisting of:

(a) At least one-sixth of the tension reinforcement required for negative moment at the support, but not less than two bars; and

(b) At least one-quarter of the tension reinforcement required for positive moment at midspan, but not less than two bars.

7.13.2.3 — Where splices are needed to provide the required continuity, the top reinforcement shall be spliced at or near midspan and bottom reinforcement shall be spliced at or near the support. Splices shall be Class A tension splices or mechanical or welded splices satisfying 12.14.3. The continuous reinforcement required in 7.13.2.2(a) and 7.13.2.2(b) shall be enclosed by the corners of U-stirrups having not less than 135-deg hooks around the continuous top bars, or by one-piece closed stirrups with not less than 135-deg hooks around one of the continuous top bars. Stirrups need not be extended through any joints.

7.13.2.4 — In other than perimeter beams, when stirrups, as defined in 7.13.2.3 are not provided, at least one-quarter of the positive moment reinforcement required at midspan, but not less than two bars, shall be continuous or shall be spliced over or near the support with a Class A tension splice or a mechanical or welded splice satisfying 12.14.3, and at noncontinuous supports shall be terminated with a standard hook.

7.13.2.5 — For two-way slab construction, see 13.3.8.5.

7.13.3 — For precast concrete construction, tension ties shall be provided in the transverse, longitudinal, and vertical directions and around the perimeter of the structure to effectively tie elements together. The provisions of 16.5 shall apply.

7.13.4 — For lift-slab construction, see 13.3.8.6 and 18.12.6.

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R7.13.2 — With damage to a support, top reinforcement that is continuous over the support, but not confined by stirrups, will tend to tear out of the concrete and will not provide the catenary action needed to bridge the damaged support. By making a portion of the bottom reinforcement continuous, catenary action can be provided.

Requiring continuous top and bottom reinforcement in perimeter or spandrel beams provides a continuous tie around the structure. It is not the intent to require a tensile tie of continuous reinforcement of constant size around the entire perimeter of a structure, but simply to require that one half of the top flexural reinforcement required to extend past the point of inflection by 12.12.3 be further extended and spliced at or near midspan. Similarly, the bottom reinforcement required to extend into the support by 12.11.1 should be made continuous or spliced with bottom reinforcement from the adjacent span. If the depth of a continuous beam changes at a support, the bottom reinforcement in the deeper member should be terminated with a standard hook and bottom reinforcement in the shallower member should be extended into and fully developed in the deeper member.

In the current code, provisions were added to permit the use of mechanical or welded splices for splicing reinforcement, and the detailing requirements for the longitudinal reinforcement and stirrups in beams were revised. Section 7.13.2 was revised to require U-stirrups with not less than 135-deg hooks around the continuous bars, or one-piece close stirrups, because a cross tie forming the top of a two-piece closed stirrup is ineffective in preventing the top continuous bars from tearing out of the top of the beam.

R7.13.3 — The code requires tension ties for precast concrete buildings of all heights. Details should provide connections to resist applied loads. Connection details that rely solely on friction caused by gravity forces are not permitted.

Connection details should be arranged so as to minimize the potential for cracking due to restrained creep, shrinkage, and temperature movements. For information on connections and detailing requirements, see Reference 7.18.

Reference 7.19 recommends minimum tie requirements for precast concrete bearing wall buildings.

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COMMENTARY

Notes

PART 4 — GENERAL REQUIREMENTS

CHAPTER 8 — ANALYSIS AND DESIGN — GENERAL CONSIDERATIONS

CODE

8.0 — Notation

A_s = area of nonprestressed tension reinforcement, in.^{2}
A_s' = area of compression reinforcement, in.^{2}
b = width of compression face of member, in.
d = distance from extreme compression fiber to centroid of tension reinforcement, in.
E_c = modulus of elasticity of concrete, psi. See 8.5.1
E_s = modulus of elasticity of reinforcement, psi. See 8.5.2 and 8.5.3
f_c' = specified compressive strength of concrete, psi
f_y = specified yield strength of nonprestressed reinforcement, psi
ℓ_n = clear span for positive moment or shear and average of adjacent clear spans for negative moment, in.
V_c = nominal shear strength provided by concrete, lb
w_c = unit weight of concrete, lb/ft^3
w_u = factored load per unit length of beam or per unit area of slab
β_1 = factor defined in 10.2.7.3
ε_t = net tensile strain in extreme tension steel at nominal strength
ρ = ratio of nonprestressed tension reinforcement = A_s/bd
ρ' = ratio of nonprestressed compression reinforcement = A_s'/bd
ρ_b = reinforcement ratio producing balanced strain conditions. See 10.3.2
ϕ = strength reduction factor. See 9.3

8.1 — Design methods

8.1.1 — In design of environmental engineering concrete structures, members shall be proportioned for adequate strength and serviceability in accordance with provisions of this code, using load factors and strength reduction factors ϕ specified in Chapter 9.

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R8.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip.

The definition of net tensile strain in 2.1 excludes strains due to effective prestress, creep, shrinkage, and temperature.

R8.1 — Design methods

R8.1.1 — The strength design method requires service loads or related internal moments and forces to be increased by specified load factors (required strength) and computed nominal strengths to be reduced by specified strength reduction factors ϕ (design strength).

Many current design aids are based on the strength design method. For strength design of environmental engineering concrete structures, the code requires the required strength be increased by the environmental durability factor. Using this factor produces conservative service-load stresses in nonprestressed reinforcement, and crack control similar to that historically obtained with the alternate design method.

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For additional design information when using concrete made with shrinkage-compensating cement, see Chapter 3 of ACI 223[8.1] combined with recommendations of previous ACI 350 reports.

8.1.2 — Design of nonprestressed reinforced concrete members using Appendix I, Alternate Design Method, shall be permitted.

R8.1.2 — The alternate method of design, outlined in Appendix I, is similar to the working stress design method of the 1963 ACI Building Code. The general serviceability requirements of the code, such as the requirements for deflection and crack control must be met whether the strength design method of the code or the alternate design method of Appendix I is used.

Although prestressed members may not be designed under the provisions of the alternate design method, Chapter 18 requires linear stress-strain assumptions for computing service load stresses and prestress transfer stresses for investigation of behavior at service conditions, while using the strength design method for computing flexural strength (see 18.7).

In environmental engineering concrete structures, service-load performance is of paramount importance.

For environmental engineering concrete structures, minimal cracking is generally a paramount requisite. For some types of structures, leakage into potable water or out of contaminated water facilities or hazardous material containment must be avoided to protect the public health. For other types of structures, cracking and leakage should be avoided to prevent deterioration and ensure adequate service life. Design procedures are established in the code for the control of cracking and resultant leakage.

8.1.3 — Anchors within the scope of Appendix D, Anchoring to Concrete, installed in concrete to transfer loads between connected elements shall be designed using Appendix D.

8.1.3 — The code has included specific provisions for anchoring to concrete for the first time in the current edition. The new material has been presented as an appendix.

An appendix may be judged not to be an official part of a legal document unless specifically adopted. Therefore, specific reference is made to Appendix D in the main part of the code to make it a legal part of the code.

8.2 — Loading

8.2.1 — Design provisions of this code are based on the assumption that structures shall be designed to resist all applicable loads.

8.2.2 — Service loads shall be in accordance with the general building code of which this code forms a part, with such live load reductions as are permitted in the general building code.

R8.2 — Loading

The provisions in the code are for live, wind, and earthquake loads such as those recommended in **"Minimum Design Loads for Buildings and Other Structures,"** (ASCE 7), of the American Society of Civil Engineers (ASCE) (formerly ANSI A58.1). If the service loads specified by the general building code (of which ACI 350 may form a part) differ from those of ASCE 7, the general building code governs. However, if the nature of the loads contained in a general building code differ considerably from ASCE 7 loads, some provisions of this code may need modification to reflect the difference.

Roofs should be designed with sufficient slope or camber to ensure adequate drainage accounting for any long-term deflection of the roof due to the dead loads, or the loads should be increased to account for all likely accumulations of water. If deflection of roof members may result in ponding of water accompanied by increased deflection and additional ponding, the design must ensure that this process is self-limiting.

Walls of some types of environmental engineering concrete structures may be subject to significant deflections or displacements due to lateral earth pressures and should be designed for the effects of this soil-structure interaction.

ACI 350.4R contains further discussion of loading, as well as a complete table of chemical weights.

8.2.3 — In design for earth, hydrostatic, hydrodynamic, wind, and earthquake loads, integral structural parts shall be designed to resist the total lateral loads.

R8.2.3 — Any reinforced concrete wall that is monolithic with other structural elements is considered to be an "integral part." Partition walls may or may not be integral structural parts. If partition walls may be removed, the primary lateral load resisting system should provide all of the required resistance without contribution of the removable partition. The effects of all partition walls attached to the structure, however, must be considered in the analysis of the structure because they may lead to increased design forces in some or all elements. Special provisions for seismic design are given in Chapter 21.

8.2.4 — Consideration shall be given to effects of forces due to prestressing, crane loads, vibration, impact, shrinkage, temperature changes, creep, and unequal settlement of supports. Consideration shall also be given to the effects caused by the interior pressure of the structure, movement of joints and joint spacing, filling and emptying of tanks, and ice formation in cold climates.

R8.2.4 — Concrete superstructure built over the tanks should be designed to similar crack control criteria as the tanks, due to the possibility of freezing and thawing of moisture in the structure.

8.3 — Methods of analysis

8.3.1 — All members of frames or continuous construction shall be designed for the maximum effects of factored loads as determined by the theory of elastic analysis, except as modified according to 8.4. It shall be permitted to simplify design by using the assumptions specified in 8.6 through 8.9.

8.3.2 — Except for prestressed concrete, approximate methods of frame analysis are permitted for environmental structures of usual types of construction, spans, and story heights.

8.3.3 — As an alternate to frame analysis, the following approximate moments and shears shall be permitted for design of continuous beams and one-way slabs (slabs reinforced to resist flexural stresses in only one direction), provided:

(a) There are two or more spans;

R8.3 — Methods of analysis

R8.3.1 — Factored loads are service loads multiplied by appropriate load factors. If the alternate design method of Appendix I is used, the loads used in design are service loads (load factors of unity). For both the strength design method and the alternate design method, elastic analysis is used to obtain moments, shears, reactions, etc.

Moment and shear coefficients generally used for analysis of plates and shells used in environmental structures are contained in References 8.2, 8.3, 8.4, and 8.5.

R8.3.3 — The approximate moments and shears give reasonably conservative values for the stated conditions if the flexural members are part of a frame or continuous construction. Because the load patterns that produce critical values for moments in columns of frames differ from those for maximum negative moments in beams, column moments must be evaluated separately.

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(b) Spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 percent;

(c) Loads are uniformly distributed,

(d) Unit live load does not exceed three times unit dead load; and

(e) Members are prismatic.

Positive moment

End spans

Discontinuous end unrestrained .... $w_u \ell_n^2/11$

Discontinuous end integral with support .... $w_u \ell_n^2/14$

Interior spans .... $w_u \ell_n^2/16$

Negative moments at exterior face of first interior support

Two spans .... $w_u \ell_n^2/9$

More than two spans .... $w_u \ell_n^2/10$

Negative moment at other faces of interior supports .... $w_u \ell_n^2/11$

Negative moment at face of all supports for

Slabs with spans not exceeding 10 ft; and beams where ratio of sum of column stiffnesses to beam stiffness exceeds eight at each end of the span .... $w_u \ell_n^2/12$

Negative moment at interior face of exterior support for members built integrally with supports

Where support is spandrel beam .... $w_u \ell_n^2/24$

Where support is a column .... $w_u \ell_n^2/16$

Shear in end members at face of first interior support .... $1.15 w_u \ell_n/2$

Shear at face of all other supports .... $w_u \ell_n/2$

8.4 — Redistribution of negative moments in continuous flexural members

8.4.1 — Except where approximate values for moments are used, it shall be permitted to increase or decrease negative moments calculated by elastic

COMMENTARY

R8.4 — Redistribution of negative moments in continuous flexural members

Moment redistribution is dependent on adequate ductility in plastic hinge regions. These plastic hinge regions develop at points of maximum moment and cause a shift in the elastic

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theory at supports of continuous flexural members for any assumed loading arrangement by not more than1000 ε_t percent, with a maximum of 20 percent.

8.4.2 — The modified negative moments shall be used for calculating moments at sections within the spans.

8.4.3 — Redistribution of negative moments shall be made only when ε_t is equal to or greater than 0.0075 at the section at which moment is reduced.

COMMENTARY

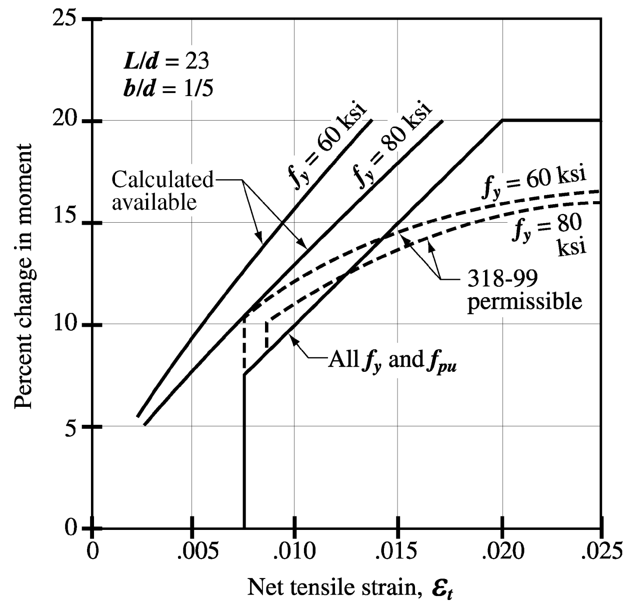

Fig R8.4—Permissible moment redistribution for minimum rotation capacity.

moment diagram. The usual result is a reduction in the values of negative moments in the plastic hinge region and an increase in the values of positive moments from those computed by elastic analysis. Because negative moments are determined for one loading arrangement and positive moments for another, each section has a reserve capacity that is not fully utilized for any one loading condition. The plastic hinges permit the utilization of the full capacity of more cross sections of a flexural member at ultimate loads. Using conservative values of limiting concrete strains and lengths of plastic hinges derived from extensive tests, flexural members with small rotation capacity were analyzed for moment redistribution up to 20 percent, depending on the reinforcement ratio. The results were found to be conservative (see Fig. R8.4). Studies by Cohn[8.6] and Mattock[8.7] support this conclusion and indicate that cracking and deflection of beams designed for moment redistribution are not significantly greater at service loads than for beams designed by the elastic theory distribution of moments. Also, these studies indicated that adequate rotation capacity for the moment redistribution allowed by the code is available if the members satisfy the code requirements.

Moment redistribution does not apply to members designed by the alternate design method of Appendix I; nor may it be used for slab systems designed by the direct design method (see 13.6.1.7).

In the previous code, Section 8.4 specified the permissible redistribution percentage in terms of reinforcement indices. The 2006 code specifies the permissible redistribution percentage in terms of the net tensile strain. See Reference 8.8 for a comparison of these moment redistribution provisions.

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8.5 — Modulus of elasticity

8.5.1 — Modulus of elasticity E_c for concrete shall be permitted to be taken as $w_c^{1.5} 33 \sqrt{f_c'}$ (in psi) for values of w_c between 90 and 155 lb/ft^3. For normalweight concrete, E_c shall be permitted to be taken as $57{,}000\sqrt{f_c'}$.

8.5.2 — Modulus of elasticity E_s for nonprestressed reinforcement shall be permitted to be taken as 29,000,000 psi.

8.5.3 — Modulus of elasticity E_s for prestressing shall be determined by tests or supplied by the manufacturer.

8.6 — Stiffness

8.6.1 — Use of any set of reasonable assumptions shall be permitted for computing relative flexural and torsional stiffnesses of columns, walls, floors, and roof systems. The assumptions adopted shall be consistent throughout analysis.

8.6.2 — Effect of haunches shall be considered both in determining moments and in design of members.

8.7 — Span length

8.7.1 — Span length of members not built integrally with supports shall be considered the clear span plus depth of member but need not exceed distance between centers of supports.

COMMENTARY

R8.5 — Modulus of elasticity

R8.5.1 — Studies leading to the expression for modulus of elasticity of concrete in 8.5.1 are summarized in Reference 8.9 where E_c was defined as the slope of the line drawn from a stress of zero to a compressive stress of $0.45f_c'$. The modulus for concrete is sensitive to the modulus of the aggregate and may differ from the specified value. Measured values range typically from 120 to 80 percent of the specified value. Methods for determining Young's modulus for concrete are described in Reference 8.10.

R8.6 — Stiffness

R8.6.1 — Ideally, the member stiffnesses EI and GJ should reflect the degree of cracking and inelastic action that has occurred along each member before yielding. However, the complexities involved in selecting different stiffnesses for all members of a frame would make frame analyses inefficient in design offices. Simpler assumptions are required to define flexural and torsional stiffnesses.

For braced frames, relative values of stiffness are important. Two usual assumptions are to use gross EI values for all members or to use half the gross EI of the beam stem for beams and the gross EI for the columns.

For frames that are free to sway, a realistic estimate of EI is desirable and should be used if second-order analyses are carried out. Guidance for the choice of EI for this case is given in R10.11.1.

Two conditions determine whether it is necessary to consider torsional stiffness in the analysis of a given structure: (1) the relative magnitude of the torsional and flexural stiffnesses; and (2) whether torsion is required for equilibrium of the structure (equilibrium torsion) or is due to members twisting to maintain deformation compatibility (compatibility torsion). In the case of compatibility torsion, the torsional stiffness may be neglected. For cases involving equilibrium torsion, torsional stiffness should be considered.

R8.6.2 — Stiffness and fixed-end moment coefficients for haunched members may be obtained from Reference 8.11.

R8.7 — Span length

Beam moments calculated at support centers may be reduced to the moments at support faces for design of beams. Reference 8.12 provides an acceptable method of reducing moments at support centers to those at support faces.

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8.7.2 — In analysis of frames or continuous construction for determination of moments, span length shall be taken as the distance center-to-center of supports.

8.7.3 — For beams built integrally with supports, design on the basis of moments at faces of support shall be permitted.

8.7.4 — It shall be permitted to analyze solid or ribbed slabs built integrally with supports, with clear spans not more than 10 ft, as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and width of beams otherwise neglected.

8.8 — Columns

8.8.1 — Columns shall be designed to resist the axial forces from factored loads on all floors or roof and the maximum moment from factored loads on a single adjacent span of the floor or roof under consideration. Loading condition giving the maximum ratio of moment to axial load shall also be considered.

8.8.2 — In frames or continuous construction, consideration shall be given to the effect of unbalanced floor or roof loads on both exterior and interior columns and of eccentric loading due to other causes.

8.8.3 — In computing gravity load moments in columns, it shall be permitted to assume far ends of columns built integrally with the structure to be fixed.

8.8.4 — Resistance to moments at any floor or roof level shall be provided by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffnesses and conditions of restraint.

8.9 — Arrangement of live load

8.9.1 — It shall be permitted to assume that:

(a) The live load is applied only to the floor or roof under consideration;

(b) The far ends of columns built integrally with the structure are considered to be fixed.

8.9.2 — It shall be permitted to assume that the arrangement of live load is limited to combinations of:

(a) Factored dead load on all spans with full factored live load on two adjacent spans;

(b) Factored dead load on all spans with full factored live load on alternate spans.

COMMENTARY

R8.8 — Columns

Section 8.8 has been developed with the intent of making certain that the most demanding combinations of axial load and moments be identified for design.

Section 8.8.4 has been included to make certain that moments in columns are recognized in design if the girders have been proportioned using 8.3.3. The “moment” in 8.8.4 refers to the difference between the moments in a given vertical plane, exerted at column centerline by members framing into that column.

R8.9 — Arrangement of live load

For determining column, wall, and beam moments and shears caused by gravity loads, the code permits the use of a model limited to the beams in the level considered and the columns above and below that level. Far ends of columns are to be considered as fixed for the purpose of analysis under gravity loads. This assumption does not apply to lateral load analysis. In analysis for lateral loads, however, simplified methods (such as the portal method) may be used to obtain the moments, shears, and reactions for structures that are symmetrical and satisfy the assumptions used for such simplified methods. For unsymmetrical and high-rise structures, rigorous methods recognizing all structural displacements should be used.

The engineer is expected to establish the most demanding sets of design forces by investigating the effects of live load placed in various critical patterns.

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COMMENTARY

Most approximate methods of analysis neglect effects of deflections on geometry and axial flexibility. Therefore, beam and column moments may have to be amplified for column slenderness in accordance with 10.11, 10.12, and 10.13.

8.10 — T-beam construction

8.10.1 — In T-beam construction, the flange and web shall be built integrally or otherwise effectively bonded together.

8.10.2 — Width of slab effective as a T-beam flange shall not exceed one-quarter of the span length of the beam, and the effective overhanging flange width on each side of the web shall not exceed:

(a) Eight times the slab thickness;

(b) One-half the clear distance to the next web.

8.10.3 — For beams with a slab on one side only, the effective overhanging flange width shall not exceed:

(a) One-twelfth the span length of the beam;

(b) Six times the slab thickness;

(c) One-half the clear distance to the next web.

8.10.4 — Isolated beams, in which the T-shape is used to provide a flange for additional compression area, shall have a flange thickness not less than one-half the width of web and an effective flange width not more than four times the width of web.

8.10.5 — Where primary flexural reinforcement in a slab that is considered as a T-beam flange (excluding joist construction) is parallel to the beam, reinforcement perpendicular to the beam shall be provided in the top of the slab in accordance with the following:

8.10.5.1 — Transverse reinforcement shall be designed to carry the factored load on the overhanging slab width assumed to act as a cantilever. For isolated beams, the full width of overhanging flange shall be considered. For other T-beams, only the effective overhanging slab width need be considered.

8.10.5.2 — Transverse reinforcement shall be spaced not farther apart than 12 in.

R8.10 — T-beam construction

This section contains provisions identical to those of previous ACI Building Codes for limiting dimensions related to stiffness and flexural calculations. Special provisions related to T-beams and other flanged members are stated in 11.6.1 with regard to torsion.

8.11 — Joist construction

8.11.1 — Joist construction consists of a monolithic combination of regularly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions.

8.11.2 — Ribs shall be not less than 4 in. in width, and shall have a depth of not more than 3-1/2 times the minimum width of rib.

R8.11 — Joist construction

The size and spacing limitations for concrete joist construction meeting the limitations of 8.11.1 through 8.11.3 are based on successful performance in the past.

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8.11.3 — Clear spacing between ribs shall not exceed 30 in.

8.11.4 — Joist construction not meeting the limitations of 8.11.1 through 8.11.3 shall be designed as slabs and beams.

8.11.5 — When permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to that of the specified strength of concrete in the joists are used:

8.11.5.1 — For shear and negative moment strength computations, it shall be permitted to include the vertical shells of fillers in contact with the ribs. Other portions of fillers shall not be included in strength computations.

8.11.5.2 — Slab thickness over permanent fillers shall be not less than one-twelfth the clear distance between ribs, nor less than 1-1/2 in.

8.11.5.3 — In one-way joists, reinforcement normal to the ribs shall be provided in the slab as required by 7.12.

8.11.6 — When removable forms or fillers not complying with 8.11.5 are used:

8.11.6.1 — Slab thickness shall be not less than one-twelfth the clear distance between ribs, nor less than 2 in.

8.11.6.2 — Reinforcement normal to the ribs shall be provided in the slab as required for flexure, considering load concentrations, if any, but not less than required by 7.12.

8.11.7 — Where conduits or pipes as permitted by 6.3 are embedded within the slab, slab thickness shall be at least 1 in. greater than the total overall depth of the conduits or pipes at any point. Conduits or pipes shall not impair significantly the strength of the construction.

8.11.8 — For joist construction, contribution of concrete to shear strength V_c shall be permitted to be 10 percent more than that specified in Chapter 11. It shall be permitted to increase shear strength using shear reinforcement or by widening the ends of ribs.

8.12 — Separate floor finish

8.12.1 — A floor finish shall not be included as part of a structural member unless placed monolithically with the floor slab or designed in accordance with requirements of Chapter 17.

COMMENTARY

R8.11.3 — A limit on the maximum spacing of ribs is required because of the special provisions permitting higher shear strengths and less concrete protection for the reinforcement for these relatively small, repetitive members.

R8.11.8 — The increase in shear strength permitted by 8.11.8 is justified on the basis of: (1) satisfactory performance of joist construction with higher shear strengths, designed under previous ACI Building Codes, which allowed comparable shear stresses; and (2) redistribution of local overloads to adjacent joists.

R8.12 — Separate floor finish

The code does not specify an additional thickness for wearing surfaces subjected to unusual conditions of wear. The need for added thickness for unusual wear is left to the discretion of the designer.

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8.12.2 — It shall be permitted to consider all concrete floor finishes as part of required cover or total thickness for nonstructural considerations.

COMMENTARY

As in previous editions of the ACI 318 code, a floor finish may be considered for strength purposes only if it is cast monolithically with the slab. Permission is given to include a separate finish in the structural thickness if composite action is provided for in accordance with Chapter 17.

All floor finishes may be considered for nonstructural purposes such as cover for reinforcement, fire protection, etc. Provisions should be made, however, to ensure that the finish will not spall off, thus causing decreased cover. Furthermore, development of reinforcement considerations require minimum monolithic concrete cover according to 7.7.

CHAPTER 9 — STRENGTH AND SERVICEABILITY REQUIREMENTS

CODE

9.0 — Notation

A_g = gross area of section, in.^{2}

A_s = area of nonprestressed tension reinforcement, in.^{2}

A'_s = area of compression reinforcement, in.^{2}

b = width of compression face of member, in.

c = distance from extreme compression fiber to neutral axis, in.

d = distance from extreme compression fiber to centroid of tension reinforcement, in.

d' = distance from extreme compression fiber to centroid of compression reinforcement, in.

d_s = distance from extreme tension fiber to centroid of tension reinforcement, in.

d_t = distance from extreme compression fiber to extreme tension steel, in.

D = dead loads, or related internal moments and forces

E = load effects of earthquake, or related internal moments and forces

E_c = modulus of elasticity of concrete, psi. See 8.5.1

f'_c = specified compressive strength of concrete, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi

f_r = modulus of rupture of concrete, psi

f_s = permissible tensile stress in reinforcement, psi

f_y = specified yield strength of nonprestressed reinforcement, psi

F = loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces

h = overall thickness of member, in.

H = loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces

I_{cr} = moment of inertia of cracked section transformed to concrete, in.^{4}

COMMENTARY

R9.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip.

The definition of net tensile strain in 2.1 excludes strains due to effective prestress, creep, shrinkage, and temperature.

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I_e = effective moment of inertia for computation of deflection, in.^{4}

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.^{4}

ℓ = span length of beam or one-way slab, as defined in 8.7; clear projection of cantilever, in.

ℓ_n = length of clear span in long direction of two-way construction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases, in.

L = live loads, or related internal moments and force

L_r = roof live load, or related internal moments and forces

M_a = maximum moment in member at stage deflection is computed, in.-lb

M_{cr} = cracking moment, in.-lb. See 9.5.2.3

P_n = nominal axial load strength at given eccentricity, lb

R = rain load, or related internal moments and forces

S = snow load, or related internal moments and forces

S_d = environmental durability factor. See 9.2.6

T = cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete

U = required strength to resist factored loads or related internal moments and forces

V_c = nominal shear strength provided by concrete, lb

V_s = nominal shear strength provided by reinforcement, lb

V_u = factored shear force at section, lb

W = wind load, or related internal moments and forces

w_c = unit weight of concrete, lb/ft^3

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, in.

α = ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of beam. See Chapter 13

α_m = average value of α for all beams on edges of a panel

β = ratio of clear spans in long to short direction of two-way slabs

ε_t = net tensile strain in extreme tension steel at nominal strength

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γ = Combined load factor. See 9.2.6

λ = multiplier for additional long-term deflection as defined in 9.5.2.5

ξ = time-dependent factor for sustained load. See 9.5.2.5

ρ = ratio of nonprestressed tension reinforcement, A_s/bd

ρ' = reinforcement ratio for nonprestressed compression reinforcement, A'_s/bd

ρ_b = reinforcement ratio producing balanced strain conditions. See 10.3.2

ϕ = strength reduction factor. See 9.3

9.1 — General

9.1.1 — Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in this code.

9.1.2 — Members also shall meet all other requirements of this code to ensure adequate performance at service load levels.

9.1.3 — Design of structures and structural members using the load factor combinations and strength reduction factors of Appendix C shall be permitted. Use of load factor combinations from this chapter in conjunction with strength reduction factors of Appendix C shall not be permitted.

9.2 — Required strength

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R9.1 — General

In the 2006 code, the load factor combinations and strength reduction factors of the 2001 code were revised and moved to Appendix C. The 2001 combinations have been replaced with those of ASCE 7-02.[9.1] The strength reduction factors were replaced with those of ACI 318-99 Appendix C, except that the factor for flexure was increased.

The changes were made to further unify the design profession on one set of load factors and combinations, and to facilitate the proportioning of concrete building structures that include members of materials other than concrete. When used with the strength reduction factors in 9.3, the designs for gravity loads will be comparable to those obtained using the strength reduction and load factors of the 2001 code. For combinations with lateral loads, some designs will be different, but the results of either set of load factors are considered acceptable.

Chapter 9 defines the basic strength and serviceability conditions for proportioning reinforced concrete members.

The basic requirement for strength design may be expressed as follows:

$$\text{design strength} \geq \text{required strength}$$

$$\phi\ (\text{nominal strength}) \geq \boldsymbol{U}$$

In the strength design procedure, the margin of safety is provided by multiplying the service load by a load factor and the nominal strength by a strength reduction factor.

R9.2 — Required strength

The required strength U is expressed in terms of factored loads, or related internal moments and forces. Factored loads are the loads specified in the general building code multiplied by appropriate load factors.

The factor assigned to each load is influenced by the degree of accuracy to which the load effect usually can be calculated

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and the variation that might be expected in the load during the lifetime of the structure. Dead loads, because they are more accurately determined and less variable, are assigned a lower load factor than live loads. Load factors also account for variability in the structural analysis used to compute moments and shears.

The code gives load factors for specific combinations of loads. In assigning factors to combinations of loading, some consideration is given to the probability of simultaneous occurrence. While most of the usual combinations of loadings are included, the designer should not assume that all cases are covered.

Due regard is to be given to sign in determining U for combinations of loadings, as one type of loading may produce effects of opposite sense to that produced by another type. The load combinations with $0.9D$ are specifically included for the case where a higher dead load reduces the effects of other loads. The loading case may also be critical for tension-controlled column sections. In such a case, a reduction in axial load and an increase in moment may result in a critical load combination.

Consideration should be given to various combinations of loading to determine the most critical design condition. This is particularly true when strength is dependent on more than one load effect, such as strength for combined flexure and axial load or shear strength in members with axial load.

If special circumstances require greater reliance on the strength of particular members than encountered in usual practice, some reduction in the stipulated strength reduction factor ϕ or increase in the stipulated load factors U may be appropriate for such members.

The wind load equation in ASCE 7-02[9.1] and IBC 2000[9.2] includes a factor for wind directionality that is equal to 0.85 for buildings. The corresponding load factor for wind in the load combination equations was increased accordingly (1.3/0.85 = 1.53 rounded up to 1.6). The code allows use of the previous wind load factor of 1.3 when the design wind load is obtained from other sources that do not include the wind directionality factor.

Model building codes and design load references have converted earthquake forces to strength level, and reduced the earthquake load factor to 1.0 (ASCE 7-93;[9.3] BOCA/NBC 93;[9.4] SBC 94;[9.5] UBC 97;[9.6] and IBC 2000[9.2]). The code requires use of the previous load factor for earthquake loads, approximately 1.4, when service-level earthquake forces from earlier editions of these references are used.

9.2.1 — Required strength U shall be at least equal to the effects of factored loads in Eq. (9-1) through (9-7). The effect of one or more loads not acting simultaneously shall be investigated.

R9.2.1 — Due to the significant uncertainty in determining soil pressures, it is conservative to disregard earth pressures where they reduce the effects of other loads. It may be appropriate, however, for some loading conditions to consider forces due to earth pressures as opposing other applied forces. When doing so, a reduced load factor should

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$$U = 1.4(D + F) \quad (9\text{-}1)$$

$$U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \quad (9\text{-}2)$$

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W) \quad (9\text{-}3)$$

$$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \quad (9\text{-}4)$$

$$U = 1.2D + 1.2F + 1.0E + 1.6H + 1.0L + 0.2S \quad (9\text{-}5)$$

$$U = 0.9D + 1.2F + 1.6W + 1.6H \quad (9\text{-}6)$$

$$U = 0.9D + 1.2F + 1.0E + 1.6H \quad (9\text{-}7)$$

except as follows:

(a) The load factor on ***L*** in Eq. (9-3) to (9-5) shall be permitted to be reduced to 0.5 where it can be justified that no greater than 50 percent of the design live load is expected to be present during normal operating conditions. Reduction of the load factor shall not be permitted in areas occupied as places of public assembly, and areas where the live load ***L*** is greater than 100 lb/ft^2;

(b) Where wind load ***W*** has not been reduced by a directionality factor, it shall be permitted to use **1.3*W*** in place of **1.6*W*** in Eq. (9-4) and (9-6);

(c) Where earthquake load ***E*** is based on service-level seismic forces, **1.4*E*** shall be used in place of **1.0*E*** in Eq. (9-5) and (9-7);

(d) The load factor on ***H*** shall be reduced to 0.6 where ***H*** reduces the effect of ***D***, ***L***, or ***F***. Earth pressure shall be permitted to be used to reduce other load effects only if investigation and analysis shows that structure movement and soil characteristics are appropriate to develop that pressure;

(e) Both the full value and the zero value of ***L*** and ***F*** shall be used in the above load combinations to determine the most severe condition.

9.2.2 — If resistance to impact effects is taken into account in design, such effects shall be included with live load ***L***.

9.2.3 — Estimations of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change shall be based on a realistic assessment of such effects occurring in service.

9.2.4 — For a structure in a flood zone, the flood load and load combinations of ASCE 7 shall be used.

9.2.5 — For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum prestressing steel jacking force.

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be applied to ***H*** as noted, and the magnitude of earth pressure used should be developed conservatively by a geotechnical engineer.

Both ***L*** and ***F*** are considered to be transient loads, so designs must consider the effects for such loads being present or absent.

R9.2.2 — If the live load is applied rapidly, as may be the case for vehicle loads, cranes, etc., impact effects should be considered. In all equations, substitute (***L*** + impact) for ***L*** when impact should be considered.

R9.2.3 — The designer should consider the effects of differential settlement, creep, shrinkage, temperature, and shrinkage-compensating concrete. The term realistic assessment is used to indicate that the most probable values rather than the upper bound values of the variables should be used.

R9.2.4 — Areas subject to flooding are defined by flood hazard maps, usually maintained by local governmental jurisdictions.

R9.2.5 — The load factor of 1.2 applied to the maximum tendon jacking force results in a design load of about 113 percent of the specified prestressing steel yield strength, but

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not more than 96 percent of the nominal ultimate strength of the prestressing steel. This compares well with the maximum attainable jacking force, which is limited by the anchor efficiency factor.

9.2.6 — Required strength ***U*** for other than compression-controlled sections, as defined in 10.3.3, shall be multiplied by the following environmental durability factor (S_d) in portions of an environmental engineering concrete structure where durability, liquid-tightness, or similar serviceability are considerations. In the case of shear design, this factor is applied to the excess shear strength carried by shear reinforcement only. This durability factor shall not be used for designs using service loads and permissible service load stresses. For applicable use of the environmental durability factor (S_d) in conjunction with load combinations that include earthquake loads, see Section 21.2.1.8.

$$S_d = \frac{\phi f_y}{\gamma f_s} \geq 1.0 \qquad (9\text{-}8)$$

where $\gamma = \dfrac{\text{factored load}}{\text{unfactored load}}$

and where f_s is the permissible tensile stress in reinforcement as given below:

R9.2.6 — In environmental engineering concrete structures, durability and long-term service life are paramount. The resulting stresses in nonprestressed reinforcement using normal building code load factors are higher than would be desirable in environmental engineering concrete structures. The intent of the environmental durability factor is to reduce the effective stress in nonprestressed reinforcement under service load conditions, such that stress levels are considered to be in an acceptable range for control of cracking. The environmental durability factor in Eq. (9-8) will vary with individual load combinations and with applicable ϕ factors (for example, flexure versus shear). As a conservative simplification, the ϕ factor may be taken as the maximum ϕ factor (0.90) in Eq. (9-8).

The limitation of $S_d \geq 1.0$ is to ensure that the strength requirements of 318 are always met as a minimum regardless of crack control considerations. This limitation will likely control where bars of relatively low yield strength are used.

In effect, for tension-controlled sections and shear strength contributed by reinforcement, Eq. (9-8) eliminates the effects of code-prescribed load factors and ϕ factors and applies an effective load factor equal to f_y/f_s with ϕ factors set to 1.0. Thus, where the environmental durability factor is applicable in these types of sections, the following design procedure will achieve the same results:

1. Multiply the unfactored loads by a uniform load factor equal to f_y/f_s ($\geq$1.0);

2. Use a value of 1.0 for applicable design ϕ factors.

The normal load factors would still be applicable to some design conditions, such as shear strength from concrete and compression-controlled members.

9.2.6.1 — Flexural stress: See 10.6.4.

R9.2.6.1 — Required flexural strength $\geq S_d U$.

9.2.6.2 — Direct and hoop tensile stress in normal environmental exposures

$$f_s = 20{,}000 \text{ psi}$$

9.2.6.3 — Direct and hoop tensile stress in severe environmental exposures

$$f_s = 17{,}000 \text{ psi}$$

R9.2.6.2 and R9.2.6.3 — Required strength in direct and hoop tension $\geq S_d U$.

Some designers prefer to use a maximum steel stress equal to 14,000 psi for hoop tension. This practice is based on an earlier version of the PCA publication, "Circular Concrete Tanks without Prestressing."[9.7]

9.2.6.4 — Shear stress carried by shear reinforcement in normal environmental exposures

$$f_s = 24{,}000 \text{ psi}$$

9.2.6.5 — Shear stress carried by shear reinforcement in severe environmental exposures

$$f_s = 20{,}000 \text{ psi}$$

R9.2.6.4 and R9.2.6.5 — Shear stress carried by the shear reinforcing is defined as the excess shear strength required in addition to the design shear strength provided by the concrete ϕV_c

$$\phi V_s \geq S_d (V_u - \phi V_c)$$

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9.2.7 — S_d shall be taken as 1.0 for the flexural design of compression-controlled sections, all prestressed reinforcement, and post-tensioned anchorage zone reinforcement, regardless of exposure.

COMMENTARY

R9.2.7 — The environmental durability factor is taken equal to 1.0 for compression controlled sections because by definition, their steel strains are less than or equal to.002 per 10.3.3, and therefore have low tensile steel stress levels and minimal concern for cracking.

9.3 — Design strength

9.3.1 — Design strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this code, multiplied by the strength reduction factors ϕ in 9.3.2 and 9.3.4.

R9.3—Design strength

R9.3.1 — The design strength of a member refers to the nominal strength calculated in accordance with the requirements stipulated in this code multiplied by a strength reduction factor ϕ, which is always less than one.

The purposes of the strength reduction factor ϕ are: (1) to allow for the probability of understrength members due to variations in material strengths and dimensions; (2) to allow for inaccuracies in the design equations; (3) to reflect the degree of ductility and required reliability of the member under the load effects being considered; and (4) to reflect the importance of the member in the structure.[9.8,9.9]

In the ACI 318-02 code, the strength reduction factors were adjusted to be compatible with the ASCE 7-98[9.10] load combinations, which were the basis for the required factored load combinations in model building codes at that time. These factors are essentially the same as those published in Appendix C of the ACI 318-95, except the factor for flexure/tension controlled limits is increased from 0.80 to 0.90. This change is based on past[9.8] and current reliability analyses,[9.11] statistical study of material properties, as well as the opinion of the committee that the historical performance of concrete structures supports ϕ = 0.90.

9.3.2 — Strength reduction factor ϕ shall be as follows:

9.3.2.1 — Tension-controlled sections as defined in 10.3.4..0.90

9.3.2.2 — Compression-controlled sections, as defined in 10.3.3:

(a) Members with spiral reinforcement conforming to 10.9.3... 0.70

(b) Other reinforced members.............................0.65

For sections in which the net tensile strain in the extreme tension steel at nominal strength is between the limits for compression-controlled and tension-controlled sections, ϕ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as the net tensile strain in the extreme tension steel at nominal strength increases from the compression-controlled strain limit to 0.005.

Alternatively, when Appendix B is used, for members in which f'_y does not exceed 60,000 psi, with symmetric reinforcement, and with $(h - d' - d_s)/h$ not

R9.3.2.1 — In applying 9.3.2.1 and 9.3.2.2, the axial tensions and compressions to be considered are those caused by external forces. Effects of prestressing forces are not included.

R9.3.2.2 — Before the 2006 edition, the code specified the magnitude of the ϕ-factor for cases of axial load or flexure, or both, in terms of the type of loading. For these cases, the ϕ-factor is now determined by the strain conditions at a cross section, at nominal strength.

A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections. Members with spiral reinforcement are assigned a higher ϕ than tied columns because they have greater ductility or toughness.

For sections subjected to axial load with flexure, design strengths are determined by multiplying both P_n and M_n by the appropriate single value of ϕ. Compression-controlled and tension-controlled sections are defined in 10.3.3 and

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less than 0.70, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10f'_c A_g$ to zero. For other reinforced members, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10f'_c A_g$ or ϕP_b whichever is smaller, to zero.

9.3.2.3 — Shear and torsion..............................0.75

9.3.2.4 — Bearing on concrete (except for post-tensioned anchorage zones).................................0.65

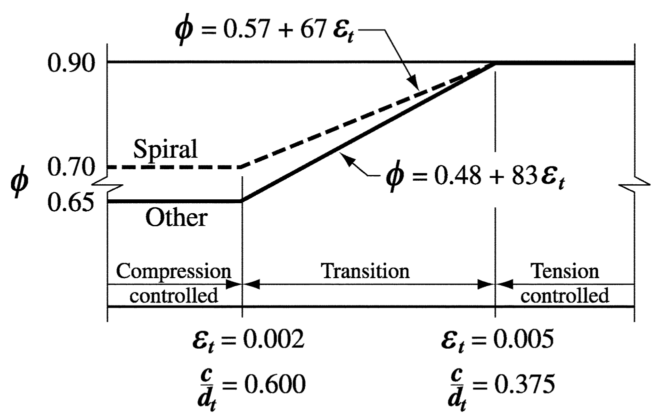
9.3.2.5 — Post-tensioned anchorage zones 0.85

COMMENTARY

10.3.4 as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than 0.005, respectively. For sections with net tensile strain ε_t in the extreme tension steel at nominal strength between the above limits, the value of ϕ may be determined by linear interpolation, as shown in Fig. R9.3.2. The concept of net tensile strain ε_t is discussed in R10.3.3.

Because the compressive strain in the concrete at nominal strength is assumed in 10.2.3 to be 0.003, the net tensile strain limits for compression-controlled members may also be stated in terms of the ratio c/d_t, where c is the depth of the neutral axis at nominal strength, and d_t is the distance from the extreme compression fiber to the extreme tension steel. The c/d_t limits for compression-controlled and tension-controlled sections are 0.6 and 0.375, respectively. The 0.6 limit applies to sections reinforced with Grade 60 steel and to prestressed sections. Figure R9.3.2 also gives equations for ϕ as a function of c/d_t.

The net tensile strain limit for tension-controlled sections may also be stated in terms of the ρ/ρ_b as defined in the 2002 edition of the 318 code. The net tensile strain limit of 0.005 corresponds to a ρ/ρ_b ratio of 0.63 for rectangular sections with Grade 60 reinforcement. For a comparison of these provisions with the ACI 318-02 code Section 9.3, see Reference 9.12.



Interpolation on c/d_t: Spiral $\phi = 0.37 + 0.20/(c/d_t)$
Other $\phi = 0.23 + 0.25/(c/d_t)$

Fig. R9.3.2—Variation of ϕ with net tensile ε_t and c/d_t for Grade 60 reinforcement and for prestressing steel.

R9.3.2.5 — The ϕ-factor of 0.85 reflects the wide scatter of results of experimental anchorage zone studies. Because 18.13.4.2 limits the nominal compressive strength of unconfined concrete in the general zone to $0.7\lambda f'_{ci}$, the effective design strength for unconfined concrete is $0.85 \times 0.7\lambda f'_{ci} \approx 0.6\lambda f'_{ci}$.

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9.3.2.6 — Flexure sections without axial load in pretensioned members where strand embedment is less than the development length as provided in 12.9.1.1...0.75

9.3.3 — Development lengths specified in Chapter 12 do not require a ϕ-factor.

9.3.4 — In structures that rely on special moment resisting frames or special reinforced concrete structural walls to resist earthquake effects, the strength reduction factors ϕ shall be modified as given in (a) through (c):

(a) The strength reduction factor for shear shall be 0.60 for any structural member that is designed to resist earthquake effects if its nominal shear strength is less than the shear corresponding to the development of the nominal flexural strength of the member. The nominal flexural strength shall be determined considering the most critical factored axial loads and including earthquake effects;

(b) The strength reduction factor for shear in diaphragms shall not exceed the minimum strength reduction factor for shear used for the vertical components of the primary lateral-force-resisting system;

(c) The strength reduction factor for shear in joints and diagonally reinforced coupling beams shall be 0.85.

9.4 — Design strength for reinforcement

Designs shall not be based on a yield strength of reinforcement f_y in excess of 80,000 psi, except for prestressing steel.

9.5 — Control of deflections

9.5.1 — Reinforced concrete members subjected to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that adversely affect strength or serviceability of a structure.

COMMENTARY

R9.3.2.6 — If a critical section occurs in a region where strand is not fully developed, failure may be by bond slip. Such a failure resembles a brittle shear failure; hence, the requirements for a reduced ϕ.

R9.3.4 — Strength reduction factors in 9.3.4 are intended to compensate for uncertainties in estimation of strength of structural members. They are based primarily on experience with constant or steadily increasing applied load. For construction in regions of high seismic risk, some of the strength reduction factors have been modified in 9.3.4 to account for the effects on strength of displacements into the nonlinear range of response.

Section 9.3.4(a) refers to brittle members such as low-rise walls, portions of walls between openings, or diaphragms that are impractical to reinforce to raise their nominal shear strength above nominal flexural strength for the pertinent loading conditions.

Short structural walls were the primary vertical elements of the lateral-force-resisting system in many of the parking structures that sustained damage during the 1994 Northridge earthquake. Section 9.3.4(b) requires the shear strength reduction factor for diaphragms to be 0.60 if the shear strength reduction factor for the walls is 0.60

R9.4 — Design strength for reinforcement

Reinforcing bars with a yield strength of 75,000 psi in sizes No. 11, 14, and 18 and yield measured at a strain of 0.0035 and so meeting the requirements of this code were first included in ASTM A 615-87.

In addition to the upper limit of 80,000 psi for yield strength of nonprestressed reinforcement, there are limitations on yield strength in other sections of the code:

In 11.5.2, 11.6.3.4, and 11.7.6: the maximum f_y that may be used in design for shear and torsion reinforcement is 60,000 psi, except that f_y up to 80,000 psi may be used for shear reinforcement meeting the requirements of ASTM A 497.

In 19.3.2 and 21.2.5: the maximum specified f_y is 60,000 psi in shells, folded plates, and structures governed by the special seismic provisions of Chapter 21.

The deflection provisions of 9.5 and the limitations on distribution of flexural reinforcement of 10.6 become increasingly critical as f_y increases.

R9.5 — Control of deflections

R9.5.1 — The provisions of 9.5 are concerned only with deflections or deformations that may occur at service load levels. Where long-term deflections are computed, only the

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dead load and that portion of the live load that is sustained need be considered.

Two methods are given for controlling deflections.[9.13] For nonprestressed beams and one-way slabs, and for composite members, provision of a minimum overall thickness as required by Table 9.5(a) will satisfy the requirements of the code for members not supporting or attached to partitions or other construction likely to be damaged by large deflections. For nonprestressed two-way construction, minimum thickness as required by 9.5.3.1, 9.5.3.2, and 9.5.3.3 will satisfy the requirements of the code.

For nonprestressed members that do not meet these minimum thickness requirements, or that support or are attached to partitions or other construction likely to be damaged by large deflections, and for all prestressed concrete flexural members, deflections must be calculated by the procedures described or referred to in the appropriate sections of the code, and are limited to the values in Table 9.5(b).

9.5.2 — One-way construction (nonprestressed)

9.5.2.1 — Minimum thickness stipulated in Table 9.5(a) shall apply for one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation of deflection indicates a lesser thickness can be used without adverse effects.

TABLE 9.5(a)—MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED

	Minimum thickness h			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs	**ℓ/20**	**ℓ/24**	**ℓ/28**	**ℓ/10**
Beams or ribbed one-way slabs	**ℓ/16**	**ℓ/18.5**	**ℓ/21**	**ℓ/8**

Notes:
1) Span length ℓ is in inches.
2) Values given shall be used directly for members with normalweight concrete (w_c = 145 lb/ft^3) and Grade 60 reinforcement. For other conditions, the values shall be modified as follows:
a) For structural lightweight concrete having unit weight in the range of 90 to 120 lb/ft^3, the values shall be multiplied by **($1.65 - 0.005w_c$)** but not less than 1.09, where w_c is the unit weight in lb/ft^3;
b) For f_y other than 60,000 psi, the values shall be multiplied by (**$0.4 + f_y/100{,}000$**).

9.5.2.2 — Where deflections are to be computed, deflections that occur immediately on application of load shall be computed by usual methods or formulas for elastic deflections, considering effects of cracking and reinforcement on member stiffness.

R9.5.2 — One-way construction (nonprestressed)

R9.5.2.1 — The minimum thicknesses of Table 9.5(a) apply for nonprestressed beams and one-way slabs (see 9.5.2), and for composite members (see 9.5.5).

It should be emphasized that these minimum thicknesses apply only to members not supporting or attached to partitions and other construction likely to be damaged by deflection.

Values of minimum thickness must be modified if other than normalweight concrete and Grade 60 reinforcement are used. The notes beneath the table are essential to its use for reinforced concrete members constructed with structural lightweight concrete and/or with reinforcement having a yield strength other than 60,000 psi. If both of these conditions exist, the corrections in footnotes (a) and (b) should both be applied.

The modification for lightweight concrete in footnote (a) is based on studies of the results and discussions in Reference 9.14. No correction is specified for concretes weighing between 120 and 145 lb/ft^3 because the correction term would be close to unity in this range.

The modification for yield strength in footnote (b) is approximate, but should yield conservative results for the type of members considered in the table, for typical reinforcement ratios, and for values of f_y between 40,000 and 80,000 psi.

R9.5.2.2 — For calculation of immediate deflections of uncracked prismatic members, the usual methods or formulas for elastic deflections may be used with a constant value of E_cI_g along the length of the member. However, if the member is cracked at one or more sections, or if its depth varies along the span, a more exact calculation becomes necessary.

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9.5.2.3 — Unless stiffness values are obtained by a more comprehensive analysis, immediate deflection shall be computed with the modulus of elasticity E_c for concrete as specified in 8.5.1 (normalweight or lightweight concrete) and with the effective moment of inertia as follows, but not greater than I_g

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \tag{9-9}$$

where

$$M_{cr} = \frac{f_r I_g}{y_t} \tag{9-10}$$

and for normalweight concrete

$$f_r = 7.5\sqrt{f'_c} \tag{9-11}$$

When lightweight aggregate concrete is used, one of the following modifications shall apply:

(a) When f_{ct} is specified and concrete is proportioned in accordance with 5.2, f_r shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f'_c}$, but the value of $f_{ct}/6.7$ shall not exceed $\sqrt{f'_c}$;

(b) When f_{ct} is not specified, f_r shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation shall be permitted if partial sand replacement is used.

9.5.2.4 — For continuous members, effective moment of inertia shall be permitted to be taken as the average of values obtained from Eq. (9-9) for the critical positive and negative moment sections. For prismatic members, effective moment of inertia shall be permitted to be taken as the value obtained from Eq. (9-9) at midspan for simple and continuous spans, and at support for cantilevers.

9.5.2.5 — Unless values are obtained by a more comprehensive analysis, additional long-term deflection resulting from creep and shrinkage of flexural members (normalweight or lightweight concrete) shall be determined by multiplying the immediate deflection caused by the sustained load considered, by the factor

$$\lambda = \frac{\xi}{1 + 50\rho'} \tag{9-12}$$

where ρ' shall be the value at midspan for simple and continuous spans, and at support for cantilevers. It shall be permitted to assume the time-dependent factor ξ for sustained loads to be equal to

5 years or more..2.0

12 months...1.4

COMMENTARY

R9.5.2.3 — The effective moment of inertia procedure described in the code and developed in Reference 9.15 was selected as being sufficiently accurate for use to control deflections.[9.16-9.18] The effective I_e was developed to provide a transition between the upper and lower bounds of I_g and I_{cr} as a function of the ratio M_{cr}/M_a. For most practical cases, I_e will be less than I_g.

R9.5.2.4 — For continuous members, the code procedure suggests a simple averaging of I_e values for the positive and negative moment sections. The use of the midspan section properties for continuous prismatic members is considered satisfactory in approximate calculations primarily because the midspan rigidity (including the effect of cracking) has the dominant effect on deflections, as shown by ACI Committee 435[9.19,9.20] and SP-43.[9.13]

R9.5.2.5 — Shrinkage and creep due to sustained loads cause additional "long-term deflections" over and above those that occur when loads are first placed on the structure. Such deflections are influenced by temperature, humidity, curing conditions, age at time of loading, quantity of compression reinforcement, magnitude of the sustained load, and other factors. The expression given in this section is considered satisfactory for use with the code procedures for the calculation of immediate deflections, and with the limits given in Table 9.5(b). It should also be noted that the deflection computed in accordance with this section is the additional long-term deflection due to the dead load and that portion of the live load that will be sustained for a sufficient period to cause significant time-dependent deflections.

Equation (9-12) was developed in Reference 9.21. In Eq. (9-12), the multiplier on ξ accounts for the effect of compression reinforcement in reducing long-term deflections, and ξ = 2.0

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TABLE 9.5(b)—MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	$\ell/180$*
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	$\ell/360$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\ell/480$‡
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\ell/240$§

*Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

†Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.2, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

‡Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

§Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

6 months...1.2

3 months...1.0

9.5.2.6 — Deflection computed in accordance with 9.5.2.2 through 9.5.2.5 shall not exceed limits stipulated in Table 9.5(b).

represents a nominal time-dependent factor for 5 years duration of loading. The curve in Fig. R9.5.2.5 may be used to estimate values of ξ for loading periods less than 5 years.

If it is desired to consider creep and shrinkage separately, approximate equations provided in References 9.15, 9.16, 9.21, and 9.22 may be used.

R9.5.2.6 — It should be noted that the limitations given in this table relate only to supported or attached nonstructural elements. For those structures in which structural members are likely to be affected by deflection or deformation of members to which they are attached in such a manner as to affect adversely the strength of the structure, these deflections and the resulting forces should be considered explicitly in the analysis and design of the structures as required by 9.5.1. (See Reference 9.18.)

Where long-term deflections are computed, the portion of the deflection before attachment of the nonstructural elements may be deducted. In making this correction use may be made of the curve in Fig. R9.5.2.5 for members of usual sizes and shapes.

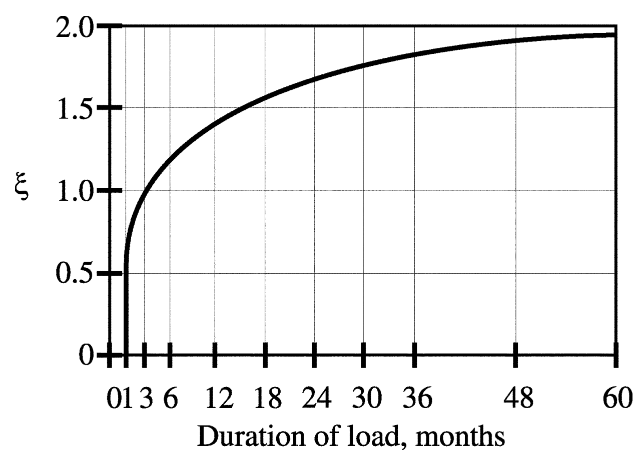

Fig. R9.5.2.5 — Multipliers for long-term deflections.

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9.5.3 — Two-way construction (nonprestressed)

9.5.3.1—Section 9.5.3 shall govern the minimum thickness of slabs or other two-way construction designed in accordance with the provisions of Chapter 13 and conforming with the requirements of 13.6.1.2. The thickness of slabs without interior beams spanning between the supports on all sides shall satisfy the requirements of 9.5.3.2 or 9.5.3.4. The thickness of slabs with beams spanning between the supports on all sides shall satisfy requirements of 9.5.3.3 or 9.5.3.4.

9.5.3.2 — For slabs without interior beams spanning between the supports and having a ratio of long to short span not greater than 2, the minimum thickness shall be in accordance with the provisions of Table 9.5(c) and shall not be less than the following values:

(a) Slabs without drop panels as defined in 13.3.7.1 and 13.3.7.2..5 in.

(b) Slabs with drop panels as defined in 13.3.7.1 and 13.3.7.2..4 in.

TABLE 9.5(c)—MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS

	Without drop panels†			With drop panels†		
	Exterior panels		Interior panels	Exterior panels		Interior panels
Yield strength f_y, psi*	Without edge beams	With edge beams‡		Without edge beams	With edge beams‡	
40,000	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{40}$	$\frac{\ell_n}{40}$
60,000	$\frac{\ell_n}{30}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$
75,000	$\frac{\ell_n}{28}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{34}$	$\frac{\ell_n}{34}$

* For values of reinforcement yield strength between the values given in the table, minimum thickness shall be determined by linear interpolation.

† Drop panel is defined in 13.3.7.1 and 13.3.7.2.

‡ Slabs with beams between columns along exterior edges. The value of α for the edge beam shall not be less than 0.8.

9.5.3.3 — For slabs with beams spanning between the supports on all sides, the minimum thickness shall be as follows:

(a) For α_m equal to or less than 0.2, the provisions of 9.5.3.2 shall apply;

(b) For α_m greater than 0.2 but not greater than 2.0, the thickness shall not be less than

$$h = \frac{\ell_n\left(0.8 + \frac{f_y}{200{,}000}\right)}{36 + 5\beta(\alpha_m - 0.2)} \qquad (9\text{-}13)$$

and not less than 5 in.;

(c) For α_m greater than 2.0, the thickness shall not be less than

COMMENTARY

R9.5.3 — Two-way construction (nonprestressed)

R9.5.3.2 — The minimum thicknesses in Table 9.5(c) are those that have evolved through the years in building codes. It is assumed that slabs conforming to those limits have not resulted in systematic problems related to stiffness for short- and long-term loads. Naturally, this conclusion applies in only the domain of previous experience in loads, environment, materials, boundary conditions, and spans.

R9.5.3.3 — For panels having a ratio of long to short span greater than 2, the use of Eq. (9-13) and (9-14), which express the minimum thickness as a fraction of the long span, may give unreasonable results. For such panels, the rules applying to one-way construction in 9.5.2 should be used.

The requirement in 9.5.3.3(a) for α_m equal to 0.2 makes it possible to eliminate Eq. (9-13) of ACI 318-89. That equation gave values essentially the same as those in Table 9.5(c), as does Eq. (9-13) at a value of α_m equal to 0.2.

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$$h = \frac{\ell_n\left(0.8 + \frac{f_y}{200{,}000}\right)}{36 + 9\beta} \qquad (9\text{-}14)$$

and not less than 3.5 in.;

(d) At discontinuous edges, an edge beam shall be provided with a stiffness ratio α not less than 0.80 or the minimum thickness required by Eq. (9-13) or (9-14) shall be increased by at least 10 percent in the panel with a discontinuous edge.

9.5.3.4 — Slab thickness less than the minimum thickness required by 9.5.3.1, 9.5.3.2, and 9.5.3.3 shall be permitted to be used if shown by computation that the deflection will not exceed the limits stipulated in Table 9.5(b). Deflections shall be computed taking into account size and shape of the panel, conditions of support, and nature of restraints at the panel edges. The modulus of elasticity of concrete E_c shall be as specified in 8.5.1.The effective moment of inertia shall be that given by Eq. (9-9); other values shall be permitted to be used if they result in computed deflections in reasonable agreement with results of comprehensive tests. Additional long-term deflection shall be computed in accordance with 9.5.2.5.

9.5.4 — Prestressed concrete construction

9.5.4.1 — For flexural members designed in accordance with provisions of Chapter 18, immediate deflection shall be computed by usual methods or formulas for elastic deflections, and the moment of inertia of the gross concrete section shall be permitted to be used for Class U flexural members, as defined in 18.3.3.

9.5.4.2 — For Class T flexural members, as defined in 18.3.3, deflection calculations shall be based on a cracked transformed section analysis. It shall be permitted to base computations on a bilinear moment-deflection relationship, or an effective moment of inertia as defined by Eq. (9-8).

9.5.4.3 — Additional long-term deflection of prestressed concrete members shall be computed taking into account stresses in concrete and steel under sustained load and including effects of creep and shrinkage of concrete and relaxation of steel.

COMMENTARY

R9.5.3.4 — The calculation of deflections for slabs is complicated even if linear elastic behavior can be assumed. For immediate deflections, the values of E_c and I_e specified in 9.5.2.3 may be used.[9.18] Other procedures and other values of the stiffness EI may be used, however, if they result in predictions of deflection in reasonable agreement with the results of comprehensive tests.

Because available data on long-term deflections of slabs are too limited to justify more elaborate procedures, the additional long-term deflection for two-way construction is required to be computed using the multipliers given in 9.5.2.5.

R9.5.4 — Prestressed concrete construction

The code requires deflections for all prestressed concrete flexural members to be computed and compared with the allowable values in Table 9.5(b).

R9.5.4.1 — Immediate deflections of Class U prestressed concrete members may be calculated by the usual methods or formulas for elastic deflections using the moment of inertia of the gross (uncracked) concrete section and the modulus of elasticity for concrete specified in 8.5.1.

R9.5.4.2 — Class C and Class T prestressed flexural members are defined in 18.3.3. Reference 9.23 gives information on deflection calculations using a bilinear moment-deflection relationship and using an effective moment of inertia. Reference 9.24 gives additional information on deflection of cracked prestressed concrete members.

Reference 9.25 shows that the I_e method can be used to compute deflections of Class T prestressed members loaded above the cracking load. For this case, the cracking moment should take into account the effect of prestress. A method for predicting the effect of nonprestressed tension steel in reducing creep camber is also given in Reference 9.25, with approximate forms given in References 9.18 and 9.26.

R9.5.4.3 — Calculation of long-term deflections of prestressed concrete flexural members is complicated. The calculations must consider not only the increased deflections due to flexural stresses, but also the additional long-term deflections resulting from time-dependent

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9.5.4.4 — Deflection computed in accordance with 9.5.4.1 or 9.5.4.2 and 9.5.4.3 shall not exceed limits stipulated in Table 9.5(b).

9.5.5 — Composite construction

9.5.5.1 — Shored construction

If composite flexural members are supported during construction so that, after removal of temporary supports, dead load is resisted by the full composite section, it shall be permitted to consider the composite member equivalent to a monolithically cast member for computation of deflection. For nonprestressed members, the portion of the member in compression shall determine whether values in Table 9.5(a) for normalweight or lightweight concrete shall apply. If deflection is computed, account shall be taken of curvatures resulting from differential shrinkage of precast and cast-in-place components, and of axial creep effects in a prestressed concrete member.

9.5.5.2 — Unshored construction

If the thickness of a nonprestressed precast flexural member meets the requirements of Table 9.5(a),

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shortening of the flexural member.

Prestressed concrete generally shortens more with time than similar nonprestressed members. This is due to the precompression in the slab or beam that causes axial creep. This creep, together with shrinkage of the concrete, results in significant shortening of the flexural members that continues for several years after construction and must be considered in design. The shortening tends to reduce the tension in the prestressing steel, thus reducing the precompression in the member and thereby causing increased long-term deflections.

Another factor that can influence long-term deflections of prestressed flexural members is adjacent concrete or masonry nonprestressed in the direction of the prestressed member. This can be a slab nonprestressed in the beam direction adjacent to a prestressed beam or a nonprestressed slab system. As the prestressed member tends to shrink and creep more than the adjacent nonprestressed concrete, the structure will tend to reach a compatibility of the shortening effects. This results in a reduction of the precompression in the prestressed member as the adjacent concrete absorbs the compression. This reduction in precompression of the prestressed member can occur over a period of years and will result in additional long-term deflections and in increased stresses in the prestressed member.

Any suitable method for calculating long-term deflections of prestressed members may be used, provided all effects are considered. Guidance may be found in References 9.18, 9.27, 9.28, and 9.29.

R9.5.5 — Composite construction

Because few tests have been made to study the immediate and long-term deflections of composite members, the rules given in 9.5.5.1 and 9.5.5.2 are based on the judgment of ACI Committee 318 and on experience.

If any portion of a composite member is prestressed or if the member is prestressed after the components have been cast, the provisions of 9.5.4 apply, and deflections must be calculated. For nonprestressed composite members, deflections need to be calculated and compared with the limiting values in Table 9.5(b) only when the thickness of the member or the precast part of the member is less than the minimum thickness given in Table 9.5(a). In unshored construction, the thickness of concern depends on whether the deflection before or after the attainment of effective composite action is being considered. (In Chapter 17, it is stated that distinction need not be made between shored and unshored members. This refers to strength calculations, not to deflections.)

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deflection need not be computed. If the thickness of a nonprestressed composite member meets the requirements of Table 9.5(a), it is not required to compute deflection occurring after the member becomes composite, but the long-term deflection of the precast member shall be investigated for magnitude and duration of load prior to beginning of effective composite action.

9.5.5.3 — Deflection computed in accordance with 9.5.5.1 or 9.5.5.2 shall not exceed limits stipulated in Table 9.5(b).

CHAPTER 10 — FLEXURE AND AXIAL LOADS

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10.0 — Notation

a = depth of equivalent rectangular stress block as defined in 10.2.7.1, in.

A_b = area of an individual horizontal bar or wire, in.^{2}

A_c = area of core of spirally reinforced compression member measured to outside diameter of spiral, in.^{2}

A_g = gross area of section, in.^{2}

A_s = area of nonprestressed tension reinforcement, in.^{2}

$A_{s,min}$ = minimum amount of flexural reinforcement, in.^{2} See 10.5

A_{st} = total area of longitudinal reinforcement, (bars or steel shapes), in.^{2}

A_t = area of structural steel shape, pipe, or tubing in a composite section, in.^{2}

A_1 = loaded area, in.^{2}

A_2 = the area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal, in.^{2}

b = width of compression face of member, in.

b_w = web width, in.

c = distance from extreme compression fiber to neutral axis, in.

c_c = clear cover from the nearest surface in tension to the surface of the flexural tension reinforcement, in

C_m = a factor relating actual moment diagram to an equivalent uniform moment diagram

d = distance from extreme compression fiber to centroid of tension reinforcement, in.

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto, in.

d_b = nominal diameter of bar, wire, or prestressing strand, in.

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R10.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip.

The definition of net tensile strain in 2.1 excludes strains due to effective prestress, creep, shrinkage, and temperature.

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d_t = distance from extreme compression fiber to extreme tension steel, in.

E_c = modulus of elasticity of concrete, psi. See 8.5.1

E_s = modulus of elasticity of reinforcement, psi. See 8.5.2 or 8.5.3

EI = flexural stiffness of compression member. See Eq. (10-13) and Eq. (10-14), in.^{2}-lb

f'_c = specified compressive strength of concrete, psi

f_s = calculated stress in reinforcement at service loads, ksi

$f_{s\,max}$ = maximum allowable stress in reinforcement at service load, ksi

f_y = specified yield strength of nonprestressed reinforcement, psi

h = overall thickness of member, in.

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.^{4}

I_{se} = moment of inertia of reinforcement about centroidal axis of member cross section, in.^{4}

I_t = moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross section, in.^{4}

k = effective length factor for compression members

ℓ_c = length of compression member in a frame, measured from center to center of the joints in the frame, in.

ℓ_u = unsupported length of compression member, in.

M_c = factored moment to be used for design of compression member, in.-lb

M_s = moment due to loads causing appreciable sway, in.-lb

M_u = factored moment at section, in.-lb

M_1 = smaller factored end moment on a compression member, positive if member is bent in single curvature, negative if bent in double curvature, in.-lb

M_{1ns} = factored end moment on a compression member at the end at which M_1 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, in.-lb

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M_{1s} = factored end moment on compression member at the end at which M_1 acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, in.-lb

M_2 = larger factored end moment on compression member, always positive, in.-lb

$M_{2,min}$ = minimum value of M_2, in.-lb

M_{2ns} = factored end moment on compression member at the end at which M_2 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, in.-lb

M_{2s} = factored end moment on compression member at the end at which M_2 acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, in.-lb

P_c = critical load. See Eq. (10-12), lb

P_n = nominal axial load strength at given eccentricity, lb

P_o = nominal axial load strength at zero eccentricity, lb

P_u = factored axial load at given eccentricity, lb $\leq \phi P_n$

Q = stability index for a story. See 10.11.4

r = radius of gyration of cross section of a compression member, in.

s = center-to-center spacing of deformed bars

s_{sk} = spacing of skin reinforcement, in.

V_u = factored horizontal shear in a story, lb

β = ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement

β_1 = factor defined in 10.2.7.3

β_d = (a) for nonsway frames, β_d is the ratio of the maximum factored axial dead load to the total factored axial load; (b) for sway frames, except as required in (c) β_d is the ratio of the maximum factored sustained shear within a story to the total factored shear in that story; and (c) for stability checks of sway frames carried out in accordance with 10.13.6, β_d is the ratio of the maximum factored sustained axial load to the total factored axial load

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δ_{ns} = moment magnification factor for frames braced against sidesway, to reflect effects of member curvature between ends of compression member

δ_s = moment magnification factor for frames not braced against sidesway, to reflect lateral drift resulting from lateral and gravity loads

Δ_o = relative lateral deflection between the top and bottom of a story due to V_u, computed using a first-order elastic frame analysis and stiffness values satisfying 10.11.1, in.

ε_t = net tensile strain in extreme tension steel at nominal strength

ρ = ratio of nonprestressed tension reinforcement = A_s/bd

ρ_b = reinforcement ratio producing balanced strain conditions. See 10.3.2

ρ_s = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member

ϕ = strength reduction factor. See 9.3

ϕ_K = stiffness reduction factor. See R10.12.3

10.1 — Scope

Provisions of Chapter 10 shall apply for design of members subject to flexure or axial loads or to combined flexure and axial loads.

10.2 — Design assumptions

10.2.1 — Strength design of members for flexure and axial loads shall be based on assumptions given in 10.2.2 through 10.2.7, and on satisfaction of applicable conditions of equilibrium and compatibility of strains.

10.2.2 — Strain in reinforcement and concrete shall be assumed directly proportional to the distance from the neutral axis, except, for deep beams with overall depth to clear span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a nonlinear distribution of strain shall be considered. See 10.7.

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R10.2 — Design assumptions

R10.2.1 — The strength of a member computed by the strength design method of the code requires that two basic conditions be satisfied: (1) static equilibrium; and (2) compatibility of strains. Equilibrium between the compressive and tensile forces acting on the cross section at nominal strength must be satisfied. Compatibility between the stress and strain for the concrete and the reinforcement at nominal strength conditions should also be established within the design assumptions allowed by 10.2.

R10.2.2 — Many tests have confirmed that the distribution of strain is essentially linear across a reinforced concrete cross section, even near ultimate strength.

The strain in both reinforcement and in concrete is assumed to be directly proportional to the distance from the neutral axis. This assumption is of primary importance in design for determining the strain and corresponding stress in the reinforcement.

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10.2.3 — Maximum usable strain at extreme concrete compression fiber shall be assumed equal to 0.003.

10.2.4 — Stress in reinforcement below specified yield strength f_y for grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.

10.2.5 — Tensile strength of concrete shall be neglected in axial and flexural calculations of reinforced concrete, except when meeting requirements of 18.4.

10.2.6 — The relationship between concrete compressive stress distribution and concrete strain shall be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

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R10.2.3 — The maximum concrete compressive strain at crushing of the concrete has been observed in tests of various kinds to vary from 0.003 to higher than 0.008 under special conditions. The strain at which ultimate moments are developed, however, is usually about 0.003 to 0.004 for members of normal proportions and materials.

R10.2.4 — For deformed reinforcement, it is reasonably accurate to assume that the stress in reinforcement is proportional to strain below the yield strength f_y. The increase in strength due to the effect of strain hardening of the reinforcement is neglected for strength computations. In strength computations, the force developed in tensile or compressive reinforcement is computed as,

when $\varepsilon_s < \varepsilon_y$ (yield strain)

$$A_s f_s = A_s E_s \varepsilon_s$$

when $\varepsilon_s \geq \varepsilon_y$

$$A_s f_s = A_s f_y$$

where ε_s is the value from the strain diagram at the location of the reinforcement. For design, the modulus of elasticity of steel reinforcement E_s may be taken as 29,000,000 psi (see 8.5.2).

R10.2.5 — The tensile strength of concrete in flexure (modulus of rupture) is a more variable property than the compressive strength and is about 10 to 15 percent of the compressive strength. Tensile strength of concrete in flexure is neglected in strength design. For members with normal percentages of reinforcement, this assumption is in good agreement with tests. For very small percentages of reinforcement, neglect of the tensile strength at ultimate is usually correct.

The strength of concrete in tension, however, is important in cracking and deflection considerations at service loads.

R10.2.6 — This assumption recognizes the inelastic stress distribution of concrete at high stress. As maximum stress is approached, the stress-strain relationship for concrete is not a straight line but some form of a curve (stress is not proportional to strain). The general shape of a stress-strain curve is primarily a function of concrete strength and consists of a rising curve from zero to a maximum at a compressive strain between 0.0015 and 0.002 followed by a descending curve to an ultimate strain (crushing of the concrete) from 0.003 to higher than 0.008. As discussed under R10.2.3, the code sets the maximum usable strain at 0.003 for design.

The actual distribution of concrete compressive stress is complex and usually not known explicitly. Research has shown that the important properties of the concrete stress distribution can be approximated closely using any one of several different assumptions as to the form of stress distribution. The code permits any particular stress distribution to be assumed in design if shown to result in predictions of

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10.2.7 — Requirements of 10.2.6 are satisfied by an equivalent rectangular concrete stress distribution defined by the following:

10.2.7.1 — Concrete stress of $0.85f_c'$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fiber of maximum compressive strain.

10.2.7.2—Distance c from fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

10.2.7.3—Factor β_1 shall be taken as 0.85 for concrete strengths f_c' up to and including 4000 psi. For strengths above 4000 psi, β_1 shall be reduced continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi, but β_1 shall not be taken less than 0.65.

10.3 — General principles and requirements

10.3.1 — Design of cross sections subject to flexure or axial loads or to combined flexure and axial loads shall be based on stress and strain compatibility using assumptions in 10.2.

10.3.2 — Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as concrete in compression reaches its assumed ultimate strain of 0.003.

10.3.3 — Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit at the time the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 60 reinforcement, and for all prestressed reinforcement, it shall be permitted to set the compression-controlled strain limit equal to 0.002.

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ultimate strength in reasonable agreement with the results of comprehensive tests. Many stress distributions have been proposed. The three most common are the parabola, trapezoid, and rectangle.

R10.2.7 — For practical design, the code allows the use of a rectangular compressive stress distribution (stress block) to replace the more exact concrete stress distributions. In the equivalent rectangular stress block, an average stress of $0.85f_c'$ is used with a rectangle of depth $a = \beta_1 c$. The β_1 of 0.85 for concrete with $f_c' \leq 4000$ psi and 0.05 less for each 1000 psi of f_c' in excess of 4000 was determined experimentally.

In the 1976 supplement to ACI 318-71, a lower limit of β_1 equal to 0.65 was adopted for concrete strengths greater than 8000 psi. Research data from tests with high-strength concretes[10.1,10.2] supported the equivalent rectangular stress block for concrete strengths exceeding 8000 psi, with a β_1 equal to 0.65. Use of the equivalent rectangular stress distribution specified in ACI 318-71, with no lower limit on β_1, resulted in inconsistent designs for high-strength concrete for members subject to combined flexure and axial load.

The equivalent rectangular stress distribution does not represent the actual stress distribution in the compression zone at ultimate, but does provide essentially the same results as those obtained in tests.[10.3]

R10.3 — General principles and requirements

R10.3.1 — Design strength equations for members subject to flexure or combined flexure and axial load are derived in the paper, "Rectangular Concrete Stress Distribution in Ultimate Strength Design."[10.3] and previous editions of this commentary also give the derivations of strength equations for cross sections other than rectangular.

R10.3.2 — A balanced strain condition exists at a cross section when the maximum strain at the extreme compression fiber just reaches 0.003 simultaneously with the first yield strain f_y/E_s in the tension reinforcement. The reinforcement ratio ρ_b, which produces balanced conditions under flexure, depends on the shape of the cross section and the location of the reinforcement.

R10.3.3 — The nominal flexural strength of a member is reached when the strain in the extreme compression fiber reaches the assumed strain limit 0.003. The net tensile strain ε_t is the tensile strain in the extreme tension steel at nominal strength, exclusive of strains due to prestress, creep, shrinkage, and temperature. The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength, shown in Fig. R10.3.3, using similar triangles.

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be

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expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, whereas compression members are usually compression-controlled. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections. Section 9.3.2 specifies the appropriate strength reduction factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

Before the development of these provisions, the limiting tensile strain for flexural members was not stated, but was implicit in the maximum tension reinforcement ratio that was given as a fraction of ρ_b, which was dependent on the yield strength of the reinforcement. The net tensile strain limit of 0.005 for tension-controlled sections was chosen to be a single value that applies to all types of steel (prestressed and nonprestressed) permitted by this code.

Unless unusual amounts of ductility are required, the 0.005 limit will provide ductile behavior for most designs. One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames. Section 8.4 permits redistribution of negative moments. Because moment redistribution is dependent on adequate ductility in hinge regions, moment redistribution is limited to sections that have a net tensile strain of at least 0.0075.

For beams with compression reinforcement, or T-beams, the effects of compression reinforcement and flanges are automatically accounted for in the computation of net tensile strain ε_t.

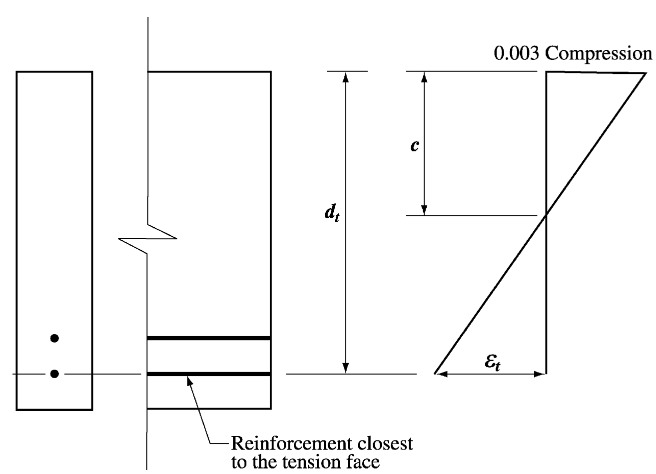

Fig. R10.3.3—Strain distribution and net tensile strain.

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10.3.4 — Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.

10.3.5 — For nonprestressed flexural members and nonprestressed members with axial load less than $0.10f_c'A_g$, the net tensile strain ε_t at nominal strength shall not be less than 0.004.

10.3.6 — Use of compression reinforcement shall be permitted in conjunction with additional tension reinforcement to increase the strength of flexural members.

10.3.7 — Design axial load strength ϕP_n of compression members shall not be taken greater than the following:

10.3.7.1 — For nonprestressed members with spiral reinforcement conforming to 7.10.4 or composite members conforming to 10.16

$$\phi P_{n(max)} = 0.85\phi[0.85f_c'(A_g - A_{st}) + f_yA_{st}] \quad (10\text{-}1)$$

10.3.7.2 — For nonprestressed members with tie reinforcement conforming to 7.10.5

$$\phi P_{n(max)} = 0.80\phi[0.85f_c'(A_g - A_{st}) + f_yA_{st}] \quad (10\text{-}2)$$

10.3.7.3 — For prestressed members, design axial load strength ϕP_n shall not be taken greater than 0.85 (for members with spiral reinforcement) or 0.80 (for members with tie reinforcement) of the design axial load strength at zero eccentricity ϕP_o.

10.3.8 — Members subject to compressive axial load shall be designed for the maximum moment that can accompany the axial load. The factored axial load P_u at given eccentricity shall not exceed that given in 10.3.7.The maximum factored moment M_u shall be magnified for slenderness effects in accordance with 10.10.

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R10.3.5 — The effect of this limitation is to restrict the reinforcement ratio in nonprestressed beams to about the same ratio as in editions of the ACI 318 code before 2002. The reinforcement limit of $0.75\rho_b$ results in a net tensile strain at nominal strength of 0.00376. The proposed limit of 0.004 is slightly more conservative. This limitation does not apply to prestressed members.

R10.3.7 and R10.3.8 — The minimum design eccentricities included in the 1963 and 1971 ACI 318 codes were deleted from the ACI 318-77 code except for consideration of slenderness effects in compression members with small or zero computed end moments (see 10.12.3.2). The specified minimum eccentricities were originally intended to serve as a means of reducing the axial load design strength of a section in pure compression to account for accidental eccentricities not considered in the analysis that may exist in a compression member, and to recognize that concrete strength may be less than f_c' under sustained high loads. The primary purpose of the minimum eccentricity requirement was to limit the maximum design axial load strength of a compression member. This is now accomplished directly in 10.3.7 by limiting the design axial load strength of a section in pure compression to 85 or 80 percent of the nominal strength. These percentage values approximate the axial load strengths at e/h ratios of 0.05 and 0.10, specified in the earlier codes for the spirally reinforced and tied members, respectively. The same axial load limitation applies to both cast-in-place and precast compression members. Design aids and computer programs based on the minimum eccentricity requirement of the 1963 and 1971 318 codes are equally applicable.

For prestressed members, the design axial load strength in pure compression is computed by the strength design methods of Chapter 10, including the effect of the prestressing force.

Compression member end moments should be considered in the design of adjacent flexural members. In nonsway frames, the effects of magnifying the end moments need not be considered in the design of the adjacent beams. In sway frames, the magnified end moments must be considered in designing the flexural members, as required in 10.13.7.

Corner and other columns exposed to known moments about each axis simultaneously should be designed for biaxial bending and axial load. Satisfactory methods are available in the *ACI Design Handbook*[10.4] and the *CRSI Handbook*.[10.5] The reciprocal load method[10.6] and the load contour method[10.7] are the methods used in those two handbooks. Research[10.8,10.9] indicates that using the rectangular stress block provisions of 10.2.7 produces satisfactory strength estimates for doubly symmetric sections. A simple and somewhat conservative estimate of nominal strength P_{ni} can be obtained from the reciprocal load relationship[10.6]

$$\frac{1}{P_{ni}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_o}$$

where

P_{ni} = nominal axial load strength at given eccentricity along both axes;

P_o = nominal axial load strength at zero eccentricity;

P_{nx} = nominal axial load strength at given eccentricity along x-axis;

P_{ny} = nominal axial load strength at given eccentricity along y-axis.

This relationship is most suitable when values P_{nx} and P_{ny} are greater than the balanced axial force P_b for the particular axis.

10.4 — Distance between lateral supports of flexural members

10.4.1 — Spacing of lateral supports for a beam shall not exceed 50 times the least width b of compression flange or face.

10.4.2 — Effects of lateral eccentricity of load shall be taken into account in determining spacing of lateral supports.

R10.4 — Distance between lateral supports of flexural members

Tests[10.10,10.11] have shown that laterally unbraced reinforced concrete beams of any reasonable dimensions, even when very deep and narrow, will not fail prematurely by lateral buckling provided the beams are loaded without lateral eccentricity that could cause torsion.

Laterally unbraced beams are frequently loaded off center (lateral eccentricity) or with slight inclination. Stresses and deformations set up by such loading become detrimental for narrow, deep beams, the more so as the unsupported length increases. Lateral supports spaced closer than $50b$ may be required by actual loading conditions.

10.5 — Minimum reinforcement of flexural members

10.5.1 — At every section of a flexural member where tensile reinforcement is required by analysis, except as provided in 10.5.2, 10.5.3, and 10.5.4, the area A_s provided shall not be less than that given by

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d \qquad (10\text{-}3)$$

and not less than $200 b_w d / f_y$.

R10.5 — Minimum reinforcement of flexural members

The provision for a minimum amount of reinforcement applies to flexural members, which, for architectural or other reasons, are larger in cross section than required for strength. With a very small amount of tensile reinforcement, the computed moment strength as a reinforced concrete section using cracked section analysis becomes less than that of the corresponding unreinforced concrete section computed from its modulus of rupture. Failure in such a case can be sudden.

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To prevent such a failure, a minimum amount of tensile reinforcement is required by 10.5.1. This is required in both positive and negative moment regions. When concrete strength higher than about 5000 psi is used, the $200/f_y$ value previously prescribed may not be sufficient. Equation (10-3) gives the same amount of reinforcement as $200/f_y$ when f'_c equals 4440 psi. When the flange of a section is in tension, the amount of tensile reinforcement needed to make the strength of the reinforced section equal that of the unreinforced section is about twice that for a rectangular section or that of a flanged section with the flange in compression. A higher amount of minimum tensile reinforcement is particularly necessary in cantilevers and other statically determinate members where there is no possibility for redistribution of moments.

10.5.2 — For statically determinate members with a flange in tension, the area $A_{s,min}$ shall be equal to or greater than the value given by Eq. (10 -3) with b_w replaced by either $2b_w$ or the width of the flange, whichever is smaller.

10.5.3 — The requirements of 10.5.1 and 10.5.2 need not be applied if at every section the area of tensile reinforcement provided is at least one-third greater than that required by analysis for required strength U, not including the environmental durability factor S_d.

R10.5.3 — The minimum reinforcement required by Eq. (10-3) must be provided wherever reinforcement is needed, except where such reinforcement is at least one-third greater than that required by analysis. This exception provides sufficient additional reinforcement in large members where the amount required by 10.5.1 or 10.5.2 would be excessive.

10.5.4 — For structural slabs, mats, and footings of uniform thickness, the minimum area of tensile reinforcement in the direction of the span shall be the same as that required by 7.12. For walls, the minimum areas of reinforcement steel shall be as required by 14.3.2 and 14.3.3.

R10.5.4 — The minimum reinforcement required for slabs should be equal to the same amount as that required by 7.12 for shrinkage and temperature reinforcement.

Soil-supported slabs, such as slabs-on-grade, are not considered to be structural slabs in the context of this section unless they transmit vertical loads from other parts of the structure to the soil. Reinforcement, if any, in soil-supported slabs should be proportioned with due consideration of all design forces. Mat foundations and other slabs that help support the structure vertically should meet the requirements of this section.

In reevaluating the overall treatment of 10.5, the maximum spacing for reinforcement in structural slabs (including footings) was reduced from the $5h$ for temperature and shrinkage reinforcement to the compromise value of $3h$, which is somewhat larger than the $2h$ limit of 13.3.2 for two-way slab systems.

10.6 — Distribution of flexural reinforcement

10.6.1 — This section prescribes rules for distribution of flexural reinforcement and the allowable stresses used to control flexural cracking in all members that are not compression controlled sections. The requirements of this section do not apply to load combinations that include earthquake loads.

R10.6 — Distribution of flexural reinforcement

R10.6.1 — Many structures designed by working stress methods and with low steel stress served their intended functions with very limited flexural cracking. When high strength reinforcing steels are used at high service load stresses, however, visible cracks must be expected, and steps must be taken in detailing of the reinforcement to

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control cracking. For protection of reinforcement against corrosion, and for aesthetic reasons, many fine hairline cracks are preferable to a few wide cracks.

Control of cracking is particularly important when reinforcement with a yield strength in excess of 40,000 psi is used. Current good detailing practices will usually lead to adequate crack control even when reinforcement of 60,000 psi yield is used.

Extensive laboratory work[10.12-10.14] involving deformed bars has confirmed that crack width at service loads is proportional to steel stress. The significant variables reflecting steel detailing were found to be thickness of concrete cover and the area of concrete in the zone of maximum tension surrounding each individual reinforcing bar.

Crack width is inherently subject to wide scatter, even in careful laboratory work, and is influenced by shrinkage and other time-dependent effects. The best crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

10.6.2 — Distribution of flexural reinforcement in two-way slabs shall also meet the requirements of 13.3. For the application of 10.6.4, slabs with an aspect ratio (long span to short span) not greater than 2.0 shall be considered as two-way members and slabs with an aspect ratio greater than 2.0 shall be considered as one-way members.

R10.6.2 — For the purposes of design of environmental engineering concrete structures, no distinction is made between one-way and two-way elements with the exception of minimum stress levels for two-way members with aspect ratios less than or equal to 2.0. Two-way members with an aspect ratio greater than 2.0 have moment and shear diagrams at the midpoint along the long span that are basically indistinguishable from a one-way slab. Based on this observation, two-way members with aspect ratios greater than 2.0 are considered to be one-way members for the purposes of crack control. Crack width prediction in two-way elements is not as well defined as one-way elements; however, the intent of the design practice for environmental structures is to control stress levels to limits shown to control corrosion effectively rather than predict crack widths with any precision.

10.6.3 — Flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross section as required by 10.6.4.

R10.6.3 — Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

10.6.4 — The calculated stress f_s in reinforcement closest to a surface in tension at service loads shall not exceed that given by Eq. (10-4) and (10-5) and shall not exceed a maximum of 36,000 psi:

10.6.4.1 — In normal environmental exposure areas as defined in 10.6.4.5

$$f_{s,max} = \frac{320}{\beta\sqrt{s^2 + 4(2 + d_b/2)^2}} \qquad (10\text{-}4)$$

but need not be less than 20,000 psi for one-way and 24,000 psi for two-way members.

R10.6.4 — This section replaces the z factor requirements of the 2001 code edition. The maximum allowable stresses are now specified directly as a function of bar spacing.[10.15] The figures R10.6.4(a) through R10.6.4(d) are plots of Eq. (10-4) and (10-5) including the simplifications of Sections 10.6.4.3 and 10.6.4.4 and limitations for one- and two-way members. β is defined as the ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement. These figures may be used to select an allowable stress based on a maximum bar spacing to be used in bar selection.

Crack widths in environmental structures are highly variable. In previous codes, provisions were given for distribution of reinforcement that were based on empirical equations using

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10.6.4.2 — In severe environmental exposure areas as defined in 10.6.4.5

$$f_{s,max} = \frac{260}{\beta\sqrt{s^2 + 4(2 + d_b/2)^2}} \quad (10\text{-}5)$$

but need not be less than 17,000 psi for one-way and 20,000 psi for two-way members.

10.6.4.3 — In Eq. (10-4) and (10-5) it shall be permitted to use the value 25 for the term $4(2 + d_b/2)^2$ as a simplification.

10.6.4.4 — The strain gradient amplification factor shall be given by

$$\beta = \frac{h - c}{d - c} \quad (10\text{-}6)$$

where c is calculated at service loads. In lieu of this more precise calculation, it shall be permitted to use β equal to 1.2 for $h \geq 16$ in. and 1.35 for $h < 16$ in. in Eq. (10-4) and (10-5).

10.6.4.5 — For liquid retention, normal environmental exposure is defined as exposure to liquids with a pH greater than 5, or exposure to sulfate solutions of 1000 ppm or less. Severe environmental exposures are conditions in which the limits defining normal environmental exposure are exceeded.

10.6.4.6 — Calculated flexural stress in reinforcement at service load f_s (in ksi) shall be computed as the unfactored moment divided by the product of steel area and internal moment arm.

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a calculated maximum crack width of 0.010 in. for normal environmental exposure. The current provisions for spacing are intended to limit surface cracks to a width that is generally acceptable in practice, but may vary widely in a given structure.

The role of cracks in the corrosion of reinforcement is controversial. Research[10.16,10.17] shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels. Although a number of studies have been conducted, clear experimental evidence is not available regarding the crack width beyond which a corrosion danger exists. Environmental engineering concrete structures have traditionally performed well using quality concrete, as defined by this code, using adequate compaction, limiting maximum bar stresses, and equally distributing more smaller bars rather than few larger bars on tension faces.

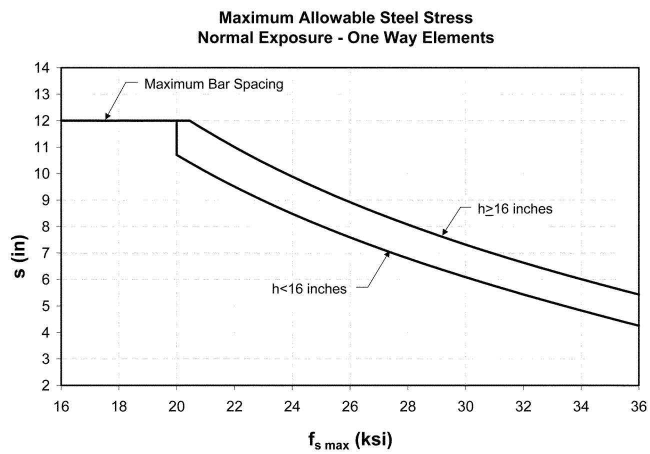

Fig. R10.6.4(a)—Maximum allowable steel stress, normal exposure—one-way elements.

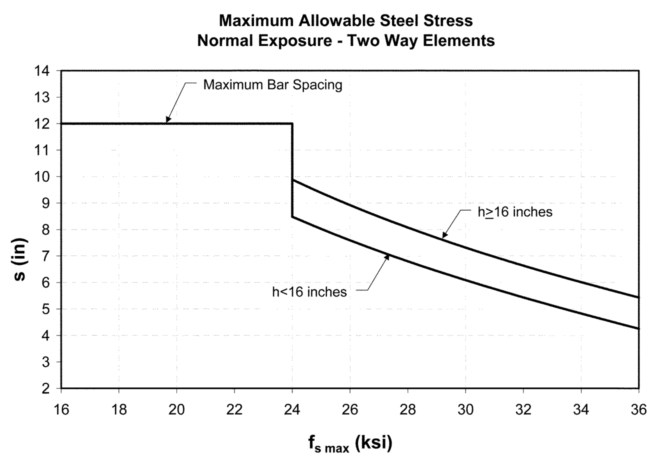

Fig. R10.6.4(b)—Maximum allowable steel stress, normal exposure—two-way elements.

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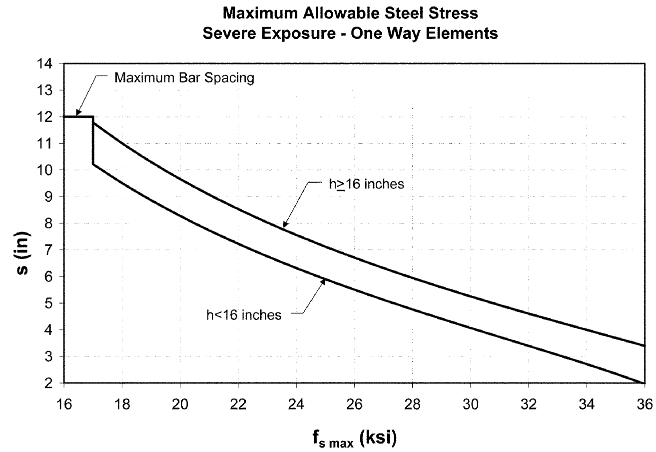

Fig. R10.6.4(c)—Maximum allowable steel stress, severe exposure—one-way elements.

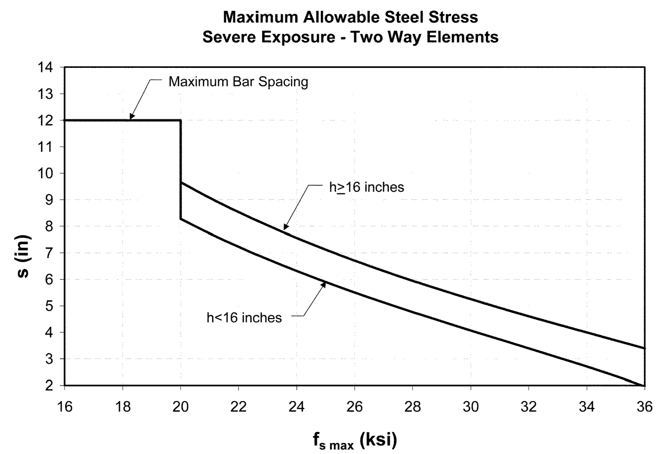

Fig. R10.6.4(d)—Maximum allowable steel stress, severe exposure—two-way elements.

10.6.5 — Where appearance of the concrete surface is of concern and concrete cover exceeds 3 in., the service load flexural tension stress must not exceed the values given in 10.6.4, and the spacing s of reinforcement closest to the surface in tension shall not exceed that given by

$$s = \frac{540}{f_s} - 2.5c_c \qquad (10\text{-}7)$$

but not greater than 12 in.

R10.6.5 — For most conditions, crack control criteria for environmental engineering concrete structures will satisfy appearance considerations. The exception is where a cover greater than 2 in. is used, because the cover in excess of 2 in. is neglected in Eq. (10-4) and (10-5). Equation (10-6) is taken from ACI 318-02, 10.6.4, and is intended to limit surface cracks to a width that is generally acceptable in practice.

10.6.6 — Where flanges of T-beam construction are in tension, part of the flexural tension reinforcement shall be distributed over an effective flange width as defined in 8.10, or a width equal to one-tenth the span, whichever is smaller. If the effective flange width exceeds one-tenth the span, some longitudinal reinforcement shall be provided in the outer portions of the flange.

R10.6.6 — In major T-beams, distribution of the negative reinforcement for control of cracking must take into account two considerations: (1) wide spacing of the reinforcement across the full effective width of flange may cause some wide cracks to form in the slab near the web; and (2) close spacing near the web leaves the outer regions of the flange unprotected. The 1/10 limitation is to guard against too wide

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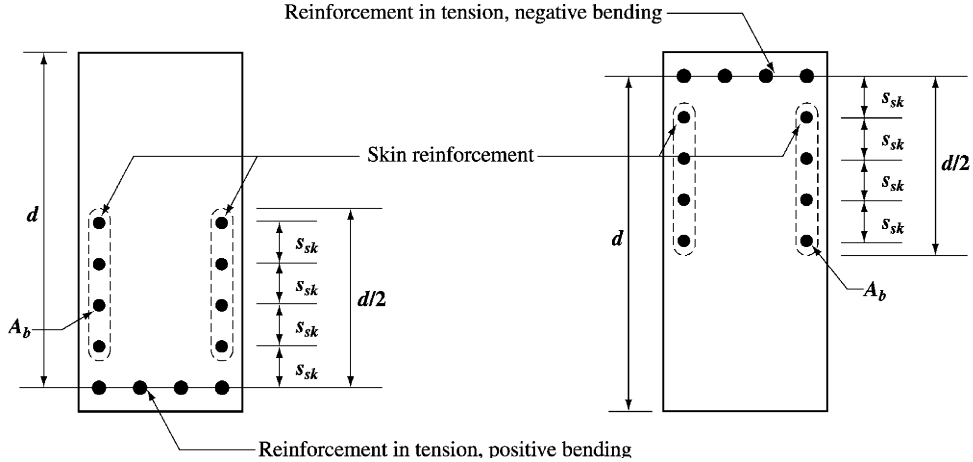

*Fig. R10.6.7—Skin reinforcement for beams and joists with **d** > 36 in.*

a spacing, with some additional reinforcement required to protect the outer portions of the flange.

10.6.7 — If the effective depth d of a beam or joist exceeds 36 in., longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance $d/2$ nearest the flexural tension reinforcement. The spacing s_{sk} between longitudinal bars or wires of the skin reinforcement shall not exceed the least of $d/6$, 12 in., and $1000A_b/(d-30)$. It shall be permitted to include such reinforcement in strength computations if a strain compatibility analysis is made to determine stress in the individual bars or wires. The total area of longitudinal skin reinforcement in both faces need not exceed one-half of the required flexural tensile reinforcement.

R10.6.7 — For relatively deep flexural members, some reinforcement should be placed near the vertical faces of the tension zone to control cracking in the web.[10.15] (See Fig. R10.6.7.) Without such auxiliary steel, the width of the cracks in the web may exceed the crack widths at the level of the flexural tension reinforcement.

Where the provisions for deep beams, walls, or precast panels require more steel, those provisions (along with their spacing requirements) will govern.

10.7 — Deep beams

10.7.1 — Beams with overall depth to clear span ratios greater than 2/5 for continuous spans, or 4/5 for simple spans, shall be designed as deep beams taking into account nonlinear distribution of strain and lateral buckling. (See also 12.10.6.)

10.7.2 — Shear strength of deep beams shall be in accordance with 11.8.

10.7.3 — Minimum flexural tension reinforcement shall conform to 10.5.

10.7.4 — Minimum horizontal and vertical reinforcement in the side faces of deep beams shall be the greater of the requirements of 11.8.8, 11.8.9, and 11.8.10 or 14.3.2 and 14.3.3.

R10.7 — Deep beams

The code does not contain detailed requirements for designing deep beams for flexure except that nonlinearity of strain distribution and lateral buckling must be considered. Suggestions for the design of deep beams for flexure are given in References 10.18, 10.19, and 10.20.

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10.8 — Design dimensions for compression members

10.8.1 — Isolated compression member with multiple spirals

Outer limits of the effective cross section of a compression member with two or more interlocking spirals shall be taken at a distance outside the extreme limits of the spirals equal to the minimum concrete cover required by 7.7.

10.8.2 — Compression member built monolithically with wall

Outer limits of the effective cross section of a spirally reinforced or tied reinforced compression member built monolithically with a concrete wall or pier shall be taken not greater than 1-1/2 in. outside the spiral or tie reinforcement.

10.8.3 — Equivalent circular compression member

As an alternative to using the full gross area for design of a compression member with a square, octagonal, or other shaped cross section, it shall be permitted to use a circular section with a diameter equal to the least lateral dimension of the actual shape. Gross area considered, required percentage of reinforcement, and design strength shall be based on that circular section.

10.8.4 — Limits of section

For a compression member with a cross section larger than required by considerations of loading, it shall be permitted to base the minimum reinforcement and strength on a reduced effective area A_g not less than one-half the total area. This provision shall not apply in regions of high seismic risk.

10.9 — Limits for reinforcement of compression members

10.9.1 — Area of longitudinal reinforcement for noncomposite compression members shall be not less than 0.01 nor more than 0.08 times gross area A_g of section.

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R10.8 — Design dimensions for compression members

With the 1971 edition of the ACI 318 Building Code, minimum sizes for compression members were eliminated to allow wider utilization of reinforced concrete compression members in smaller size and lightly loaded structures, such as low rise residential and light office buildings. The engineer should recognize the need for careful workmanship, as well as the increased significance of shrinkage stresses with small sections.

R10.8.2, R10.8.3, and R10.8.4 — For column design,[10.21] the code provisions for quantity of reinforcement, both vertical and spiral, are based on the gross column area and core area, and the design strength of the column is based on the gross area of the column section. In some cases, however, the gross area is larger than necessary to carry the factored load. The basis of 10.8.2, 10.8.3, and 10.8.4 is that it is satisfactory to design a column of sufficient size to carry the factored load and then simply add concrete around the designed section without increasing the reinforcement to meet the minimum percentages required by 10.9.1. The additional concrete must not be considered as carrying load; however, the effects of the additional concrete on member stiffness must be included in the structural analysis. The effects of the additional concrete also must be considered in design of the other parts of the structure that interact with the oversize member.

R10.9 — Limits for reinforcement of compression members

R10.9.1 — This section prescribes the limits on the amount of longitudinal reinforcement for noncomposite compression members. If the use of high reinforcement ratios would involve practical difficulties in the placing of concrete, a lower percentage, and hence, a larger column or higher-strength concrete or reinforcement (see R9.4) should be considered. The percentage of reinforcement in columns

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should usually not exceed 4 percent if the column bars are required to be lap spliced.

Minimum reinforcement. Because the design methods for columns incorporate separate terms for the load carried by concrete and by reinforcement, it is necessary to specify some minimum amount of reinforcement to ensure that only reinforced concrete columns are designed by these procedures. Reinforcement is necessary to provide resistance to bending, which may exist whether or not computations show that bending exists, and to reduce the effects of creep and shrinkage of the concrete under sustained compressive stresses. Tests have shown that creep and shrinkage tend to transfer load from the concrete to the reinforcement, with a consequent increase in stress in the reinforcement, and that this increase is greater as the ratio of reinforcement decreases. Unless a lower limit is placed on this ratio, the stress in the reinforcement may increase to the yield level under sustained service loads. This phenomenon was emphasized in the report of ACI Committee 105[10.22] and minimum reinforcement ratios of 0.01 and 0.005 were recommended for spiral and tied columns, respectively. In all editions of the ACI Building Code since 1936, however, the minimum ratio has been 0.01 for both types of laterally reinforced columns.

Maximum reinforcement. Extensive tests of the ACI column investigation[10.22] included reinforcement ratios no greater than 0.06. Although other tests with as much as 17 percent reinforcement in the form of bars produced results similar to those obtained previously, it is necessary to note that the loads in these tests were applied through bearing plates on the ends of the columns and the problem of transferring a proportional amount of the load to the bars was thus minimized or avoided. Maximum ratios of 0.08 and 0.03 were recommended by ACI Committee 105[10.22] for spiral and tied columns, respectively. In the 1936 ACI Building Code, these limits were made 0.08 and 0.04, respectively. In the 1956 code, the limit for tied columns with bending was raised to 0.08. Since the ACI 318-63 code, it has been required that bending be considered in the design of all columns, and the maximum ratio of 0.08 has been applied to both types of columns. This limit can be considered a practical maximum for reinforcement in terms of economy and requirements for placing.

10.9.2 — Minimum number of longitudinal bars in compression members shall be four for bars within rectangular or circular ties, three for bars within triangular ties, and six for bars enclosed by spirals conforming to 10.9.3.

R10.9.2 — For compression members, a minimum of four longitudinal bars is required when bars are enclosed by rectangular or circular ties. For other shapes, one bar should be provided at each apex or corner and proper lateral reinforcement provided. For example, tied triangular columns require three longitudinal bars, one at each apex of the triangular ties. For bars enclosed by spirals, six bars are required.

When the number of bars in a circular arrangement is less than eight, the orientation of the bars will affect the moment strength of eccentrically loaded columns and must be considered in design.

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10.9.3 — Ratio of spiral reinforcement ρ_s shall be not less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \qquad (10\text{-}8)$$

where f_y is the specified yield strength of spiral reinforcement but not more than 60,000 psi.

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R10.9.3 — The effect of spiral reinforcement in increasing the load-carrying strength of the concrete within the core is not realized until the column has been subjected to a load and deformation sufficient to cause the concrete shell outside the core to spall off. The amount of spiral reinforcement required by Eq. (10-7) is intended to provide additional load-carrying strength for concentrically loaded columns equal to or slightly greater than the strength lost when the shell spalls off. This principle was recommended by ACI Committee 105[10.22] and has been a part of the ACI 318 code since 1963. The derivation of Eq. (10-7) is given in the ACI Committee 105 report. Tests and experience show that columns containing the amount of spiral reinforcement required by this section exhibit considerable toughness and ductility.

10.10 — Slenderness effects in compression members

R10.10 — Slenderness effects in compression members

Provisions for slenderness effects in compression members and frames were revised in the ACI 318-95 code to better recognize the use of second-order analyses and to improve the arrangement of the provisions dealing with sway (unbraced) and nonsway (braced) frames.[10.23] The use of a refined nonlinear second-order analysis is permitted in 10.10.1. Sections 10.11, 10.12, and 10.13 present an approximate design method based on the traditional moment magnifier method. For sway frames, the magnified sway moment $\delta_s M_s$ may be calculated using a second-order elastic analysis, by an approximation to such an analysis, or by the traditional sway moment magnifier.

10.10.1 — Except as allowed in 10.10.2, the design of compression members, restraining beams, and other supporting members shall be based on the factored forces and moments from a second-order analysis considering material nonlinearity and cracking, as well as the effects of member curvature and lateral drift, duration of the loads, shrinkage and creep, and interaction with the supporting foundation. The dimensions of each member cross section used in the analysis shall be within 10 percent of the dimensions of the members shown on the design drawings or the analysis shall be repeated. The analysis procedure shall have been shown to result in prediction of strength in substantial agreement with the results of comprehensive tests of columns in statically indeterminate reinforced concrete structures.

R10.10.1 — Two limits are placed on the use of the refined second-order analysis. First, the structure that is analyzed must have members similar to those in the final structure. If the members in the final structure have cross-sectional dimensions more than 10 percent different from those assumed in the analysis, new member properties should be computed and the analysis repeated. Second, the refined second-order analysis procedure should have been shown to predict ultimate loads within 15 percent of those reported in tests of indeterminate reinforced concrete structures. At the very least, the comparison should include tests of columns in planar nonsway frames, sway frames, and frames with varying column stiffnesses. To allow for variability in the actual member properties and in the analysis, the member properties used in analysis should be multiplied by a stiffness reduction factor ϕ_K less than one. For consistency with the second-order analysis in 10.13.4.1, the stiffness reduction factor ϕ_K can be taken as 0.80. The concept of a stiffness reduction factor ϕ_K is discussed in R10.12.3.

10.10.2 — As an alternate to the procedure prescribed in 10.10.1, it shall be permitted to base the design of compression members, restraining beams, and other supporting members on axial forces and moments from the analyses described in 10.11.

R10.10.2 — As an alternate to the refined second-order analysis of 10.10.1, design may be based on elastic analyses and the moment magnifier approach.[10.24,10.25] For sway frames, the magnified sway moments may be calculated using a second-order elastic analysis based on realistic stiffness values. See R10.13.4.1.

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10.11 — Magnified moments—General

10.11.1 — The factored axial forces P_u, the factored moments M_1 and M_2 at the ends of the column, and, where required, the relative lateral story deflections Δ_o shall be computed using an elastic first-order frame analysis with the section properties determined taking into account the influence of axial loads, the presence of cracked regions along the length of the member, and effects of duration of the loads. Alternatively, it shall be permitted to use the following properties for the members in the structure:

(a) Modulus of elasticity........................ E_c from 8.5.1

(b) Moments of inertia

Beams.. $0.35I_g$

Columns.. $0.70I_g$

Walls—Uncracked.................................... $0.70I_g$

—Cracked .... $0.35I_g$

Flat plates and flat slabs .... $0.25I_g$

(c) Area .... $1.0A_g$

The moments of inertia shall be divided by $(1 + \beta_d)$;

(d) When sustained lateral loads act; or

(e) For stability checks made in accordance with 10.13.6.

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R10.11 — Magnified moments—General

This section describes an approximate design procedure that uses the moment magnifier concept to account for slenderness effects. Moments computed using an ordinary first-order frame analysis are multiplied by a "moment magnifier" that is a function of the factored axial load P_u and the critical buckling load P_c for the column. Nonsway and sway frames are treated separately in 10.12 and 10.13. Provisions applicable to both nonsway and sway columns are given in 10.11. A first-order frame analysis is an elastic analysis that does not include the internal force effects resulting from deflections.

R10.11.1 — The stiffnesses EI used in an elastic analysis used for strength design should represent the stiffnesses of the members immediately prior to failure. This is particularly true for a second-order analysis that should predict the lateral deflections at loads approaching ultimate. The EI value should not be based totally on the moment-curvature relationship for the most highly loaded section along the length of each member. Instead, they should correspond to the moment end rotation relationship for a complete member.

The alternative values of E, I, and A given in 10.11.1 have been chosen from the results of frame tests and analyses, and include an allowance for the variability of the computed deflections. The modulus of elasticity E is based on the specified concrete strength while the sway deflections are a function of the average concrete strength, which is higher. The moments of inertia were taken as 0.875 times those in Reference 10.26. These two effects result in an overestimation of the second-order deflections in the order of 20 to 25 percent, corresponding to an implicit stiffness reduction factor ϕ_K of 0.80 to 0.85 on the stability calculation. The concept of a stiffness reduction factor ϕ_K is discussed in R10.12.3.

The moment of inertia of T-beams should be based on the effective flange width defined in 8.10. It is generally sufficiently accurate to take I_g of a T-beam as two times the I_g for the web, $2(b_w h^3/12)$.

If the factored moments and shears from an analysis based on the moment of inertia of a wall taken equal to $0.70I_g$ indicate that the wall will crack in flexure, based on the modulus of rupture, the analysis should be repeated with $I = 0.35I_g$ in those stories where cracking is predicted at factored loads.

The alternative values of the moments of inertia given in 10.11.1 were derived for nonprestressed members. For prestressed members, the moments of inertia may differ from the values in 10.11.1 depending on the amount, location, and type of the reinforcement and the degree of cracking before ultimate. The stiffness values for prestressed concrete members should include an allowance for the variability of the stiffnesses.

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Sections 10.11 through 10.13 provide requirements for strength and assume frame analyses will be carried out using factored loads. Analyses of deflections, vibrations, and building periods are needed at various service (unfactored) load levels[10.27,10.28] to determine the serviceability of the structure and to estimate the wind forces in wind tunnel laboratories. The seismic base shear is also based on the service load periods of vibration. The magnified service loads and deflections by a second-order analysis should also be computed using service loads. The moments of inertia of the structural members in the service load analyses should, therefore, be representative of the degree of cracking at the various service load levels investigated. Unless a more accurate estimate of the degree of cracking at design service load level is available, it is satisfactory to use **1/0.70 = 1.43** times the moments of inertia given in 10.11.1 for service load analyses.

The last sentence in 10.11.1 refers to the unusual case of sustained lateral loads. Such a case might exist, for example, if there were permanent lateral loads resulting from unequal earth pressures on two sides of a building.

10.11.2 — It shall be permitted to take the radius of gyration $\boldsymbol{r}$ equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

10.11.3 — Unsupported length of compression members

10.11.3.1 — The unsupported length ℓ_u of a compression member shall be taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support in the direction being considered.

10.11.3.2 — Where column capitals or haunches are present, the unsupported length shall be measured to the lower extremity of the capital or haunch in the plane considered.

10.11.4 — Columns and stories in structures shall be designated as nonsway or sway columns or stories. The design of columns in nonsway frames or stories shall be based on 10.12. The design of columns in sway frames or stories shall be based on 10.13.

10.11.4.1 — It shall be permitted to assume a column in a structure is nonsway if the increase in column end moments due to second-order effects does not exceed 5 percent of the first-order end moments.

10.11.4.2 — It also shall be permitted to assume a story within a structure is nonsway if

R10.11.4 — The moment magnifier design method requires the designer to distinguish between nonsway frames that are designed according to 10.12 and sway frames that are designed according to 10.13. Frequently, this can be done by inspection by comparing the total lateral stiffness of the columns in a story to that of the bracing elements. A compression member may be assumed nonsway by inspection if it is located in a story in which the bracing elements (shearwalls, shear trusses, or other types of lateral bracing) have such substantial lateral stiffness to resist the lateral deflections of the story that any resulting lateral deflection is not large enough to affect the column strength substantially. If not readily apparent by inspection, 10.11.4.1 and 10.11.4.2 give two possible ways of doing this. In 10.11.4.1,

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$$Q = \frac{\Sigma P_u \Delta_o}{V_u \ell_c} \quad (10\text{-}9)$$

is less than or equal to 0.05, where ΣP_u and V_u are the total vertical load and the story shear, respectively, in the story in question and Δ_o is the first-order relative deflection between the top and bottom of that story due to V_u.

10.11.5 — Where an individual compression member in the frame has a slenderness $k\ell_u/r$ of more than 100, 10.10.1 shall be used to compute the forces and moments in the frame.

10.11.6 — For compression members subject to bending about both principal axes, the moment about each axis shall be magnified separately based on the conditions of restraint corresponding to that axis.

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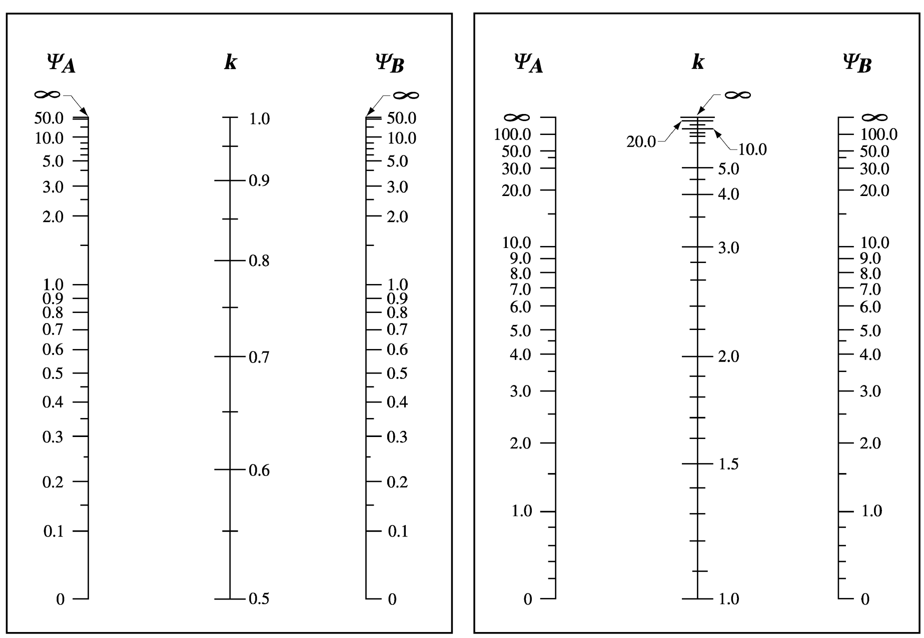
a story in a frame is said to be nonsway if the increase in the lateral load moments resulting from $P\Delta$ effects does not exceed 5 percent of the first-order moments.[10.26] Section 10.11.4.2 gives an alternative method of determining this based on the stability index for a story Q. In computing Q, ΣP_u should correspond to the lateral loading case for which ΣP_u is greatest. It should be noted that a frame may contain both nonsway and sway stories. This test would not be suitable if V_u were zero.

If the lateral load deflections of the frame have been computed using service loads and the service load moments of inertia given in 10.11.1, it is permissible to compute Q in Eq. (10-8) using 1.2 times the sum of the service gravity loads, the service load story shear, and 1.43 times the first-order service load story deflections.

R10.11.5 — An upper limit is imposed on the slenderness ratio of columns designed by the moment magnifier method of 10.11 to 10.13. No similar limit is imposed if design is carried out according to 10.10.1.The limit of $k\ell_u/r$ = **100** represents the upper range of actual tests of slender compression members in frames.

R10.11.6 — When biaxial bending occurs in a compression member, the computed moments about each of the principal axes must be magnified. The magnification factors δ are computed considering the buckling load P_c about each axis separately based on the appropriate effective length $k\ell_u$ and the stiffness EI. If the buckling capacities are different about the two axes, different magnification factors will result.



(a)
Nonsway Frames

(b)
Sway Frames

Ψ = ratio of $\Sigma(EI/\ell_c)$ of compression members to $\Sigma(EI/\ell)$ of flexural members in a plane at one end of a compression member

ℓ = span length of flexural member measured center to center of joints

Fig. R10.12.1—Effective length factors, **k**

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10.12 — Magnified moments—Nonsway frames

10.12.1 — For compression members in nonsway frames, the effective length factor k shall be taken as 1.0, unless analysis shows that a lower value is justified. The calculation of k shall be based on the E and I values used in 10.11.1.

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R10.12 — Magnified moments—Nonsway frames

R10.12.1 — The moment magnifier equations were derived for hinged end columns and should be modified to account for the effect of end restraints. This is done by using an "effective length" $k\ell_u$ in the computation of P_c.

The primary design aid to estimate the effective length factor k is the Jackson and Moreland Alignment Charts (Fig. R10.12.1), which allow a graphical determination of k for a column of constant cross section in a multibay frame.[10.29,10.30]

The effective length is a function of the relative stiffness at each end of the compression member. Studies have indicated that the effects of varying beam and column reinforcement percentages and beam cracking should be considered in determining the relative end stiffnesses. In determining ψ for use in evaluating the effective length factor k, the rigidity of the flexural members may be calculated on the basis of $0.35I_g$ for flexural members to account for the effect of cracking and reinforcement on relative stiffness, and $0.70I_g$ for compression members.

The simplified equations (A through E), listed below for computing the effective length factors for nonsway and sway members may be used. Equation (A), (B), and (E) are taken from the 1972 British Standard Code of Practice.[10.31,10.32] Equation (C) and (D) for sway members were developed in Reference 10.28.

For compression members in a nonsway frame, an upper bound to the effective length factor may be taken as the smaller of the following two expressions

$$k = 0.7 + 0.05(\psi_A + \psi_B) \le 1.0 \quad \text{(A)}$$

$$k = 0.85 + 0.05\psi_{min} \le 1.0 \quad \text{(B)}$$

where ψ_A and ψ_B are the values of ψ at the two ends of the column, and ψ_{min} is the smaller of the two values.

For compression members in a sway frame restrained at both ends, the effective length factor may be taken as:

For $\psi_m < 2$

$$k = \frac{20 - \psi_m}{20}\sqrt{1 + \psi_m} \quad \text{(C)}$$

For $\psi_m \ge 2$

$$k = 0.9\sqrt{1 + \psi_m} \quad \text{(D)}$$

where ψ_m is the average of the ψ-values at the two ends of the compression member.

For compression members in a sway frame hinged at one end, the effective length factor may be taken as

$$k = 2.0 + 0.3\psi \quad \text{(E)}$$

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10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy

$$\frac{k\ell_u}{r} \leq 34 - 12(M_1/M_2) \qquad (10\text{-}10)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

10.12.3 — Compression members shall be designed for the factored axial load P_u and the moment amplified for the effects of member curvature M_c as follows

$$M_c = \delta_{ns} M_2 \qquad (10\text{-}11)$$

where

$$\delta_{ns} = \frac{C_m}{1 - \dfrac{P_u}{0.75 P_c}} \geq 1.0 \qquad (10\text{-}12)$$

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \qquad (10\text{-}13)$$

EI shall be taken as

$$EI = \frac{(0.2E_c I_g + E_s I_{se})}{1 + \beta_d} \qquad (10\text{-}14)$$

or

$$EI = \frac{0.4E_c I_g}{1 + \beta_d} \qquad (10\text{-}15)$$

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where ψ is the value at the restrained end.

The use of the charts in Fig. R10.12.1, or the equations in this section, may be considered as satisfying the requirements of the code to justify k less than 1.0.

R10.12.2 — Equation (10-9) is derived from Eq. (10-11) assuming that a 5 percent increase in moments due to slenderness is acceptable.[10.24] The derivation did not include ϕ in the calculation of the moment magnifier. As a first approximation, k may be taken equal to 1.0 in Eq. (10-9).

R10.12.3 — The ϕ-factors used in the design of slender columns represent two different sources of variability. First, the stiffness reduction ϕ-factors in the magnifier equations in the 1989 and earlier ACI 318 codes were intended to account for the variability in the stiffness EI and the moment magnification analysis. Second, the variability of the strength of the cross section is accounted for by strength reduction ϕ-factors of 0.70 for tied columns and 0.75 for spiral columns. Studies reported in Reference 10.33 indicate that the stiffness reduction factor ϕ_K and the cross-sectional strength reduction ϕ-factors do not have the same values, contrary to the assumption in the 1989 and earlier ACI 318 codes. These studies suggest the stiffness reduction factor ϕ_K for an isolated column should be 0.75 for both tied and spiral columns. The 0.75 factors in Eq. (10-11) and (10-20) are stiffness reduction factors ϕ_K and replace the ϕ-factors in these equations in the 1989 and earlier codes. This has been done to avoid confusion between a stiffness reduction factor ϕ_K in Eq. (10-11) and (10-20), and the cross-sectional strength reduction ϕ-factors.

In defining the critical load, the main problem is the choice of a stiffness EI that reasonably approximates the variations in stiffness due to cracking, creep, and the nonlinearity of the concrete stress-strain curve. Equation (10-13) was derived for small eccentricity ratios and high levels of axial load where the slenderness effects are most pronounced.

Creep due to sustained load will increase the lateral deflections of a column and hence the moment magnification. This is approximated for design by reducing the stiffness EI used to compute P_c and hence δ_{ns} by dividing EI by $(1 + \beta_d)$. Both the concrete and steel terms in Eq. (10-13) are divided by $(1 + \beta_d)$. This reflects the premature yielding of steel in columns subjected to sustained load.

Either Eq. (10-13) or (10-14) may be used to compute EI. Equation (10-14) is a simplified approximation to Eq. (10-13). It is less accurate than Eq. (10-13).[10.34] Equation (10-14) may be simplified further by assuming $\beta_d = 0.6$. When this is done, Eq. (10-14) becomes

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$$EI = 0.25E_cI \tag{F}$$

The term β_d is defined differently for nonsway and sway frames. See 10.0. For nonsway frames, β_d is the ratio of the maximum factored axial sustained load to the maximum factored axial load.

10.12.3.1 — For members without transverse loads between supports, C_m shall be taken as

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \geq 0.4 \tag{10-16}$$

where M_1/M_2 is positive if the column is bent in single curvature. For members with transverse loads between supports, C_m shall be taken as 1.0.

R10.12.3.1 — The factor C_m is an equivalent moment correction factor. The derivation of the moment magnifier assumes that the maximum moment is at or near midheight of the column. If the maximum moment occurs at one end of the column, design must be based on an "equivalent uniform moment" C_mM_2, which would lead to the same maximum moment when magnified.[10.24]

In the case of compression members that are subjected to transverse loading between supports, it is possible that the maximum moment will occur at a section away from the end of the member. If this occurs, the value of the largest calculated moment occurring anywhere along the member should be used for the value of M_2 in Eq. (10-10). In accordance with the last sentence of 10.12.3.1, C_m is to be taken as 1.0 for this case.

10.12.3.2 — The factored moment M_2 in Eq. (10-11) shall not be taken less than

$$M_{2,min} = P_u(0.6 + 0.03h) \tag{10-17}$$

about each axis separately, where 0.6 and h are in inches. For members for which $M_{2,min}$ exceeds M_2, the value of C_m in Eq. (10-16) shall either be taken equal to 1.0, or shall be based on the ratio of the computed end moments M_1 and M_2.

R10.12.3.2 — In the code, slenderness is accounted for by magnifying the column end moments. If the factored column moments are very small or zero, the design of slender columns must be based on the minimum eccentricity given in this section. It is not intended that the minimum eccentricity be applied about both axes simultaneously.

The factored column end moments from the structural analysis are used in Eq. (10-15) in determining the ratio M_1/M_2 for the column when the design must be based on minimum eccentricity. This eliminates what would otherwise be a discontinuity between columns with computed eccentricities less than the minimum eccentricity and columns with computed eccentricities equal to or greater than the minimum eccentricity.

10.13 — Magnified moments—Sway frames

R10.13 — Magnified moments—Sway frames

The design of sway frames for slenderness was revised in the 1995 ACI Building Code. The revised procedure consists of three steps:

(1) The magnified sway moments δ_sM_s are computed. This should be done in one of three ways. First, a second-order elastic frame analysis may be used (10.13.4.1). Second, an approximation to such analysis (10.13.4.2) may be used. The third option is to use the sway magnifier δ_s from previous editions of the ACI 318 Building Code (10.13.4.3);

(2) The magnified sway moments δ_sM_s are added to the unmagnified nonsway moment M_{ns} at each end of each column (10.13.3). The nonsway moments may be computed using a first-order elastic analysis;

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(3) If the column is slender and the loads on it are high, it is checked to see whether the moments at points between the ends of the column exceed those at the ends of the column. As specified in 10.13.5 this is done using the nonsway frame magnifier δ_{ns} with P_c computed assuming k = **1.0** or less.

10.13.1 — For compression members not braced against sidesway, the effective length factor k shall be determined using E and I values in accordance with 10.11.1 and shall not be less than 1.0.

R10.13.1 — See R10.12.1.

10.13.2 — For compression members not braced against sidesway, it shall be permitted to neglect the effects of slenderness when $k\ell_u/r$ is less than 22.

10.13.3 — The moments M_1 and M_2 at the ends of an individual compression member shall be taken as

$$M_1 = M_{1ns} + \delta_s M_{1s} \qquad (10\text{-}18)$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \qquad (10\text{-}19)$$

where $\delta_s M_{1s}$ and $\delta_s M_{2s}$ shall be computed according to 10.13.4.

R10.13.3 — The analysis described in this section deals only with plane frames subjected to loads causing deflections in that plane. If torsional displacements are significant, a three-dimensional second-order analysis should be used.

10.13.4 — Calculation of $\delta_s M_s$

10.13.4.1 — The magnified way moments $\delta_s M_s$ shall be taken as the column end moments calculated using a second-order elastic analysis based on the member stiffnesses given in 10.11.1.

R10.13.4 — Calculation of $\delta_s M_s$

R10.13.4.1 — A second-order analysis is a frame analysis that includes the internal force effects resulting from deflections. When a second-order elastic analysis is used to compute $\delta_s M_s$, the deflections must be representative of the stage immediately prior to the ultimate load. For this reason the reduced EI values given in 10.11.1 must be used in the second-order analysis.

The term β_d is defined differently for nonsway and sway frames. See 10.0. Sway deflections due to short-term loads such as wind or earthquake are a function of the short-term stiffness of the columns following a period of sustained gravity load. For this case, the definition of β_d in 10.0 gives $\beta_d = 0$. In the unusual case of a sway frame where the lateral loads are sustained, β_d will not be zero. This might occur if a building on a sloping site is subjected to earth pressure on one side but not on the other.

In a second-order analysis, the axial loads in all columns that are not part of the lateral-load-resisting elements and depend on these elements for stability must be included.

In the 1989 and earlier ACI 318 codes, the moment magnifier equations for δ_b and δ_s included a stiffness reduction factor ϕ_K to cover the variability in the stability calculation. The second-order analysis method is based on the values of E and I from 10.11.1. These lead to a 20 to 25 percent overestimation of the lateral deflections that corresponds to a stiffness reduction factor ϕ_K between 0.80 and 0.85 on the $P\Delta$ moments. No additional ϕ-factor is needed in the stability calculation. Once the moments are established, selection of the cross sections of the columns involves the strength reduction factors ϕ from 9.3.2.2.

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10.13.4.2 — Alternatively it shall be permitted to calculate $\delta_s M_s$ as

$$\delta_s M_s = \frac{M_s}{1 - Q} \geq M_s \tag{10-20}$$

If δ_s calculated in this way exceeds 1.5, $\delta_s M_s$ shall be calculated using 10.13.4.1 or 10.13.4.3.

10.13.4.3 — Alternatively, it shall be permitted to calculate the magnified sway moment $\delta_s M_s$ as

$$\delta_s M_s = \frac{M_s}{1 - \dfrac{\Sigma P_u}{0.75 \Sigma P_c}} \geq M_s \tag{10-21}$$

where ΣP_u is the summation for all the vertical loads in a story and ΣP_c is the summation for all sway resisting columns in a story. P_c is calculated using Eq. (10-13) using k from 10.13.1 and EI from Eq. (10-14) or Eq. (10-15).

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R10.13.4.2 — The iterative $P\Delta$ analysis for second-order moments can be represented by an infinite series. The solution of this series is given by Eq. (10-19).[10.26] Reference 10.35 shows that Eq. (10-19) closely predicts the second-order moments in a sway frame until δ_s exceeds 1.5.

The $P\Delta$ moment diagrams for deflected columns are curved, with Δ related to the deflected shape of the columns. Equation (10-19) and most commercially available second-order frame analyses have been derived assuming that the $P\Delta$ moments result from equal and opposite forces of $P\Delta/\ell_c$ applied at the bottom and top of the story. These forces give a straight line $P\Delta$ moment diagram. The curved $P\Delta$ moment diagrams lead to lateral displacements in the order of 15 percent larger than those from the straight line $P\Delta$ moment diagrams. This effect can be included in Eq. (10-19) by writing the denominator as $(1 - 1.15Q)$ rather than $(1 - Q)$. The 1.15 factor has been left out of Eq. (10-19) to maintain consistency with commercially available computer programs.

If deflections have been calculated using service loads, Q in Eq. (10-19) should be calculated in the manner explained in R10.11.4.

In the 1989 and earlier ACI Building Codes, the moment magnifier equations for δ_b and δ_s included a stiffness reduction factor ϕ_K to cover the variability in the stability calculation. The Q factor analysis is based on deflections calculated using the values of E and I from 10.11.1 that include the equivalent of a stiffness reduction factor ϕ_K as explained in R10.13.4.1. As a result, no additional ϕ-factor is needed in the stability calculation. Once the moments are established using Eq. (10-19), selection of the cross sections of the columns involves the strength reduction factors ϕ from 9.3.2.2.

R10.13.4.3 — To check the effects of story stability, δ_s is computed as an averaged value for the entire story based on use of $\Sigma P_u / \Sigma P_c$. This reflects the interaction of all sway resisting columns in the story in the $P\Delta$ effects because the lateral deflection of all columns in the story must be equal in the absence of torsional displacements about a vertical axis. In addition, it is possible that a particularly slender individual column in a sway frame could have substantial midheight deflections even if adequately braced against lateral end deflections by other columns in the story. Such a column will have ℓ_u/r greater than the value given in Eq. (10-21), and would have to be checked using 10.13.5.

If the lateral load deflections involve a significant torsional displacement, the moment magnification in the columns farthest from the center of twist may be underestimated by the moment magnifier procedure. In such cases, a three-dimensional second-order analysis should be considered.

The 0.75 in the denominator of Eq. (10-20) is a stiffness reduction factor ϕ_K as explained in R10.12.3.

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10.13.5 — If an individual compression member has

$$\frac{\ell_u}{r} > \frac{35}{\sqrt{\dfrac{P_u}{f'_c A_g}}} \qquad (10\text{-}22)$$

it shall be designed for the factored axial load P_u and the moment M_c calculated using 10.12.3 in which M_1 and M_2 are computed in accordance with 10.13.3, β_d as defined for the load combination under consideration, and k as defined in 10.12.1.

10.13.6 — In addition to load cases involving lateral loads, the strength and stability of the structure as a whole under factored gravity loads shall be considered.

(a) When $\delta_s M_s$ is computed from 10.13.4.1, the ratio of second-order lateral deflections to first-order lateral deflections for 1.4 dead load and 1.7 live load plus lateral load applied to the structure shall not exceed 2.5;

(b) When $\delta_s M_s$ is computed according to 10.13.4.2, the value of Q computed using ΣP_u for 1.4 dead load plus 1.7 live load shall not exceed 0.60;

(c) When $\delta_s M_s$ is computed from 10.13.4.3, δ_s computed using ΣP_u and ΣP_c corresponding to the factored dead and live loads shall be positive and shall not exceed 2.5.

In cases (a), (b), and (c) above, β_d shall be taken as the ratio of the maximum factored sustained axial load to the maximum factored axial load.

COMMENTARY

In the calculation of EI, β_d will normally be zero for a sway frame because the lateral loads are generally of short duration. (See R10.13.4.1.)

R10.13.5 — The unmagnified nonsway moments at the ends of the columns are added to the magnified sway moments at the same points. Generally, one of the resulting end moments is the maximum moment in the column. For slender columns with high axial loads, however, the point of maximum moment may be between the ends of the column so that the end moments are no longer the maximum moments. If ℓ_u/r is less than the value given by Eq. (10-21), the maximum moment at any point along the height of such a column will be less than 1.05 times the maximum end moment. When ℓ_u/r exceeds the value given by Eq. (10-21), the maximum moment will occur at a point between the ends of the column and will exceed the maximum end moment by more than 5 percent.[10.23] In such a case, the maximum moment is calculated by magnifying the end moments using Eq. (10-10).

R10.13.6 — The possibility of sidesway instability under gravity loads alone should be investigated. When using second-order analyses to compile $\delta_s M_s$ (10.13.4.1), the frame should be analyzed twice for the case of factored gravity loads plus a lateral load applied to the frame. This load may be the lateral load used in design, or it may be a single lateral load applied to the top of the frame. The first analysis should be a first-order analysis, the second analysis should be a second-order analysis. The deflection from the second-order analysis should not exceed 2.5 times the deflection from the first-order analysis. If one story is much more flexible than the others, the deflection ratio should be computed in that story. The lateral load should be large enough to give deflections of a magnitude that can be compared accurately. In unsymmetrical frames that deflect laterally under gravity loads alone, the lateral load should act in the direction for which it will increase the lateral deflections.

When using 10.13.4.2 to compute $\delta_s M_s$, the value of Q evaluated using factored gravity loads should not exceed 0.60. This is equivalent to δ_s = **2.5**. The values of V_u and Δ_o used to compute Q can result from assuming any real or arbitrary set of lateral loads provided that V_u and Δ_o are both from the same loading. If Q as computed in 10.11.4.2 is 0.2 or less, the stability check in 10.13.6 is satisfied.

When $\delta_s M_s$ is computed using Eq. (10-20), an upper limit of 2.5 is placed on δ_s. For higher δ_s values, the frame will be very susceptible to variations in EI, foundation rotations, and the like. If δ_s exceeds 2.5, the frame must be stiffened to reduce δ_s. ΣP_u is to include the axial load in all columns and walls including columns that are not part of the lateral-load-resisting system. The value δ_s = **2.5** is a very high magnifier. It has been chosen to offset the conservatism inherent in the moment magnifier procedure.

For nonsway frames, β_d is the ratio of the maximum factored axial sustained load to the maximum factored axial load.

10.13.7 — In sway frames, flexural members shall be designed for the total magnified end moments of the compression members at the joint.

R10.13.7 — The strength of a sway frame is governed by the stability of the columns and by the degree of end restraint provided by the beams in the frame. If plastic hinges form in the restraining beam, the structure approaches a failure mechanism, and its axial load capacity is drastically reduced. Section 10.13.7 provides that the designer make certain that the restraining flexural members have the capacity to resist the magnified column moments.

10.14 — Axially loaded members supporting slab system

Axially loaded members supporting a slab system included within the scope of 13.1 shall be designed as provided in Chapter 10 and in accordance with the additional requirements of Chapter 13.

10.15 — Transmission of column loads through floor system

When the specified compressive strength of concrete in a column is greater than 1.4 times that specified for a floor system, transmission of load through the floor system shall be provided by 10.15.1, 10.15.2, or 10.15.3.

R10.15 — Transmission of column loads through floor system

The requirements of this section are based on a paper on the effect of floor concrete strength on column strength.[10.36] The provisions mean that where the column concrete strength does not exceed the floor concrete strength by more than 40 percent, no special precautions need be taken. For higher column concrete strengths, methods in 10.15.1 or 10.15.2 should be used for corner or edge columns. Methods in 10.15.1, 10.15.2, or 10.15.3 should be used for interior columns with adequate restraint on all four sides.

10.15.1 — Concrete of strength specified for the column shall be placed in the floor at the column location. Top surface of the column concrete shall extend 2 ft into the slab from face of column. Column concrete shall be well integrated with floor concrete, and shall be placed in accordance with 6.4.5 and 6.4.6.

R10.15.1 — Application of the concrete placement procedure described in 10.15.1 requires the placing of two different concrete mixtures in the floor system. The lower strength mixture should be placed while the higher strength concrete is still plastic and should be adequately vibrated to ensure the concretes are well integrated. This requires careful coordination of the concrete deliveries and the possible use of retarders. In some cases, additional inspection services will be required when this procedure is used. It is important that the higher strength concrete in the floor in the region of the column be placed before the lower strength concrete in the remainder of the floor to prevent accidental placing of the low strength concrete in the column area. It is the designer's responsibility to indicate on the drawings where the high and low strength concretes are to be placed.

With the 1983 ACI 318 code, the amount of column concrete to be placed within the floor is expressed as a simple 2-ft extension from face of column. Because the concrete placement requirement should be carried out in the field, it is now expressed in a way that is directly evident to workers. The new requirement will also locate the interface between column and floor concrete farther out into the floor, away from regions of very high shear.

CODE

10.15.2 — Strength of a column through a floor system shall be based on the lower value of concrete strength with vertical dowels and spirals as required.

10.15.3 — For columns laterally supported on four sides by beams of approximately equal depth or by slabs, it shall be permitted to base strength of the column on an assumed concrete strength in the column joint equal to 75 percent of column concrete strength plus 35 percent of floor concrete strength. In the application of 10.15.3, the ratio of column concrete strength shall not be taken greater than 2.5 for design.

10.16 — Composite compression members

10.16.1 — Composite compression members shall include all such members reinforced longitudinally with structural steel shapes, pipe, or tubing with or without longitudinal bars.

10.16.2 — Strength of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

10.16.3 — Any axial load strength assigned to concrete of a composite member shall be transferred to the concrete by members or brackets in direct bearing on the composite member concrete.

10.16.4 — All axial load strength not assigned to concrete of a composite member shall be developed by direct connection to the structural steel shape, pipe, or tube.

10.16.5 — For evaluation of slenderness effects, radius of gyration of a composite section shall be not greater than the value given by

$$r = \sqrt{\frac{(E_c I_g/5) + E_s I_t}{(E_c A_g/5) + E_s A_t}} \qquad (10\text{-}23)$$

and, as an alternative to a more accurate calculation, EI in Eq. (10-13) shall be taken either as Eq. (10-14) or

$$EI = \frac{(E_c I_g/5)}{1 + \beta_d} + E_s I_t \qquad (10\text{-}24)$$

COMMENTARY

R10.15.3 — Research[10.37] has shown that heavily loaded slabs do not provide as much confinement as lightly loaded slabs when ratios of column concrete strength to slab concrete strength exceed about 2.5. Consequently, a limit is placed on the concrete strength ratio assumed in design.

R10.16 — Composite compression members

R10.16.1 — Composite columns are defined without reference to classifications of combination, composite, or concrete-filled pipe column. Reference to other metals used for reinforcement has been omitted because they are seldom used with concrete in construction.

R10.16.2 — The same rules used for computing the load-moment interaction strength for reinforced concrete sections can be applied to composite sections. Interaction charts for concrete-filled tubing would have a form identical to those of ACI SP-7[10.38] and the *Design Handbook*, V. 2, Columns,[10.30] but with γ slightly greater than 1.0.

R10.16.3 and R10.16.4 — Direct bearing or direct connection for transfer of forces between steel and concrete can be developed through lugs, plates, or reinforcing bars welded to the structural shape or tubing before the concrete is cast. Flexural compressive stress need not be considered a part of direct compression load to be developed by bearing. A concrete encasement around a structural steel shape may stiffen the shape, but it would not necessarily increase its strength.

R10.16.5 — Equation (10-22) is given because the rules of 10.11.2 for estimating the radius of gyration are overly conservative for concrete-filled tubing and are not applicable for members with enclosed structural shapes.

In reinforced concrete columns subject to sustained loads, creep transfers some of the load from the concrete to the steel, thus increasing the steel stresses. In the case of lightly reinforced columns, this load transfer may cause the compression steel to yield prematurely, resulting in a loss in the effective ***EI***. Accordingly, both the concrete and steel terms in Eq. (10-13) are reduced to account for creep. For heavily reinforced columns or for composite columns in which the pipe or structural shape makes up a large percentage of the cross section, the load transfer due to creep is not significant. Accordingly, Eq. (10-23) was revised in the 1980 ACI 318 code supplement so that only the ***EI*** of the concrete is reduced for sustained load effects.

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10.16.6 — Structural steel encased concrete core

10.16.6.1 — For a composite member with concrete core encased by structural steel, thickness of the steel encasement shall be not less than $b\sqrt{f_y/3E_s}$ for each face of width b nor $h\sqrt{f_y/8E_s}$ for circular sections of diameter h.

10.16.6.2 — Longitudinal bars located within the encased concrete core shall be permitted to be used in computing A_t and I_t.

10.16.7 — Spiral reinforcement around structural steel core

A composite member with spirally reinforced concrete around a structural steel core shall conform to 10.16.7.1 through 10.16.7.5.

10.16.7.1 — Specified compressive strength of concrete f'_c shall be not less than that given in 1.1.1.

10.16.7.2 — Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 50,000 psi.

10.16.7.3 — Spiral reinforcement shall conform to 10.9.3.

10.16.7.4 — Longitudinal bars located within the spiral shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.16.7.5 — Longitudinal bars located within the spiral shall be permitted to be used in computing A_t and I_t.

10.16.8 — Tie reinforcement around structural steel core

A composite member with laterally tied concrete around a structural steel core shall conform to 10.16.8.1 through 10.16.8.8.

10.16.8.1 — Specified compressive strength of concrete f'_c shall be not less than that given in 1.1.1.

10.16.8.2 — Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 50,000 psi.

10.16.8.3 — Lateral ties shall extend completely around the structural steel core.

10.16.8.4 — Lateral ties shall have a diameter not less than 0.02 times the greatest side dimension of composite member, except that ties shall not be smaller than No. 3 and are not required to be larger than No. 5. Welded wire fabric of equivalent area shall be permitted.

10.16.8.5 — Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 tie bar

COMMENTARY

R10.16.6 — Structural steel encased concrete core

Steel-encased concrete sections should have a metal wall thickness large enough to attain longitudinal yield stress before buckling outward.

R10.16.7 — Spiral reinforcement around structural steel core

Concrete that is laterally contained by a spiral has increased load-carrying strength, and the size of spiral required can be regulated on the basis of the strength of the concrete outside the spiral by means of the same reasoning that applies for columns reinforced only with longitudinal bars. The radial pressure provided by the spiral ensures interaction between concrete, reinforcing bars, and steel core such that longitudinal bars will both stiffen and strengthen the cross section.

R10.16.8 — Tie reinforcement around structural steel core

Concrete that is laterally contained by tie bars is likely to be rather thin along at least one face of a steel core section. Therefore, complete interaction between the core, the concrete, and any longitudinal reinforcement should not be assumed. Concrete will probably separate from smooth faces of the steel core. To maintain the concrete around the structural steel core, it is reasonable to require more lateral ties than needed for ordinary reinforced concrete columns. Because of probable separation at high strains between the steel core and the concrete, longitudinal bars will be ineffective in stiffening cross sections even though they would be useful in sustaining compression forces. The yield strength of the steel core should be limited to that which exists at strains below those that can be sustained without spalling of the concrete. It has been assumed that axially compressed concrete will not spall at strains less than 0.0018. The yield strength of 0.0018 × 29,000,000, or 52,000 psi, represents an upper limit of the useful maximum steel stress.

CODE

diameters, or 0.5 times the least side dimension of the composite member.

10.16.8.6 — Longitudinal bars located within the ties shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.16.8.7 — A longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not farther apart than one-half the least side dimension of the composite member.

10.16.8.8 — Longitudinal bars located within the ties shall be permitted to be used in computing A_t for strength but not in computing I_t for evaluation of slenderness effects.

10.17 — Bearing strength

10.17.1 — Design bearing strength on concrete shall not exceed $\phi(0.85 f_c' A_1)$, except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area shall be permitted to be multiplied by $\sqrt{A_2/A_1}$ but not more than 2.

COMMENTARY

R10.17 — Bearing strength

R10.17.1 — This section deals with bearing strength on concrete supports. The permissible bearing stress of $0.85 f_c'$ is based on tests reported in Reference 10.39. (See also 15.8.)

When the supporting area is wider than the loaded area on all sides, the surrounding concrete confines the bearing area, resulting in an increase in bearing strength. No minimum depth is given for a supporting member. The minimum depth of support will be controlled by the shear requirements of 11.12.

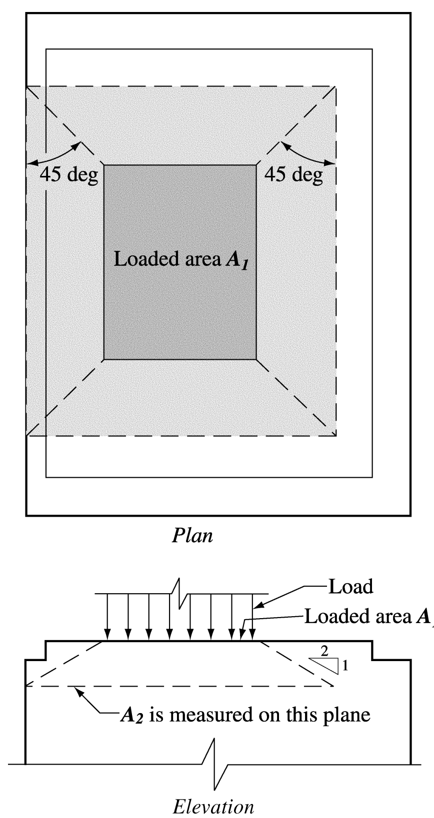

Fig. R10.17—Application of frustum to find A_2 *in stepped or sloped supports.*

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10.17.2 — Section 10.17 does not apply to post-tensioning anchorages.

COMMENTARY

When the top of the support is sloped or stepped, advantage may still be taken of the condition that the supporting member is larger than the loaded area, provided the supporting member does not slope at too great an angle. Figure R10.17 illustrates the application of the frustum to find A_2. The frustum should not be confused with the path by which a load spreads out as it travels downward through the support. Such a load path would have steeper sides. The frustum described, however, has somewhat flat side slopes to ensure that there is concrete immediately surrounding the zone of high stress at the bearing. A_1 is the loaded area but not greater than the bearing plate or bearing cross-sectional area.

R10.17.2 — Post-tensioning anchorages are normally laterally reinforced, in accordance with 18.13.

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Notes

COMMENTARY

CHAPTER 11 — SHEAR AND TORSION

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COMMENTARY

This chapter includes shear and torsion provisions for both nonprestressed and prestressed concrete members. The shear-friction concept (11.7) is particularly applicable to design of reinforcement details in precast structures. Special provisions are included for deep flexural members (11.8), brackets and corbels (11.9), and shearwalls (11.10). Shear provisions for slabs and footings are given in 11.12.

11.0 — Notation

a = shear span, distance between concentrated load and face of support, in.

A_c = area of concrete section resisting shear transfer, in.^{2}

A_{cp} = area enclosed by outside perimeter of concrete cross section, in.^{2} See 11.6.1

A_f = area of reinforcement in bracket or corbel resisting factored moment, $[V_u a + N_{uc}(h - d)]$, in.^{2}

A_g = gross area of section, in.^{2} For a hollow section, A_g is the area of the concrete only and does not include the area of the void(s)

A_h = area of shear reinforcement parallel to flexural tension reinforcement, in.^{2}

A_l = total area of longitudinal reinforcement to resist torsion, in.^{2}

A_n = area of reinforcement in bracket or corbel resisting tensile force N_{uc}, in.^{2}

A_o = gross area enclosed by shear flow path, in.^{2}

A_{oh} = area enclosed by centerline of the outermost closed transverse torsional reinforcement, in.^{2}

A_{ps} = area of prestressed reinforcement in tension zone, in.^{2}

A_s = area of nonprestressed tension reinforcement, in.^{2}

A_t = area of one leg of a closed stirrup resisting torsion within a distance s, in.^{2}

A_v = area of shear reinforcement within a distance s, or area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s for deep flexural members, in.^{2}

A_{vf} = area of shear-friction reinforcement, in.^{2}

A_{vh} = area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2, in.^{2}

b = width of compression face of member, in.

b_o = perimeter of critical section for slabs and footings, in.

b_t = width of that part of cross section containing the closed stirrups resisting torsion, in.

b_w = web width, or diameter of circular section, in.

R11.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip.

Tests[11.1] have indicated that the average shear over the full effective section also may be applied for circular sections. Note the special definition of d for such sections.

Although the value of d may vary along the span of a prestressed beam, studies[11.2] showed that, for prestressed concrete members, d need not be taken less than $0.8h$. The beams considered had some straight tendons or reinforcing bars at the bottom of the section and had stirrups that enclosed that steel.

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COMMENTARY

b_1 = width of the critical section defined in 11.12.1.2 measured in the direction of the span for which moments are determined, in.

b_2 = width of the critical section defined in 11.12.1.2 measured in the direction perpendicular to b_1, in.

c_1 = size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, in.

c_2 = size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which moments are being determined, in.

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, but need not be less than $0.80h$ for prestressed members, in. (For circular sections, d need not be less than the distance from extreme compression fiber to centroid of tension reinforcement in opposite half of member.)

f_c' = specified compressive strength of concrete, psi

$\sqrt{f_c'}$ = square root of specified compressive strength of concrete, psi

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, psi

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, psi. (In a composite member, f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone)

f_{pe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads, psi

f_{pu} = specified tensile strength of prestressing steel, psi

f_y = specified yield strength of nonprestressed reinforcement, psi

f_{yh} = specified yield strength of circular tie, hoop, or spiral reinforcement, psi

f_{yv} = yield strength of closed transverse torsional reinforcement, psi

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$f_{y\ell}$ = yield strength of longitudinal torsional reinforcement, psi
h = overall thickness of member, in.
h_v = total depth of shearhead cross section, in.
h_w = total height of wall from base to top, in.
I = moment of inertia of section resisting externally applied factored loads
ℓ_n = clear span measured face-to-face of supports, in.
ℓ_v = length of shearhead arm from centroid of concentrated load or reaction, in.
ℓ_w = horizontal length of wall, in.
M_{cr} = moment causing flexural cracking at section due to externally applied loads. See 11.4.2.1
M_m = modified moment, in.-lb
M_{max} = maximum factored moment at section due to externally applied loads, in.-lb
M_p = required plastic moment strength of shearhead cross section, in.-lb
M_u = factored moment at section, in.-lb
M_v = moment resistance contributed by shearhead reinforcement, in.-lb
N_u = factored axial load normal to cross section occurring simultaneously with V_u or T_u; to be taken as positive for compression, lb
N_{uc} = factored tensile force applied at top of bracket or corbel acting simultaneously with V_u, to be taken as positive for tension, lb
p_{cp} = outside perimeter of the concrete cross section, in. See 11.6.1.
p_h = perimeter of centerline of outermost closed transverse torsional reinforcement, in.
S_d = environmental durability factor. See 9.2.6
s = spacing of shear or torsion reinforcement measured in a direction parallel to longitudinal reinforcement, in.
s_1 = spacing of vertical reinforcement in wall, in.
s_2 = spacing of shear or torsion reinforcement measured in a direction perpendicular to longitudinal reinforcement—or spacing of horizontal reinforcement in wall, in.
t = thickness of a wall of a hollow section, in.
T_n = nominal torsional moment strength, in.-lb
T_u = factored torsional moment at section, in.-lb
V_c = nominal shear strength provided by concrete, lb
V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, lb
V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web, lb
V_d = shear force at section due to unfactored dead load, lb
V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}, lb
V_n = nominal shear strength, lb

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V_p = vertical component of effective prestress force at section, lb

V_s = nominal shear strength provided by shear reinforcement, lb

V_u = factored shear force at section, lb

v_n = nominal shear stress, psi. See 11.12.6.2

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, in.

α = angle between inclined stirrups and longitudinal axis of member

α_f = angle between shear-friction reinforcement and shear plane

α_s = constant used to compute V_c in slabs and footings

α_v = ratio of flexural stiffness of shearhead arm to that of surrounding composite slab section. See 11.12.4.5

β_c = ratio of long side to short side of concentrated load or reaction area

β_p = constant used to compute V_c in prestressed slabs

γ_f = fraction of unbalanced moment transferred by flexure at slab-column connections. See 13.5.3.2

γ_v = fraction of unbalanced moment transferred by eccentricity of shear at slab-column connections. See 11.12.6.1

= $1 - \gamma_f$

η = number of identical arms of shearhead

θ = angle of compression diagonals in truss analogy for torsion

λ = correction factor related to unit weight of concrete

μ = coefficient of friction. See 11.7.4.3

ρ = ratio of nonprestressed tension reinforcement

= A_s/bd

ρ_h = ratio of horizontal shear reinforcement area to gross concrete area of vertical section

ρ_n = ratio of vertical shear reinforcement area to gross concrete area of horizontal section

ρ_w = A_s/b_wd

ϕ = strength reduction factor. See 9.3

11.1 — Shear strength

11.1.1 — Design of cross sections subject to shear shall be based on

$$\phi V_n \geq V_u \tag{11-1}$$

where V_u is factored shear force at section considered and V_n is nominal shear strength computed by

$$V_n = V_c + V_s/S_d \tag{11-2}$$

where V_c is nominal shear strength provided by concrete in accordance with 11.3 or 11.4, and V_s is

COMMENTARY

R11.1 — Shear strength

The shear strength is based on an average shear stress on the full effective cross section b_wd. In a member without shear reinforcement, shear is assumed to be carried by the concrete web. In a member with shear reinforcement, a portion of the shear is assumed to be provided by the concrete and the remainder by the shear reinforcement.

The shear strength provided by concrete V_c is assumed to be the same for beams with and without shear reinforcement and is taken as the shear causing significant inclined cracking. These assumptions are discussed in the References 11.1, 11.2, and 11.3.

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nominal shear strength provided by shear reinforcement in accordance with 11.5.6.

11.1.1.1 — In determining shear strength V_n, effect of any openings in members shall be considered.

11.1.1.2 — In determining shear strength V_c, whenever applicable, effects of axial tension due to creep and shrinkage in restrained members shall be considered and effects of inclined flexural compression in variable depth members shall be permitted to be included.

11.1.2 — The values of $\sqrt{f_c'}$ used in this chapter shall not exceed 100 psi except as allowed in 11.1.2.1.

11.1.2.1 — Values of $\sqrt{f_c'}$ greater than 100 psi shall be permitted in computing V_c, V_{ci}, and V_{cw} for reinforced or prestressed concrete beams and concrete joist construction having minimum web reinforcement in accordance with 11.5.5.3, 11.5.5.4, or 11.6.5.2.

11.1.3 — Computation of maximum factored shear force V_u at supports in accordance with 11.1.3.1 or 11.1.3.2 shall be permitted if all conditions (a), (b), and (c) are satisfied:

(a) Support reaction, in direction of applied shear, introduces compression into the end regions of member;

(b) Loads are applied at or near the top of the member;

(c) No concentrated load occurs between face of support and location of critical section defined in 11.1.3.1 or 11.1.3.2.

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R11.1.1.1 — Openings in the web of a member can reduce its shear strength. The effects of openings are discussed in Section 4.7 of Reference 11.1 and in References 11.4 and 11.5.

R11.1.1.2 — In a member of variable depth, the internal shear at any section is increased or decreased by the vertical component of the inclined flexural stresses. Computation methods are outlined in various textbooks and in the 1940 Joint Committee Report.[11.6]

R11.1.2 — Because of a lack of test data and practical experience with concretes having compressive strengths greater than 10,000 psi, the 1989 edition of the 318 code imposed a maximum value of 100 psi on $\sqrt{f_c'}$ for use in the calculation of shear strength of concrete beams, joists, and slabs. Exceptions to the limit were permitted in beams and joists when the transverse reinforcement satisfied an increased value for the minimum amount of web reinforcement. There are limited test data on the two-way shear strength of high-strength concrete slabs. Until more experience is obtained for two-way slabs built with concretes that have strengths greater than 10,000 psi, it is prudent to limit $\sqrt{f_c'}$ to 100 psi for the calculation of shear strength.

R11.1.2.1 — Based on the test results in References 11.7, 11.8, 11.9, 11.10, and 11.11, an increase in the minimum amount of transverse reinforcement is required for high-strength concrete. These tests indicated a reduction in the reserve shear strength as $\sqrt{f_c'}$ increased in beams reinforced with the specified minimum amount of transverse reinforcement, which is equivalent to an effective shear stress of 50 psi. A provision introduced in the 318-89 code required an increase in the minimum amount of transverse reinforcement for concrete strengths between 10,000 and 15,000 psi. This provision, which led to a sudden increase in the minimum amount of transverse reinforcement at a compressive strength of 10,000 psi, has been replaced by a gradual increase in the minimum A_v as $\sqrt{f_c'}$ increases, as given by Eq. (11-13).

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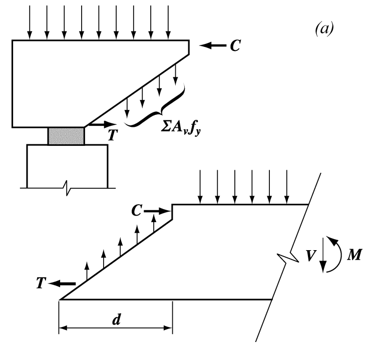

Fig. R11.1.3.1(a)—Free body diagrams of the end of a beam.

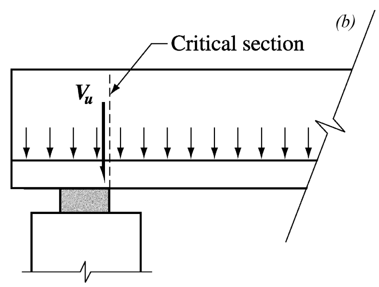

Fig. R11.1.3.1(b)—Location of critical section for shear in a member loaded near bottom.

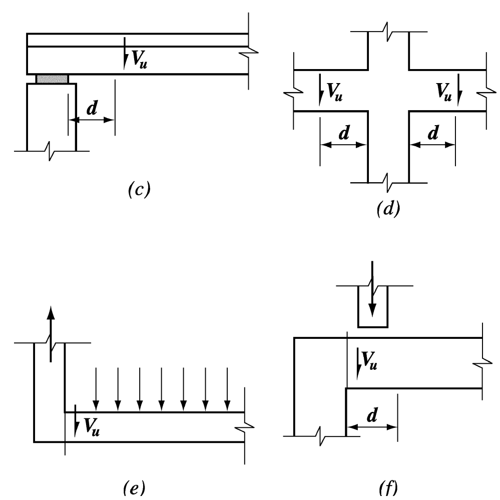

Fig. R11.1.3.1(c), (d), (e), (f)—Typical support conditions for locating factored shear force $\mathbf{V_u}$.

11.1.3.1 — For nonprestressed members, sections located less than a distance ***d*** from face of support shall be permitted to be designed for the same shear V_u as that computed at a distance ***d***.

R11.1.3.1 — The closest inclined crack to the support of the beam in Fig. R11.1.3.1(a) will extend upwards from the face of the support reaching the compression zone about ***d*** from the face of the support. If loads are applied to the top of this beam, the stirrups across this crack are stressed by loads acting on the lower freebody in Fig. R11.1.3.1(a). The loads applied to the beam between the face of the column and the point ***d*** away from the face are transferred directly to the support by compression in the web above the crack. Accordingly, the code permits design for a maximum factored shear force V_u at a distance ***d*** from the support for nonprestressed members, and at a distance ***h*/2** for prestressed members.

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Two things must be emphasized: first, stirrups are required across the potential crack designed for the shear at d from the support, and second, a tension force exists in the longitudinal reinforcement at the face of the support.

In Fig. 11.1.3.1(b), loads are shown acting near the bottom of a beam. In this case, the critical section is taken at the face of the support. Loads acting near the support should be transferred across the inclined crack extending upward from the support face. The shear force acting on the critical section include all loads applied below the potential inclined crack.

Typical support conditions where the shear force at a distance d from the support may be used include: (1) members supported by bearing at the bottom of the member, such as shown in Fig. R11.1.3.1(c); and (2) members framing monolithically into another member, as illustrated in Fig. R11.1.3.1(d).

Support conditions where this provision should not be applied include:

(1) Members framing into a supporting member in tension, such as shown in Fig. R11.1.3.1(e). For this case, the critical section for shear should be taken at the face of the support, shear within the connection should also be investigated, and special corner reinforcement should be provided;

(2) Members for which loads are not applied at or near the top of the member. This is the condition referred to in Fig. 11.1.3.1(b). For such cases, the critical section is taken at the face of the support. Loads acting near the support should be transferred across the inclined crack extending upward from the support face. The shear force acting on the critical section should include all loads below the potential inclined crack;

(3) Members loaded such that the shear at sections between the support and a distance d differs radically from the shear at distance d. This commonly occurs in brackets and in beams where a concentrated load is located close to the support, as shown in Fig. R11.1.3.1(f) or in footings supported on piles. In this case, the shear at the face of the support should be used.

11.1.3.2 — For prestressed members, sections located less than a distance $h/2$ from face of support shall be permitted to be designed for the same shear V_u as that computed at a distance $h/2$.

R11.1.3.2 — Because d frequently varies in prestressed members the location of the critical section has arbitrarily been taken as $h/2$ from the face of the support.

11.1.4 — For deep beams, brackets and corbels, walls, and slabs and footings, the special provisions of 11.8 through 11.12 shall apply.

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11.2 — Lightweight concrete

11.2.1 — Provisions for shear and torsion strength apply to normalweight concrete. When lightweight aggregate concrete is used, one of the following modifications shall apply to $\sqrt{f'_c}$ throughout Chapter 11, except 11.5.4.3, 11.5.6.9, 11.6.3.1, 11.12.3.2, and 11.12.4.8.

11.2.1.1 — When f_{ct} is specified and concrete is proportioned in accordance with 5.2, $f_{ct}/6.7$ shall be substituted for $\sqrt{f_c'}$ but the value of $f_{ct}/6.7$ shall not exceed $\sqrt{f_c'}$.

11.2.1.2 — When f_{ct} is not specified, all values of $\sqrt{f_c'}$ shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation shall be permitted when partial sand replacement is used.

11.3 — Shear strength provided by concrete for nonprestressed members

11.3.1 — Shear strength V_c shall be computed by provisions of 11.3.1.1 through 11.3.1.3, unless a more detailed calculation is made in accordance with 11.3.2.

11.3.1.1 — For members subject to shear and flexure only

$$V_c = 2\sqrt{f_c'}\, b_w d \qquad (11\text{-}3)$$

11.3.1.2 — For members subject to axial compression

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f_c'}\, b_w d \qquad (11\text{-}4)$$

Quantity N_u/A_g shall be expressed in psi.

11.3.1.3 — For members subject to significant axial tension, shear reinforcement shall be designed to carry total shear unless a more detailed analysis is made using 11.3.2.3.

11.3.2 — Shear strength V_c shall be permitted to be computed by the more detailed calculation of 11.3.2.1 through 11.3.2.3.

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R11.2 — Lightweight concrete

Two alternate procedures are provided to modify the provisions for shear when lightweight aggregate concrete is used. The lightweight concrete modification applies only to the terms containing $\sqrt{f_c'}$ in the equations of Chapter 11.

R11.2.1.1 — The first alternate bases the modification on laboratory tests to determine the relationship between splitting tensile strength f_{ct} and the compressive strength f_c' for the lightweight concrete being used. For normal weight concrete, the splitting tensile strength f_{ct} is approximately equal to $6.7\sqrt{f_c'}$.[11.9,11.13]

R11.2.1.2 — The modification may also be based on the assumption that the tensile strength of lightweight concrete is a fixed fraction of the tensile strength of normalweight concrete.[11.12] The multipliers are based on data from tests[11.13] on many types of structural lightweight aggregate concrete.

R11.3 — Shear strength provided by concrete for nonprestressed members

R11.3.1.1 — See R11.3.2.1.

R11.3.1.2 and R11.3.1.3 — See R11.3.2.2.

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11.3.2.1 — For members subject to shear and flexure only

$$V_c = \left(1.9\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d \qquad (11\text{-}5)$$

but not greater than $3.5\sqrt{f_c'}\, b_w d$. Quantity $V_u d/M_u$ shall not be taken greater than 1.0 in computing V_c by Eq. (11-5), where M_u is factored moment occurring simultaneously with V_u at section considered.

11.3.2.2 — For members subject to axial compression, it shall be permitted to compute V_c using Eq. (11-5) with M_m substituted for M_u and $V_u d/M_u$ not then limited to 1.0, where

$$M_m = M_u - N_u \frac{(4h - d)}{8} \qquad (11\text{-}6)$$

However, V_c shall not be taken greater than

$$V_c = 3.5\sqrt{f_c'}\, b_w d \sqrt{1 + \frac{N_u}{500 A_g}} \qquad (11\text{-}7)$$

Quantity N_u/A_g shall be expressed in psi. When M_m as computed by Eq. (11-6) is negative, V_c shall be computed by Eq. (11-7).

11.3.2.3 — For members subject to significant axial tension

$$V_c = 2\left(1 + \frac{N_u}{500 A_g}\right)\sqrt{f_c'}\, b_w d \qquad (11\text{-}8)$$

but not less than zero, where N_u is negative for tension. Quantity N_u/A_g shall be expressed in psi.

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R11.3.2.1 — Equation (11-5) is the basic expression for shear strength of members without shear reinforcement.[11.3] Designers should recognize that the three variables in Eq. (11-5), $\sqrt{f_c'}$ (as a measure of concrete tensile strength), ρ_w, and $V_u d/M_u$, are known to affect shear strength, although some research data[11.1,11.14] indicate that Eq. (11-5) overestimates the influence of f_c' and underestimates the influence of ρ_w and $V_u d/M_u$. Further information[11.15] has indicated that shear strength decreases as the overall depth of the member increases.

The minimum value of M_u equal to $V_u d$ in Eq. (11-5) is to limit V_c near points of inflection. For most designs, it is convenient to assume that the second term of Eq. (11-5) equals $0.1\sqrt{f_c'}$ and use V_c equal to $2\sqrt{f_c'}\, b_w d$ as permitted in 11.3.1.1.

R11.3.2.2 — Equation (11-6) and (11-7) for members subject to axial compression in addition to shear and flexure are derived in the ACI-ASCE Committee 326 report.[11.3] As N_u is increased, the value of V_c computed from Eq. (11-5) and (11-6) will exceed the upper limit given by Eq. (11-7) before the value of M_m given by Eq. (11-6) becomes negative. The value of V_c obtained from Eq. (11-5) has no physical significance if a negative value of M_m is substituted. For this condition, Equation (11-7) or (11-4) should be used to calculate V_c. Values of V_c for members subject to shear and axial load are illustrated in Fig. R11.3.2.2. The background for these equations is discussed, and comparisons are made with test data in Reference 11.2.

Because of the complexity of Eq. (11-5) and (11-6), an alternative design provision, Eq. (11-4), is permitted.

R11.3.2.3 — Equation (11-8) may be used to compute V_c for members subject to significant axial tension. Shear reinforcement may then be designed for $V_n - V_c$. The term "significant" is used to recognize that a designer must use judgment in deciding whether axial tension needs to be considered. Low levels of axial tension often occur due to volume changes, but are not important in structures with

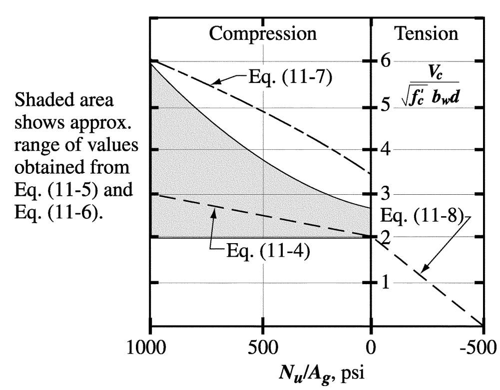

Fig. R11.3.2.2—Comparison of shear strength equations for members subjected to axial load.

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COMMENTARY

adequate expansion joints and minimum reinforcement. It may be desirable to design shear reinforcement to carry total shear if there is uncertainty about the magnitude of axial tension.

11.3.3 — For circular members, the area used to compute V_c shall be taken as the product of the diameter and effective depth of the concrete section. It shall be permitted to take the effective depth as 0.8 times the diameter of the concrete section.

R11.3.3 — Shear tests of members with circular sections indicate that the effective area can be taken as the gross area of the section or as an equivalent rectangular area.[11.1,11.6,11.7]

11.4 — Shear strength provided by concrete for prestressed members

11.4.1 — For members with effective prestress force not less than 40 percent of the tensile strength of flexural reinforcement, unless a more detailed calculation is made in accordance with 11.4.2

$$V_c = \left(0.6\sqrt{f_c'} + 700\frac{V_u d}{M_u}\right)b_w d \qquad (11\text{-}9)$$

but V_c need not be taken less than $2\sqrt{f_c'}\, b_w d$ nor shall V_c be taken greater than $5\sqrt{f_c'}\, b_w d$ nor the value given in 11.4.3 or 11.4.4. The quantity $V_u d/M_u$ shall not be taken greater than 1.0, where M_u is factored moment occurring simultaneously with V_u at section considered. When applying Eq. (11-9), d in the term $V_u d/M_u$ shall be the distance from extreme compression fiber to centroid of prestressed reinforcement.

11.4.2 — Shear strength V_c shall be permitted to be computed in accordance with 11.4.2.1 and 11.4.2.2, where V_c shall be the lesser of V_{ci} or V_{cw}.

11.4.2.1 — Shear strength V_{ci} shall be computed by

$$V_{ci} = 0.6\sqrt{f_c'}\, b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad (11\text{-}10)$$

but V_{ci} need not be taken less than $1.7\sqrt{f_c'}\, b_w d$, where

$$M_{cr} = (I/y_t)(6\sqrt{f_c'} + f_{pe} - f_d) \qquad (11\text{-}11)$$

and values of M_{max} and V_i shall be computed from the load combination causing maximum moment to occur at the section.

11.4.2.2 — Shear strength V_{cw} shall be computed by

$$V_{cw} = (3.5\sqrt{f_c'} + 0.3 f_{pc}) b_w d + V_p \qquad (11\text{-}12)$$

Alternatively, V_{cw} shall be computed as the shear force corresponding to dead load plus live load that results

R11.4 — Shear strength provided by concrete for prestressed members

R11.4.1 — Equation (11-9) offers a simple means of computing V_c for prestressed concrete beams.[11.2] It may be applied to beams having prestressed reinforcement only, or to members reinforced with a combination of prestressed reinforcement and nonprestressed deformed bars. Equation (11-9) is most applicable to members subject to uniform loading, and may give conservative results when applied to composite girders for bridges.

In applying Eq. (11-9) to simply supported members subject to uniform loads $V_u d/M_u$ can be expressed as

$$\frac{V_u d}{M_u} = \frac{d(\ell - 2_x)}{x(\ell - x)}$$

where ℓ is the span length and x is the distance from the section being investigated to the support. For concrete with f_c' equal to 5000 psi, V_c from 11.4.1 varies as shown in Fig. R11.4.1. Design aids based on this equation are given in Reference 11.18.

R11.4.2 — Two types of inclined cracking occur in concrete beams: web-shear cracking and flexure-shear cracking. These two types of inclined cracking are illustrated in Fig. R11.4.2.

Web-shear cracking begins from an interior point in a member when the principal tensile stresses exceed the tensile strength of the concrete. Flexure-shear cracking is initiated by flexural cracking. When flexural cracking occurs, the shear stresses in the concrete above the crack are increased. The flexure-shear crack develops when the combined shear and tensile stress exceeds the tensile strength of the concrete.

Equation (11-10) and (11-12) may be used to determine the shear forces causing flexure-shear and web-shear cracking, respectively. The shear strength provided by the concrete V_c is assumed equal to the lesser of V_{ci} and V_{cw}. The derivations of Eq. (11-10) and (11-12) are summarized in Reference 11.19.

In deriving Eq. (11-10) it was assumed that V_{ci} is the sum of the shear required to cause a flexural crack at the point in question given by

$$V = \frac{V_i M_{cr}}{M_{max}}$$

CODE

in a principal tensile stress of $4\sqrt{f_c'}$ at the centroidal axis of member, or at the intersection of flange and web when the centroidal axis is in the flange. In composite members, the principal tensile stress shall be computed using the cross section that resists live load.

11.4.2.3 — In Eq. (11-10) and (11-12), d shall be the distance from extreme compression fiber to centroid of prestressed reinforcement or $0.8h$, whichever is greater.

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plus an additional increment of shear required to change the flexural crack to a flexure-shear crack. The externally applied factored loads, from which V_i and M_{max} are determined, include superimposed dead load, earth pressure, live load, etc. In computing M_{cr} for substitution into Eq. (11-10), I and y_t are the properties of the section resisting the externally applied loads.

For a composite member, where part of the dead load is resisted by only a part of the section, appropriate section properties should be used to compute f_d. The shear due to dead loads, V_d and that due to other loads V_i are separated in this case. V_d is then the total shear force due to unfactored dead load acting on that part of the section carrying the dead loads acting prior to composite action plus the unfactored superimposed dead load acting on the composite member. The terms V_i and M_{max} may be taken as

$$V_i = V_u - V_d$$

$$M_{max} = M_u - M_d$$

where V_u and M_u are the factored shear and moment due to the total factored loads, and M_d is the moment due to unfactored dead load (that is, the moment corresponding to f_d).

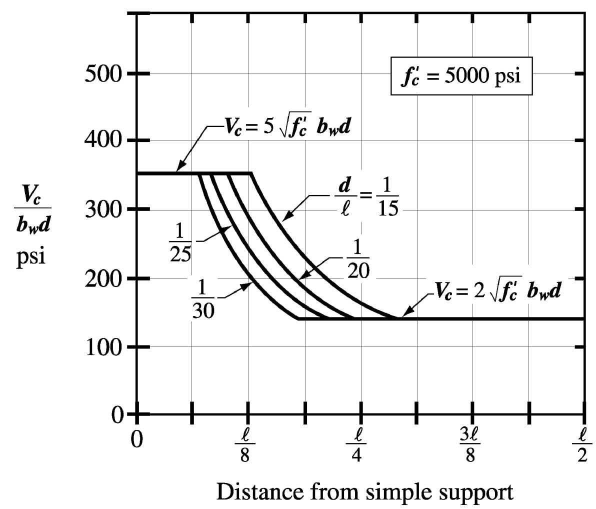

Fig. R11.4.1—Application of Eq. (11-9) to uniformly loaded prestressed members.

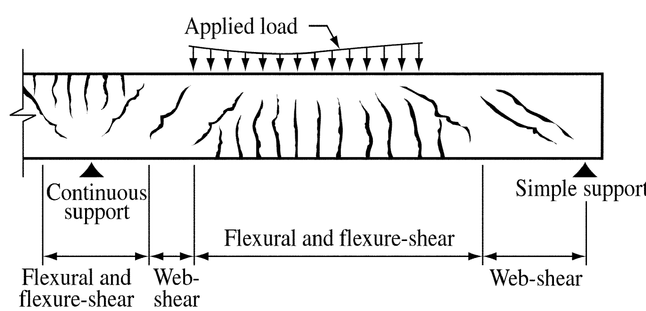

Fig. R11.4.2—Types of cracking in concrete beams.

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For noncomposite uniformly loaded beams, the total cross section resists all the shear, and the live and dead load shear force diagrams are similar. In this case Eq. (11-10) reduces to

$$V_{ci} = 0.6\sqrt{f_c'}b_w d + \frac{V_u M_{ci}}{M_u}$$

where

$$M_{ct} = (I/y_t)(6\sqrt{f_c'} + f_{pe})$$

The symbol M_{ct} in the two preceding equations represents the total moment, including dead load, required to cause cracking at the extreme fiber in tension. This is not the same as M_{cr} in code Eq. (11-10) where the cracking moment is that due to all loads except the dead load. In Eq. (11-10), the dead load shear is added as a separate term.

M_u is the factored moment on the beam at the section under consideration, and V_u is the factored shear force occurring simultaneously with M_u. Because the same section properties apply to both dead and live load stresses, there is no need to compute dead load stresses and shears separately, and the cracking moment M_{ct} reflects the total stress change from effective prestress to a tension of $6\sqrt{f_c'}$, assumed to cause flexural cracking.

Equation (11-12) is based on the assumption that web-shear cracking occurs due to the shear causing a principal tensile stress of approximately $4\sqrt{f_c'}$ at the centroidal axis of the cross section. V_p is calculated from the effective prestress force without load factors.

11.4.3 — In a pretensioned member in which the section at a distance ***h*/2** from face of support is closer to the end of member than the transfer length of the prestressing steel, the reduced prestress shall be considered when computing V_{cw}. This value of V_{cw} shall also be taken as the maximum limit for Eq. (11-9). The prestress force shall be assumed to vary linearly from zero at end of the prestressing steel to a maximum at a distance from end of the prestressing steel equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

R11.4.3 and R11.4.4 — The effect of the reduced prestress near the ends of pretensioned beams on the shear strength must be taken into account. Section 11.4.3 relates to the shear strength at sections within the transfer length of prestressing steel when bonding of prestressing steel extends to the end of the member.

Section 11.4.4 relates to the shear strength at sections within the length over which some of the prestressing steel is not bonded to the concrete, or within the transfer length of the prestressing steel for which bonding does not extend to the end of the beam.

11.4.4 — In a pretensioned member where bonding of some tendons does not extend to the end of member, a reduced prestress shall be considered when computing V_c in accordance with 11.4.1 or 11.4.2. The value of V_{cw} calculated using the reduced prestress shall also be taken as the maximum limit for Eq. (11-9). The prestress force due to tendons for which bonding does not extend to the end of member, shall be assumed to vary linearly from zero at the point at which bonding commences to a maximum at a distance from this point equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

CODE

11.5 — Shear strength provided by shear reinforcement

11.5.1 — Types of shear reinforcement

11.5.1.1 — Shear reinforcement consisting of the following shall be permitted:

(a) Stirrups perpendicular to axis of member;

(b) Welded wire fabric with wires located perpendicular to axis of member;

(c) Spirals, circular ties, or hoops.

11.5.1.2 — For nonprestressed members, shear reinforcement shall be permitted to also consist of:

(a) Stirrups making an angle of 45 deg or more with longitudinal tension reinforcement;

(b) Longitudinal reinforcement with bent portion making an angle of 30 deg or more with the longitudinal tension reinforcement;

(c) Combinations of stirrups and bent longitudinal reinforcement;

(d) Spirals.

11.5.2 — Design yield strength of shear reinforcement shall not exceed 60,000 psi, except that the design yield strength of welded deformed wire fabric shall not exceed 80,000 psi.

11.5.3 — Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fiber and shall be anchored at both ends according to 12.13 to develop the design yield strength of reinforcement.

11.5.4 — Spacing limits for shear reinforcement

11.5.4.1 — Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/2$ in nonprestressed members and $(3/4)h$ in prestressed members.

11.5.4.2 — Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45 deg line, extending toward the reaction from mid-depth of member $d/2$ to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

COMMENTARY

R11.5 — Shear strength provided by shear reinforcement

R11.5.2 — Limiting the design yield strength of shear reinforcement to 60,000 psi provides a control on diagonal crack width. In the ACI 318-95 code, however, the limitation on design yield strength of 60,000 psi for shear reinforcement was raised to 80,000 psi for welded deformed wire fabric. Recent research[11.20,11.21,11.22] has indicated that the performance of higher-strength steels as shear reinforcement has been satisfactory. In particular, full-scale beam tests described in Reference 11.21 indicated that the widths of inclined shear cracks at service load levels were less for beams reinforced with smaller-diameter deformed welded wire fabric cages designed on the basis of a yield strength of 75 ksi than beams reinforced with deformed Grade 60 stirrups.

R11.5.3 — It is essential that shear (and torsion) reinforcement be adequately anchored at both ends, to be fully effective on either side of any potential inclined crack. This generally requires a hook or bend at the end of the reinforcement as provided by 12.13.

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11.5.4.3 — When V_s exceeds $4\sqrt{f_c'}\, b_w d$, maximum spacings given in 11.5.4.1 and 11.5.4.2 shall be reduced by one-half.

11.5.4.4 — Spacing of shear reinforcement shall not exceed 12 in.

11.5.5 — Minimum shear reinforcement

11.5.5.1 — A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where factored shear force V_u exceeds one-half the shear strength provided by concrete ϕV_c, except:

(a) Slabs and footings;

(b) Concrete joist construction defined by 8.11;

(c) Beams with total depth not greater than 10 in., 2-1/2 times thickness of flange, or 1/2 the width of web, whichever is greatest.

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R11.5.4.4 — A maximum spacing of 12 in. for reinforcement is for crack control.

R11.5. 5 — Minimum shear reinforcement

R11.5.5.1 — Shear reinforcement restrains the growth of inclined cracking. Ductility is increased, and a warning of failure is provided. In an unreinforced web, the sudden formation of inclined cracking might lead directly to failure without warning. Such reinforcement is of great value if a member is subjected to an unexpected tensile force or an overload. Accordingly, a minimum area of shear reinforcement not less than that given by Eq. (11-13) or (11-14) is required wherever the total factored shear force V_u is greater than one-half the shear strength provided by concrete ϕV_c. Slabs, footings, and joists are excluded from the minimum shear reinforcement requirement because there is a possibility of load sharing between weak and strong areas. Research results,[11.23] however, have shown that deep, lightly reinforced one-way slabs, particularly if constructed with high-strength concrete, may fail at shear loads less than V_c calculated from Eq. (11-3).

Even when the total factored shear strength V_u is less than one-half of the shear strength provided by the concrete ϕV_c, the use of some web reinforcement is recommended in all thin-web post-tensioned prestressed concrete members (joists, waffle slabs, beams, and T-beams) to reinforce against tensile forces in webs resulting from local deviations from the design tendon profile, and to provide a means of supporting the tendons in the design profile during construction. If sufficient support is not provided, lateral wobble and local deviations from the smooth parabolic tendon profile assumed in design may result during placement of the concrete. In such cases, the deviations in the tendons tend to straighten out when the tendons are stressed. This process may impose large tensile stresses in webs, and severe cracking may develop if no web reinforcement is provided. Unintended curvature of the tendons, and the resulting tensile stresses in webs, may be minimized by securely tying tendons to stirrups that are rigidly held in place by other elements of the reinforcing cage and held down in the forms. The maximum spacing of stirrups used for this purpose should not exceed the smaller of $1.5h$ or 4 ft. When applicable, the shear reinforcement provisions of 11.5.4 and 11.5.5 will require closer stirrup spacings.

For repeated loading of flexural members, the possibility of inclined diagonal tension cracks forming at stresses appreciably smaller than under static loading should be taken into account in the design. In these instances, it would be prudent to use at least the minimum shear reinforcement expressed by Eq. (11-13) or (11-14), even though tests or calculations based on static loads show that shear reinforcement is not required.

CODE

11.5.5.2 — Minimum shear reinforcement requirements of 11.5.5.1 shall be permitted to be waived if shown by test that required nominal flexural and shear strengths can be developed when shear reinforcement is omitted. Such tests shall simulate effects of differential settlement, creep, shrinkage, and temperature change, based on a realistic assessment of such effects occurring in service.

11.5.5.3 — Where shear reinforcement is required by 11.5.5.1 or for strength and where 11.6.1 allows torsion to be neglected, the minimum area of shear reinforcement for prestressed (except as provided in 11.5.5.4) and nonprestressed members shall be computed by

$$A_v = 0.75\sqrt{f'_c}\frac{b_w s}{f_y} \qquad (11\text{-}13)$$

but shall not be less than $(50 b_w s)/f_y$ where b_w and s are in inches.

11.5.5.4 — For prestressed members with an effective prestress force not less than 40 percent of the tensile strength of the flexural reinforcement, the area of shear reinforcement shall not be less than the smaller A_v from Eq. (11-13) and (11-14).

$$A_v = \frac{A_{ps} f_{pu} s}{80 f_y d}\sqrt{\frac{d}{b_w}} \qquad (11\text{-}14)$$

11.5.6 — Design of shear reinforcement

COMMENTARY

R11.5.5.2 — When a member is tested to demonstrate that its shear and flexural strengths are adequate, the actual member dimensions and material strengths are known. The strength used as a basis for comparison should therefore be that corresponding to a strength reduction factor of unity ($\phi = 1.0$), that is, the required nominal strength V_n and M_n. This ensures that if the actual material strengths in the field were less than specified, or the member dimensions were in error such as to result in reduced member strength, a satisfactory margin of safety will be retained.

R11.5.5.3 — Previous editions of the code have required a minimum area of transverse reinforcement that is independent of concrete strength. Tests[11.9] have indicated the need to increase the minimum area of shear reinforcement as concrete strength increases to prevent sudden shear failures when inclined cracking occurs. Equation (11-3) provides for a gradual increase in the minimum area of transverse reinforcement, while maintaining the previous minimum value.

R11.5.5.4 — Tests of prestressed beams with minimum web reinforcement based on Eq. (11-13) and (11-14) indicated that the smaller A_v from these two equations was sufficient to develop ductile behavior.

Equation (11-14) may be used only for prestressed members meeting the minimum prestress force requirements given in 11.5.5.4. This equation is discussed in Reference 11.24.

R11.5.6 — Design of shear reinforcement

Design of shear reinforcement is based on a modified truss analogy. The truss analogy assumes that the total shear is carried by shear reinforcement. Considerable research on both nonprestressed and prestressed members, however, has indicated that shear reinforcement need be designed to carry only the shear exceeding that which causes inclined cracking, provided the diagonal members in the truss are assumed to be inclined at 45 deg.

Equation (11-15), (11-16), and (11-17) are presented in terms of shear strength V_s attributed to the shear reinforcement. When shear reinforcement perpendicular to axis of member is used, the required area of shear reinforcement A_v and its spacing s are computed by

$$\frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_y d}$$

Research[11.25,11.26] has shown that shear behavior of wide beams with substantial flexural reinforcement is improved if the transverse spacing of stirrup legs across the section is reduced.

CODE

11.5.6.1 — Where factored shear force V_u exceeds shear strength ϕV_c, shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where shear strength V_s shall be computed in accordance with 11.5.6.2 through 11.5.6.9.

11.5.6.2 — When shear reinforcement perpendicular to axis of member is used

$$V_s = \frac{A_v f_y d}{s} \tag{11-15}$$

where A_v is the area of shear reinforcement within a distance s.

11.5.6.3 — When circular ties, hoops, or spirals are used as shear reinforcement, V_s shall be computed using Eq. (11-15) where d shall be taken as the effective depth defined in 11.3.3. A_v shall be taken as two times the area of the bar in a circular tie, hoop, or spiral at a spacing s, and f_{yh} is the specified yield strength or circular tie, hoop, or spiral reinforcement.

11.5.6.4 — When inclined stirrups are used as shear reinforcement

$$V_s = \frac{A_v f_y (\sin\alpha + \cos\alpha) d}{s} \tag{11-16}$$

11.5.6.5 — When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support

$$V_s = A_v f_y \sin\alpha \tag{11-17}$$

but not greater than $3\sqrt{f_c'}\, b_w d$.

11.5.6.6 — When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, shear strength V_s shall be computed by Eq. (11-16).

11.5.6.7 — Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

11.5.6.8 — Where more than one type of shear reinforcement is used to reinforce the same portion of a member, shear strength V_s shall be computed as the sum of the V_s values computed for the various types.

11.5.6.9 — Shear strength V_s shall not be taken greater than $8\sqrt{f_c'}\, b_w d$.

11.6 — Design for torsion

COMMENTARY

R11.5.6.3 — Although the transverse reinforcement in a circular section may not consist of straight legs, tests indicate that Eq. (11-15) is conservative if d is taken as defined in 11.3.3.[11.16,11.17]

R11.6 — Design for torsion

In ACI 318-95, the design for torsion is based on a thin-walled tube, space truss analogy. A beam subjected to torsion is idealized as a thin-walled tube with the core concrete cross section in a solid beam neglected as shown in

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Fig. R11.6(a). Once a reinforced concrete beam has cracked in torsion, its torsional resistance is provided primarily by closed stirrups and longitudinal bars located near the surface of the member. In the thin-walled tube analogy, the resistance is assumed to be provided by the outer skin of the cross section roughly centered on the closed stirrups. Both hollow and solid sections are idealized as thin-walled tubes both before and after cracking.

In a closed thin-walled tube, the shear stresses τ due to torsion act as shown in Fig. R11.6(a). The product of the shear stress τ and the wall thickness t at any point in the perimeter is known as the shear flow, $q = \tau t$. The shear flow q due to torsion is constant at all points around the perimeter of the tube. The path along which it acts extends around the tube at midthickness of the walls of the tube. At any point along the perimeter of the tube, the shear stress due to torsion is $\tau = T/(2A_o t)$, where A_o is the gross area enclosed by the shear flow path, shown shaded in Fig. R11.6(b), and t is the thickness of the wall at the point where τ is being computed. The shear flow path follows the midplane of the walls of the tube, and A_o is the area enclosed by the midplane of the walls of the tube. For a hollow member with continuous walls, A_o includes the area of the hole.

In the ACI 318-95 code, the former elliptical interaction between the shear carried by the concrete V_c and the torsion carried by the concrete has been eliminated. V_c remains constant at the value it has when there is no torsion, and the torsion carried by the concrete is always taken as zero.

The design procedure is derived and compared with test results in References 11.27 and 11.28.

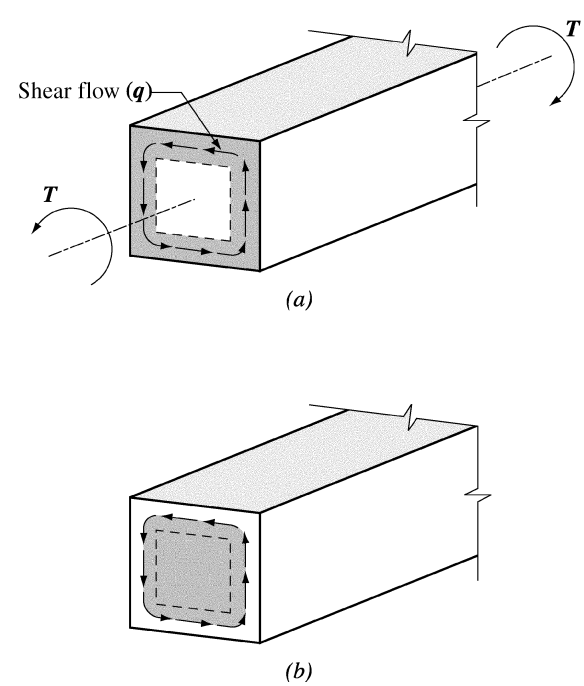

Fig. R11.6—(a) Thin-walled tube; and (b) area enclosed by shear flow path.

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11.6.1 — Threshold torsion

It shall be permitted to neglect torsion effects when the factored torsional moment T_u is less than:

(a) For nonprestressed members

$$\phi\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$$

(b) For prestressed members

$$\phi\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{f_{pc}}{4\sqrt{f_c'}}}$$

(c) For nonprestressed members subjected to an axial tensile or compressive force

$$\phi\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{4A_g\sqrt{f_c'}}}$$

For members cast monolithically with a slab, the overhanging flange width used in computing A_{cp} and p_{cp} shall conform to 13.2.4. For a hollow section, A_g shall be used in place of A_{cp} in 11.6.1, and the outer boundaries of the section shall conform to 13.2.4.

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R11.6.1 — Threshold torsion

Torques that do not exceed approximately one-quarter of the cracking torque T_{cr} will not cause a structurally significant reduction in either the flexural or shear strength, and hence can be ignored. The cracking torsion under pure torsion T_{cr} is derived by replacing the actual section with an equivalent thin-walled tube with a wall thickness t before cracking of $0.75A_{cp}/p_{cp}$ and an area enclosed by the wall centerline A_o equal to $2A_{cp}/3$. Cracking is assumed to occur when the principal tensile stress reaches $4\sqrt{f_c'}$. In a nonprestressed beam loaded with torsion alone, the principal tensile stress is equal to the torsional shear stress, $\tau = T/(2A_o t)$. Thus, cracking occurs when τ reaches $4\sqrt{f_c'}$, giving the cracking torque T_{cr} as

$$T_{cr} = 4\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$$

For solid members, the interaction between the cracking torsion and the inclined cracking shear is approximately circular or elliptical. For such a relationship, a torque of $0.25T_{cr}$, used in 11.6.1, corresponds to a reduction of 3% in the inclined cracking shear. This reduction in the inclined cracking shear was considered negligible. The stress at cracking $4\sqrt{f_c'}$ has purposely been taken as a lower-bound value.

For prestressed members, the torsional cracking load is increased by the prestress. A Mohr's circle analysis based on average stresses indicates the torque required to cause a principal tensile stress equal to $4\sqrt{f_c'}$ is $\sqrt{1+f_{pc}/(4\sqrt{f_c'})}$ times the corresponding torque in a nonprestressed beam. A similar modification is made in Part (c) of 11.6.1 for members subjected to axial load and torsion.

For torsion, a hollow member is defined as having one or more longitudinal voids, such as a single-cell or multiple-cell box girder. Small longitudinal voids, such as ungrouted post-tensioning ducts that result in A_g/A_{cp} greater than or equal to 0.95 can be ignored when computing the threshold torque in 11.6.1. The interaction between torsional cracking and shear cracking for hollow sections is assumed to vary from the elliptical relationship for members with small voids, to a straight-line relationship for thin-walled sections with large voids. For a straight-line interaction, a torque of $0.25T_{cr}$ would cause a reduction in the inclined cracking shear of about 25%. This reduction was judged to be excessive.

In the 2006 code, two changes were made to modify 11.6.1 to apply to hollow sections. First, the minimum torque limits from the 2001 code were multiplied by (A_g/A_{cp}) because tests of solid and hollow beams[11.29] indicate that the cracking torque of a hollow section is approximately (A_g/A_{cp}) times the cracking torque of a solid section with the same outside dimensions. The second change was to multiply the cracking torque by (A_g/A_{cp}) a second time to

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11.6.1.1 — For isolated members with flanges and for members cast monolithically with a slab, the overhanging flange width used in computing A_{cp} and p_{cp} shall conform to 13.2.4, except that the overhanging flanges shall be neglected in cases where the parameter A_{cp}^2/p_{cp} calculated for a beam with flanges is less than that computed for the same beam ignoring the flanges.

11.6.2 — Calculation of factored torsional moment T_u

11.6.2.1 — If the factored torsional moment T_u in a member is required to maintain equilibrium and exceeds the minimum value given in 11.6.1, the member shall be designed to carry that torsional moment in accordance with 11.6.3 through 11.6.6.

11.6.2.2 — In a statically indeterminate structure where reduction of the torsional moment in a member can occur due to redistribution of internal forces upon cracking, the maximum factored torsional moment T_u shall be permitted to be reduced to the values given in (a), (b), or (c), as applicable.

(a) For nonprestressed members, at the sections described in 11.6.2.4

$$\phi 4\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$$

(b) For prestressed members, at the sections described in 11.6.2.5

$$\phi 4\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{f_{pc}}{4\sqrt{f_c'}}}$$

(c) For nonprestressed members subjected to an axial tensile or compressive force:

$$\phi 4\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{4A_g\sqrt{f_c'}}}$$

In (a), (b), or (c), the correspondingly redistributed bending moments and shears in the adjoining members shall be used in the design of these members. For hollow sections, A_{cp} shall not be replaced with A_g in 11.6.2.2.

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reflect the transition from the circular interaction between the inclined cracking loads in shear and torsion for solid members, to the approximately linear interaction for thin-walled hollow sections.

R11.6.2 — Calculation of factored torsional moment T_u

R11.6.2.1 and R11.6.2.2 — In designing for torsion in reinforced concrete structures, two conditions may be identified:[11.30,11.31]

(a) The torsional moment cannot be reduced by redistribution of internal forces (11.6.2.1). This is referred to as "equilibrium torsion," because the torsional moment is required for the structure to be in equilibrium.

For this condition, illustrated in Fig. R11.6.2.1, torsion reinforcement designed according to 11.6.3 through 11.6.6 must be provided to resist the total design torsional moments.

(b) The torsional moment can be reduced by redistribution of internal forces after cracking (11.6.2.2) if the torsion arises from the member twisting to maintain compatibility of deformations. This type of torsion is referred to as "compatibility torsion."

For this condition, illustrated in Fig. R11.6.2.2, the torsional stiffness before cracking corresponds to that of the uncracked section according to St. Venant's theory. At torsional cracking, however, a large twist occurs under an essentially constant torque, resulting in a large redistribution of forces in the structure.[11.30,11.31] The cracking torque under combined shear, flexure, and torsion corresponds to a principle tensile stress somewhat less than the $4\sqrt{f_c'}$ quoted in R11.6.1.

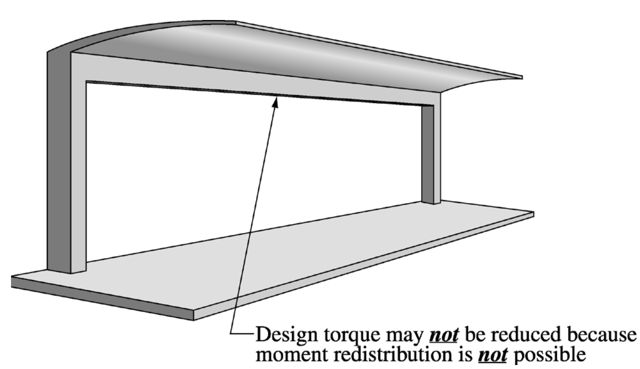

Fig. R11.6.2.1—Design torque may not be reduced (11.6.2.1).

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When the torsional moment exceeds the cracking torque, a maximum factored torsional moment equal to the cracking torque may be assumed to occur at the critical sections near the faces of the supports. This limit has been established to control the width of torsional cracks. The replacement of A_{cp} with A_g, as in the calculation of the threshold torque for hollow sections in 11.6.1, is not applied herein. Thus, the torque after redistribution is larger and hence more conservative.

Section 11.6.2.2 applies to typical and regular framing conditions. With layouts that impose significant torsional rotations within a limited length of the member, such as a heavy torque loading located close to a stiff column or a column that rotates in the reverse directions because of other loading, a more exact analysis is advisable.

When the factored torsional moment from an elastic analysis based on uncracked section properties is between the values in 11.6.1 and the values given in this section, torsion reinforcement should be designed to resist the computed torsional moments.

11.6.2.3 — Unless determined by a more exact analysis, it shall be permitted to take the torsional loading from a slab as uniformly distributed along the member.

11.6.2.4 — In nonprestressed members, sections located less than a distance d from the face of a support shall be designed for not less than the torsion T_u computed at a distance d. If a concentrated torque occurs within this distance, the critical section for design shall be at the face of the support.

11.6.2.5 — In prestressed members, sections located less than a distance $h/2$ from the face of a support shall be designed for not less than the torsion T_u computed at a distance $h/2$. If a concentrated torque occurs within this distance, the critical section for design shall be at the face of the support.

R11.6.2.4 and R11.6.2.5 — It is not uncommon for a beam to frame into one side of a girder near the support of the girder. In such a case, a concentrated shear and torque are applied to the girder.

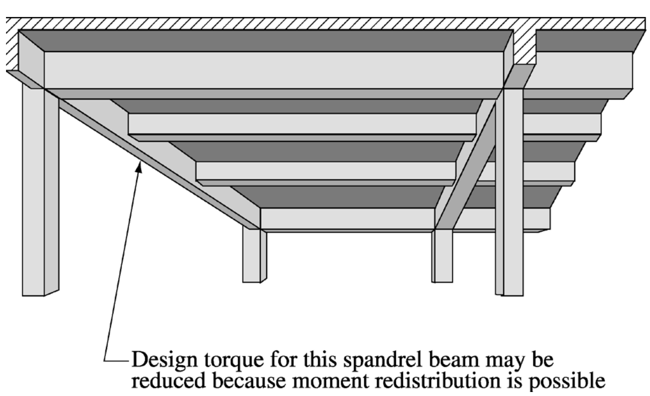

Fig. R11.6.2.2—Design torque may be reduced (11.6.2.2).

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11.6.3 — Torsional moment strength

11.6.3.1 — The cross-sectional dimensions shall be such that:

(a) For solid sections

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u p_h}{1.7A_{oh}^2}\right)^2} \le \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f_c'}\right) \qquad (11\text{-}18)$$

(b) For hollow sections

$$\left(\frac{V_u}{b_w d}\right) + \left(\frac{T_u p_h}{1.7A_{oh}^2}\right) \le \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f_c'}\right) \qquad (11\text{-}19)$$

11.6.3.2 — If the wall thickness varies around the perimeter of a hollow section, Eq. (11-19) shall be evaluated at the location where the left-hand side of Eq. (11-19) is a maximum.

11.6.3.3 — If the wall thickness is less than A_{oh}/p_h, the second term in Eq. (11-19) shall be taken as

$$\left(\frac{T_u}{1.7A_{oh}t}\right)$$

where t is the thickness of the wall of the hollow section at the location where the stresses are being checked.

COMMENTARY

R11.6.3 — Torsional moment strength

R11.6.3.1 —The size of a cross section is limited for two reasons: first, to reduce unsightly cracking, and second, to prevent crushing of the surface concrete due to inclined compressive stresses due to shear and torsion. In Eq. (11-18) and (11-19), the two terms on the left-hand side are the shear stresses due to shear and torsion. The sum of these stresses may not exceed the stress causing shear cracking plus $8\sqrt{f_c'}$, similar to the limiting strength given in 11.5.6.8 for shear without torsion. The limit is expressed in terms of V_c to allow its use for nonprestressed or prestressed concrete. It was originally derived on the basis of crack control. It is not necessary to check against crushing of the web because this happens at higher shear stresses.

In a hollow section, the shear stresses due to shear and torsion both occur in the walls of the box as shown in Fig. 11.6.3.1(a) and hence are directly additive at Point A as given in Eq. (11-19). In a solid section, the shear stresses due to torsion act in the "tubular" outside section, while the shear stresses due to V_u are spread across the width of the section as shown in Fig. R11.6.3.1(b). For this reason, stresses are combined in Eq. (11-18) using the square root of the sum of the squares rather than by direct addition.

R11.6.3.2 — If the wall thickness varies around the perimeter of a hollow section, 11.6.3.2 requires that Eq. (11-19) be evaluated at the point in the cross section where the left side of Eq. (11-19) is a maximum. Generally, this will be on the wall where the torsional and shearing stresses are additive (Point A in Fig. R11.6.3.1(a)). If the top or bottom flanges are thinner than the vertical webs, it may be necessary to evaluate Eq. (11-19) at Points B and C in Fig. R11.6.3.1(a). At these points, the stresses due to the shear force are usually negligible.

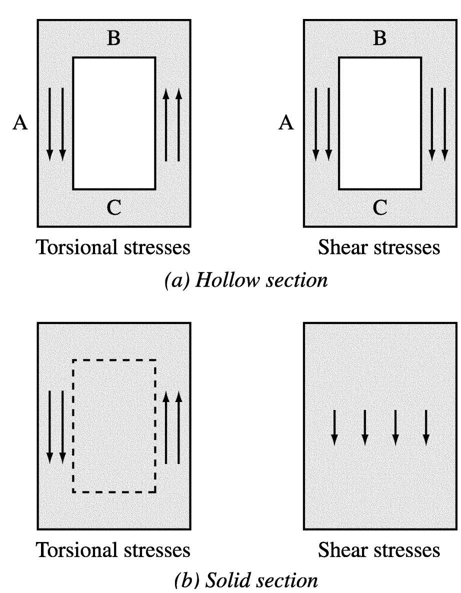

Fig. R11.6.3.1—Addition of torsional and shear stresses.

CODE

11.6.3.4 — Design yield strength of nonprestressed torsion reinforcement shall not exceed 60,000 psi.

11.6.3.5 — The reinforcement required for torsion shall be determined from

$$\phi T_n \geq T_u \qquad (11\text{-}20)$$

11.6.3.6 — The transverse reinforcement for torsion shall be designed using

$$T_n = \frac{2A_o A_t f_{yv}}{s}\cot\theta \qquad (11\text{-}21)$$

where A_o shall be determined by analysis except that it shall be permitted to take A_o equal to $0.85A_{oh}$; θ shall not be taken smaller than 30 deg nor larger than 60 deg. It shall be permitted to take θ equal to:

(a) 45 deg for nonprestressed members or members with less prestress than in (b);

(b) 37.5 deg for prestressed members with an effective prestress force not less than 40 percent of the tensile strength of the longitudinal reinforcement.

COMMENTARY

R11.6.3.4 — Limiting the design yield strength of torsion reinforcement to 60,000 psi provides a control on diagonal crack width.

R11.6.3.5 — The factored torsional resistance ϕT_n must equal or exceed the torsion T_u due to the factored loads. In the calculation of T_n, all the torque is assumed to be resisted by stirrups and longitudinal steel with $T_c = 0$. At the same time, the shear resisted by concrete V_c is assumed to be unchanged by the presence of torsion. For beams with V_u greater than about $0.8\phi V_c$ the resulting amount of combined shear and torsional reinforcement is essentially the same as required by the ACI 318-89 code. For smaller values of V_u, more shear and torsion reinforcement will be required.

R11.6.3.6 — Equation (11-21) is based on the space truss analogy shown in Fig. R11.6.3.6(a) with compression diagonals at an angle θ, assuming the concrete carries no tension and the reinforcement yields. After torsional cracking develops, the torsional resistance is provided mainly by closed stirrups, longitudinal bars, and compression diagonals. The concrete outside these stirrups is relatively ineffective. For this reason, A_o, the area enclosed by the shear flow path around the perimeter of the tube, is defined after cracking in terms of A_{oh}, the area enclosed by the centerline of the outermost closed hoops. The area A_{oh} is shown in Fig. R11.6.3.6(b) for various cross sections. In an I-, T-, or L-shaped section A_{oh} is taken as that area enclosed by the outermost legs of interlocking stirrups as shown in Fig. R11.6.3.6(b). The expression for A_o given by Hsu[11.32] may be used if greater accuracy is desired.

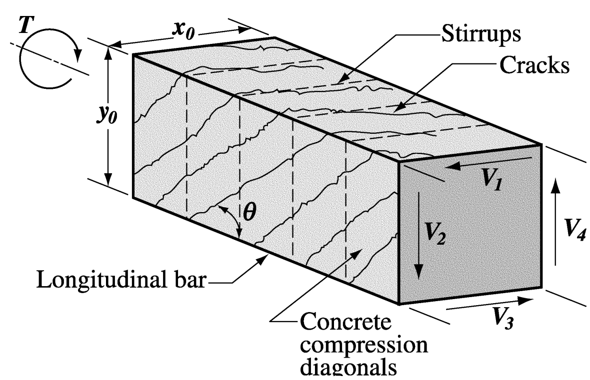

Fig. R11.6.3.6(a)—Space truss analogy.

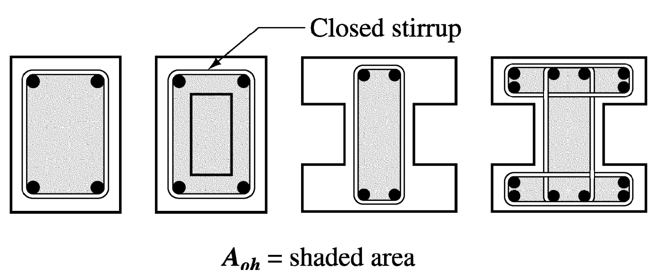

Fig. R11.6.3.6(b)—Definition of A_{oh}.

CODE

11.6.3.7 — The additional longitudinal reinforcement required for torsion shall not be less than

$$A_\ell = \frac{A_t}{s} p_h \left(\frac{f_{yv}}{f_{y\ell}}\right) \cot^2\theta \qquad (11\text{-}22)$$

where θ shall be the same value used in Eq. (11-21) and A_t/s shall be taken as the amount computed from Eq. (11-21) not modified in accordance with 11.6.5.2 or 11.6.5.3.

COMMENTARY

The shear flow q in the walls of the tube, discussed in R11.6, can be resolved into the shear forces V_1 to V_4 acting in the individual sides of the tube or space truss, as shown in Fig. R11.6.3.6(a).

The angle θ can be obtained by analysis[11.32] or may be taken to be equal to the values given in subsections (a) and (b). The same value of θ should be used in both Eq. (11-21) and (11-22). As θ gets smaller, the amount of stirrups required by Eq. (11-21) decreases. At the same time, the amount of longitudinal steel required by Eq. (11-22) increases.

R11.6.3.7 — Figure R11.6.3.6(a) shows the shear forces V_1 to V_4 resulting from the shear flow around the walls of the tube. On a given wall of the tube, the shear flow V_1 is resisted by a diagonal compression component, $D_i = V_i/\sin\theta$, in the concrete. An axial tension force, $N_i = V_i(\cot\theta)$, is needed in the longitudinal steel to complete the resolution of V_i.

Figure R11.6.3.7 shows the diagonal compressive stresses and the axial tension force N_i acting on a short segment along one wall of the tube. Because the shear flow due to torsion is constant at all points around the perimeter of the tube, the resultants of D_i and N_i act through the midheight of side i. As a result, half of N_i can be assumed to be resisted by each of the top and bottom chords as shown. Longitudinal reinforcement with a capacity of $A_\ell f_{y\ell}$ should be provided to resist the sum of the N_i forces, ΣN_i, acting in all of the walls of the tube.

In the derivation of Eq. (11-22), axial tension forces are summed along the sides of the area A_o. These sides form a perimeter length p_o approximately equal to the length of the line joining the centers of the bars in the corners of the tube. For ease in computation, this has been replaced with the perimeter of the closed stirrups p_h. Frequently, the maximum allowable stirrup spacing governs the amount of stirrups provided. Furthermore, when combined shear and torsion act, the total stirrup area is the sum of the amounts provided for shear and torsion. To avoid the need to provide excessive amounts of longitudinal reinforcement, 11.6.3.7 states that the A_t/s used in calculating A_ℓ at any given sections should be taken as the calculated A_t/s at that section using Eq. (11-21).

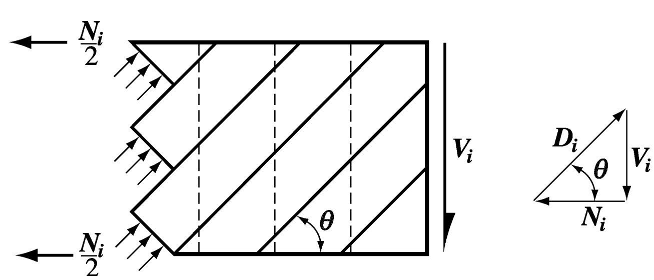

Fig. R11.6.3.7—Resolution of shear force $\mathbf{V_i}$ *into diagonal compression for* $\mathbf{D_i}$ *and axial tension force* $\mathbf{N_i}$ *in one wall of tube.*

CODE

11.6.3.8 — Reinforcement required for torsion shall be added to that required for the shear, moment, and axial force that act in combination with the torsion. The most restrictive requirements for reinforcement spacing and placement must be met.

11.6.3.9 — It shall be permitted to reduce the area of longitudinal torsion reinforcement in the flexural compression zone by an amount equal to $M_u/(0.9df_{y\ell})$, where M_u is the factored moment acting at the section in combination with T_u, except that the reinforcement provided shall not be less than that required by 11.6.5.3 or 11.6.6.2.

11.6.3.10 — In prestressed beams:

(a) The total longitudinal reinforcement including prestressing steel at each section shall resist the factored bending moment at that section plus an additional concentric longitudinal tensile force equal to $A_\ell f_{y\ell}$, based on the factored torsion at that section, and

(b) The spacing of the longitudinal reinforcement including tendons shall satisfy the requirements in 11.6.6.2.

COMMENTARY

R11.6.3.8 — The stirrup requirements for torsion and shear are added and stirrups are provided to supply at least the total amount required. Because the stirrup area A_v for shear is defined in terms of all the legs of a given stirrup while the stirrup area A_t for torsion is defined in terms of one leg only, the addition of stirrups is carried out as follows

$$\text{Total}\left(\frac{A_{v+t}}{s}\right) = \frac{A_v}{s} + 2\frac{A_t}{s}$$

If a stirrup group had four legs for shear, only the legs adjacent to the sides of the beam would be included in this summation because the inner legs would be ineffective for torsion.

The longitudinal reinforcement required for torsion is added at each section to the longitudinal reinforcement required for bending moment that acts at the same time as the torsion. The longitudinal reinforcement is then chosen for this sum, but should not be less than the amount required for the maximum bending moment at that section if this exceeds the moment acting at the same time as the torsion. If the maximum bending moment occurs at one section, say midspan, while the maximum torsional moment occurs at another, such as the support, the total longitudinal steel required may be less than that obtained by adding the maximum flexural steel plus the maximum torsional steel. In such a case, the required longitudinal steel is evaluated at several locations.

The most restrictive requirements for spacing, cut-off points, and placement for flexural, shear, and torsional steel must be satisfied. The flexural steel must be extended a distance d, but not less than $12d_b$, past where it is no longer needed for flexure as required in 12.10.3.

R11.6.3.9 — The longitudinal tension due to torsion is offset in part by the compression in the flexural compression zone, allowing a reduction in the longitudinal torsion steel required in the compression zone.

R11.6.3.10 — As explained in R11.6.3.7, torsion causes an axial tension force. In a nonprestressed beam, this force is resisted by longitudinal reinforcement having an axial tensile capacity of $A_\ell f_{y\ell}$. This steel is in addition to the flexural reinforcement, and is distributed uniformly around the sides of the perimeter so that the resultant of $A_\ell f_{y\ell}$ acts along the axis of the member.

In a prestressed beam, the same technique (providing additional reinforcing bars with capacity $A_\ell f_{y\ell}$) can be followed, or the designer can use any overcapacity of the prestressing steel to resist some of the axial force $A_\ell f_{y\ell}$ as outlined in the next paragraph.

CODE

COMMENTARY

In a prestressed beam, the tendon stress at ultimate at the section of the maximum moment is f_{ps}. At other sections, the prestressing steel stress at ultimate will be between f_{se} and f_{ps}. A portion of the $A_\ell f_{y\ell}$ force acting on the sides of the perimeter where the prestressing steel is located can be resisted by a force $A_{ps}\Delta f_p$ in the prestressing steel where Δf_p is f_{ps} minus the prestressing steel stress due to flexure at the ultimate load at the section in question. This can be taken as M_u at the section, divided by ($\phi 0.9 d_p A_{ps}$), but Δf_p should not be more than 60 ksi. Longitudinal reinforcing bars will be required on the other sides of the member to provide the remainder of the $A_\ell f_{y\ell}$ force, or to satisfy the spacing requirements given in 11.6.6.2, or both.

11.6.3.11 — In prestressed beams, it shall be permitted to reduce the area of longitudinal torsional reinforcement on the side of the member in compression due to flexure below that required by 11.6.3.10 in accordance with 11.6.3.9.

11.6.4 — Details of torsional reinforcement

11.6.4.1 — Torsion reinforcement shall consist of longitudinal bars or tendons and one or more of the following:

(a) Closed stirrups or closed ties, perpendicular to the axis of the member; or

(b) A closed cage of welded wire fabric with transverse wires perpendicular to the axis of the member; or

(c) In nonprestressed beams, spiral reinforcement.

11.6.4.2 — Transverse torsional reinforcement shall be anchored by one of the following:

(a) A 135-deg standard hook around a longitudinal bar; or

(b) According to 12.13.2.1, 12.13.2.2, or 12.13.2.3 in regions where the concrete surrounding the anchorage is restrained against spalling by a flange or slab or similar member.

11.6.4.3 — Longitudinal torsion reinforcement shall be developed at both ends.

11.6.4.4 — For hollow sections in torsion, the distance from the centerline of the transverse torsional reinforcement to the inside face of the wall of the hollow section shall not be less than $0.5A_{oh}/p_h$.

R11.6.4 — Details of torsional reinforcement

R11.6.4.1 — Both longitudinal and closed transverse reinforcement are required to resist the diagonal tension stresses due to torsion. The stirrups must be closed, because inclined cracking due to torsion may occur on all faces of a member.

In the case of sections subjected primarily to torsion, the concrete side cover over the stirrups spalls off at high torques.[11.33] This renders lapped-spliced stirrups ineffective, leading to a premature torsional failure.[11.34] In such cases, closed stirrups should not be made up of pairs of U-stirrups lapping one another.

R11.6.4.2 — When a rectangular beam fails in torsion, the corners of the beam tend to spall off due to the inclined compressive stresses in the concrete diagonals of the space truss changing direction at the corner as shown in Fig. 11.6.4.2(a). In tests, closed stirrups anchored by 90-deg hooks failed when this occurred.[11.33] For this reason, 135-deg hooks are preferable for torsional stirrups in all cases. In regions where this spalling is prevented by an adjacent slab or flange, 11.6.4.2(b) relaxes this and allows 90-deg hooks.

R.11.6.4.3 — If high torsion acts near the end of a beam, the longitudinal torsion reinforcement must be adequately anchored. Sufficient development length must be provided outside the inner face of the support to develop the needed tension force in the bars or tendons. In the case of bars this may require hooks or horizontal U-shaped bars lapped with the longitudinal torsion reinforcement.

R11.6.4.4 — The closed stirrups provided for torsion in a hollow section should be located in the outer half of the wall thickness effective for torsion where the wall thickness can be taken as A_{oh}/p_h.

CODE

11.6.5 — Minimum torsion reinforcement

11.6.5.1 — A minimum area of torsion reinforcement shall be provided in all regions where the factored torsional moment T_u exceeds the values specified in 11.6.1.

11.6.5.2 — Where torsional reinforcement is required by 11.6.5.1, the minimum area of transverse closed stirrups shall be computed by:

$$(A_v + 2A_t) = -0.75\sqrt{f_c'}\frac{b_w s}{f_{yv}} \qquad \text{(11-23)}$$

but shall not be less than $(50 b_w s)/f_{yv}$.

11.6.5.3 — Where torsional reinforcement is required by 11.6.5.1, the minimum total area of longitudinal torsional reinforcement shall be computed by

$$A_{\ell,min} = \frac{5\sqrt{f_c'}A_{cp}}{f_{y\ell}} - \left(\frac{A_t}{s}\right)p_h\frac{f_{yv}}{f_{y\ell}} \qquad \text{(11-24)}$$

where A_t/s shall not be taken less than $25 b_w/f_{yv}$.

COMMENTARY

R11.6.5 — Minimum torsion reinforcement

R11.6.5.1 and R11.6.5.2 — If a member is subject to a factored torsional moment T_u greater than the values specified in 11.6.1, the minimum amount of transverse web reinforcement for combined shear and torsion is $50 b_w s/f_{yv}$. The differences in the definition of A_v and the symbol A_t should be noted; A_v is the area of two legs of a closed stirrup while A_t is the area of only one leg of a closed stirrup.

Tests[11.9] of high-strength reinforced concrete beams have indicated the need to increase the minimum area of shear reinforcement to prevent shear failures when inclined cracking occurs. Although there are a limited number of tests of high-strength concrete beams in torsion, the equation for the minimum area of transverse closed stirrups has been changed for consistency with calculations required for minimum shear reinforcement.

R11.6.5.3 — Reinforced concrete beam specimens with less than 1 percent torsional reinforcement by volume have failed in pure torsion at torsional cracking.[11.27] In the 1989 and prior 318 codes, a relationship was presented that required about 1 percent torsional reinforcement in beams loaded in pure torsion and less in beams with combined shear and torsion, as a function of the ratio of shear stresses due to torsion and shear. Equation (11-24) was simplified by assuming a single value of this reduction factor and results in a volumetric ratio of about 0.5 percent.

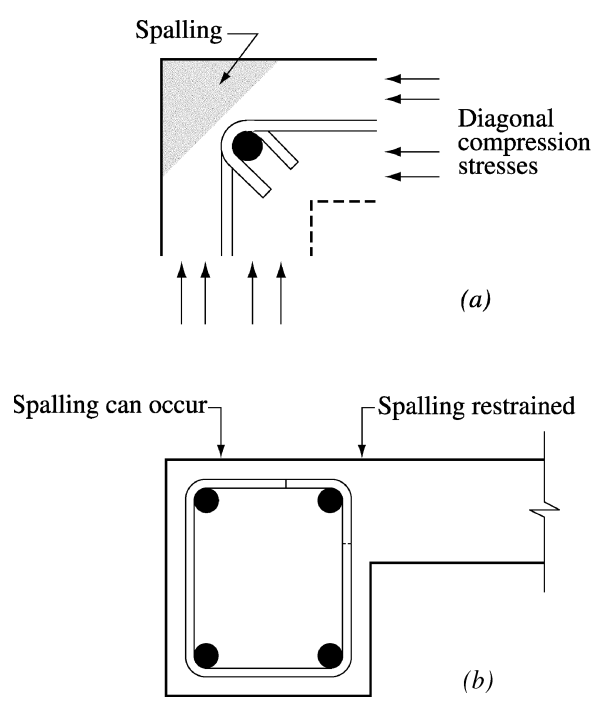

Fig. R11.6.4.2—Spalling of corners of beams loaded in torsion.

CODE

11.6.6 — Spacing of torsion reinforcement

11.6.6.1 — The spacing of transverse torsion reinforcement shall not exceed the smaller of $p_h/8$ or 12 in.

11.6.6.2 — The longitudinal reinforcement required for torsion shall be distributed around the perimeter of the closed stirrups with a maximum spacing of 12 in. The longitudinal bars or tendons shall be inside the stirrups. There shall be at least one longitudinal bar or tendon in each corner of the stirrups. Bars shall have a diameter at least 1/24 of the stirrup spacing, but not less than a No. 3 bar.

11.6.6.3 — Torsion reinforcement shall be provided for a distance of at least $(b_t + d)$ beyond the point theoretically required.

11.7 — Shear-friction

11.7.1 — Provisions of 11.7 are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

11.7.2 — Design of cross sections subject to shear transfer as described in 11.7.1 shall be based on Eq. (11-1), where V_n is calculated in accordance with provisions of 11.7.3 or 11.7.4.

11.7.3 — A crack shall be assumed to occur along the shear plane considered. The required area of shear-friction reinforcement A_{vf} across the shear plane shall be designed using either 11.7.4 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests.

11.7.3.1 — Provisions of 11.7.5 through 11.7.10 shall apply for all calculations of shear transfer strength.

COMMENTARY

R11.6.6 — Spacing of torsion reinforcement

R11.6.6.1 — The spacing of the stirrups is limited to ensure the development of the ultimate torsional strength of the beam, to prevent excessive loss of torsional stiffness after cracking, and to control crack widths. For a square cross section, the $p_h/8$ limitation requires stirrups at $d/2$, which corresponds to 11.5.4.1.

R11.6.6.2 — In R11.6.3.7, it was shown that longitudinal reinforcement is needed to resist the sum of the longitudinal tensile forces due to torsion in the walls of the thin-walled tube. Because the force acts along the centroidal axis of the section, the centroid of the additional longitudinal reinforcement for torsion should approximately coincide with the centroid of the section. The code accomplishes this by requiring the longitudinal torsional reinforcement to be distributed around the perimeter of the closed stirrups. Longitudinal bars or tendons are required in each corner of the stirrups to provide anchorage for the legs of the stirrups. Corner bars have also been found to be very effective in developing torsional strength and in controlling cracks.

R11.6.6.3 — The distance $(b_t + d)$ beyond the point theoretically required for torsional reinforcement is larger than that used for shear and flexural reinforcement because torsional diagonal tension cracks develop in a helical form.

R11.7 — Shear-friction

R11.7.1 — With the exception of 11.7, virtually all provisions regarding shear are intended to prevent diagonal tension failures rather than direct shear transfer failures. The purpose of 11.7 is to provide design methods for conditions where shear transfer should be considered: an interface between concretes cast at different times, an interface between concrete and steel, reinforcement details for precast concrete structures, and other situations where it is considered appropriate to investigate shear transfer across a plane in structural concrete. (See References 11.35 and 11.36).

R11.7.3 — Although uncracked concrete is relatively strong in direct shear, there is always the possibility that a crack will form in an unfavorable location. The shear-friction concept assumes that such a crack will form, and that reinforcement must be provided across the crack to resist relative displacement along it. When shear acts along a crack, one crack face slips relative to the other. If the crack faces are rough and irregular, this slip is accompanied by separation of the crack faces. At ultimate, the separation is sufficient to stress the reinforcement crossing the crack to its yield point. The reinforcement provides a clamping force

$A_{vf}f_y$ across the crack faces. The applied shear is then resisted by friction between the crack faces, by resistance to the shearing off of protrusions on the crack faces, and by dowel action of the reinforcement crossing the crack. Successful application of 11.7 depends on proper selection of the location of an assumed crack.[11.18,11.35]

The relationship between shear-transfer strength and the reinforcement crossing the shear plane can be expressed in various ways. Equation (11-25) and (11-26) of 11.7.4 are based on the shear-friction model. This gives a conservative prediction of shear-transfer strength. Other relationships that give a closer estimate of shear-transfer strength[11.18,11.37,11.38] can be used under the provisions of 11.7.3. For example, when the shear-friction reinforcement is perpendicular to the shear plane, the shear strength V_n is given by[11.37,11.38]

$$V_n = 0.8A_{vf}f_y + A_cK_1$$

where A_c is the area of concrete section resisting shear transfer (square inches) and $K_1 = 400$ **psi** for normalweight concrete, 200 psi for "all-lightweight" concrete, and 250 psi for "sand-lightweight" concrete. These values of K_1 apply to both monolithically cast concrete and to concrete cast against hardened concrete with a rough surface, as defined in 11.7.9.

In this equation, the first term represents the contribution of friction to shear-transfer resistance (0.8 representing the coefficient of friction). The second term represents the sum of: (1) the resistance to shearing of protrusions on the crack faces; and (2) the dowel action of the reinforcement.

When the shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in that reinforcement, the shear strength V_n is given by

$$V_n = A_{vf}f_y\,(0.8\sin\alpha_f + \cos\alpha_f) + A_cK_1\sin^2\alpha_f$$

where α_f is the angle between the shear-friction reinforcement and the shear plane (that is, $0 < \alpha_f < 90$ **deg**).

When using the modified shear-friction method, the terms $(A_{vf}f_y/A_c)$ or $(A_{vf}f_y\sin\alpha_f/A_c)$ must not be less than 200 psi for the design equations to be valid.

11.7.4 — Shear-friction design method

11.7.4.1 — When shear-friction reinforcement is perpendicular to shear plane, shear strength V_n shall be computed by

$$V_n = A_{vf}f_y\mu \qquad (11\text{-}25)$$

where μ is coefficient of friction in accordance with 11.7.4.3.

R11.7.4 — Shear-friction design method

R11.7.4.1 — The required area of shear-transfer reinforcement A_{vf} is computed using

$$A_{vf} = \frac{V_u}{\phi f_y\mu}$$

The specified upper limit on shear strength should also be observed.

CODE

11.7.4.2 — When shear-friction reinforcement is inclined to shear plane, such that the shear force produces tension in shear-friction reinforcement, shear strength V_n shall be computed by

$$V_n = A_{vf} f_y(\mu \sin \alpha_f + \cos \alpha_f) \qquad (11\text{-}26)$$

where α_f is angle between shear-friction reinforcement and shear plane.

11.7.4.3 — The coefficient of friction μ in Eq. (11-25) and Eq. (11-26) shall be

Concrete placed monolithically 1.4λ

Concrete placed against hardened concrete with surface intentionally roughened as specified in 11.7.9 .... 1.0λ

Concrete placed against hardened concrete not intentionally roughened 0.6λ

Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see 11.7.10) 0.7λ

where λ = 1.0 for normalweight concrete, 0.85 for "sand-lightweight" concrete and 0.75 for "all lightweight" concrete. Linear interpolation shall be permitted when partial sand replacement is used.

COMMENTARY

R11.7.4.2 — When the shear-friction reinforcement is inclined to the shear plane, such that the component of the shear force parallel to the reinforcement tends to produce tension in the reinforcement, as shown in Fig. R11.7.4, part of the shear is resisted by the component parallel to the shear plane of the tension force in the reinforcement.[11.38] Equation (11-26) must be used only when the shear force component parallel to the reinforcement produces tension in the reinforcement, as shown in Fig. R11.7.4. When α_f is greater than 90 deg, the relative movement of the surfaces tends to compress the bar, and Eq. (11-26) is not valid.

R11.7.4.3 — In the shear-friction method of calculation, it is assumed that all the shear resistance is due to the friction between the crack faces. It is, therefore, necessary to use artificially high values of the coefficient of friction in the shear-friction equations, so that the calculated shear strength will be in reasonable agreement with test results. For the case of concrete cast against hardened concrete not roughened in accordance with 11.7.9, shear resistance is primarily due to dowel action of the reinforcement and tests[11.39] indicate that reduced value of μ = **0.6**λ specified for this case is appropriate.

The value of μ specified for concrete placed against as-rolled structural steel relates to the design of connections between precast concrete members, or between structural steel members and structural concrete members. The shear-transfer reinforcement may be either reinforcing bars or headed stud shear connectors; also, field welding to steel plates after casting of concrete is common. The design of shear connectors for composite action of concrete slabs and steel beams is not covered by these provisions, but should be in accordance with Reference 11.40.

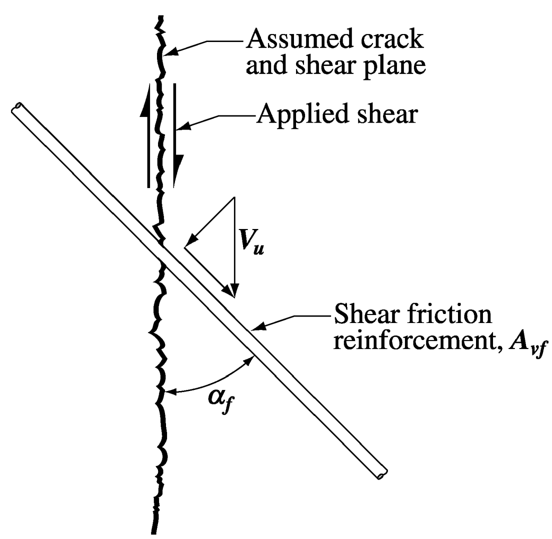

Fig. R11.7.4—Shear-friction reinforcement at angle to assumed crack.

CODE

11.7.5 — Shear strength V_n shall not be taken greater than $0.2 f_c' A_c$ nor 800 A_c in lb, where A_c is area of concrete section resisting shear transfer.

11.7.6 — Design yield strength of shear-friction reinforcement shall not exceed 60,000 psi.

11.7.7 — Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane shall be permitted to be taken as additive to the force in the shear-friction reinforcement $A_{vf} f_y$ when calculating required A_{vf}.

11.7.8 — Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop the specified yield strength on both sides by embedment, hooks, or welding to special devices.

11.7.9 — For the purpose of 11.7, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, interface shall be roughened to a full amplitude of approximately 1/4 in.

COMMENTARY

R11.7.5 — This upper limit on shear strength is specified because Eq. (11-25) and (11-26) become unconservative if V_n has a greater value.

R11.7.7 — If a resultant tensile force acts across a shear plane, reinforcement to carry that tension must be provided in addition to that provided for shear transfer. Tension may be caused by restraint of deformations due to temperature change, creep, and shrinkage. Such tensile forces have caused failures, particularly in beam bearings.

When moment acts on a shear plane, the flexural tension stresses and flexural compression stresses are in equilibrium. There is no change in the resultant compression $A_{vf} f_y$ acting across the shear plane, and the shear-transfer strength is not changed. It is therefore not necessary to provide additional reinforcement to resist the flexural tension stresses, unless the required flexural tension reinforcement exceeds the amount of shear-transfer reinforcement provided in the flexural tension zone. This has been demonstrated experimentally.[11.41]

It has also been demonstrated experimentally[11.36] that if a resultant compressive force acts across a shear plane, the shear-transfer strength is a function of the sum of the resultant compressive force and the force $A_{vf} f_y$ in the shear-friction reinforcement. In design, advantage should be taken of the existence of a compressive force across the shear plane to reduce the amount of shear-friction reinforcement required, only if it is absolutely certain that the compressive force is permanent.

R11.7.8 — If no moment acts across the shear plane, reinforcement should be uniformly distributed along the shear plane to minimize crack widths. If a moment acts across the shear plane, it is desirable to distribute the shear-transfer reinforcement primarily in the flexural tension zone.

Because the shear-friction reinforcement acts in tension, it must have full tensile anchorage on both sides of the shear plane. Further, the shear-friction reinforcement anchorage must engage the primary reinforcement, otherwise a potential crack may pass between the shear-friction reinforcement and the body of the concrete. This requirement applies particularly to welded headed studs used with steel inserts for connections in precast and cast-in-place concrete. Anchorage may be developed by bond, by a welded mechanical anchorage, or by threaded dowels and screw inserts. Space limitations often require a welded mechanical anchorage. For anchorage of headed studs in concrete, see Reference 11.18.

CODE

11.7.10 — When shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint.

11.8 — Deep beams

11.8.1 — The provisions of 11.8 shall apply to members with ℓ_n/d less than 5 that are loaded on one face and supported on the opposite face so that compression struts can develop between the loads and the supports. See also 12.10.6.

11.8.2 — The design of simply supported deep beams for shear shall be based on Eq. (11-1) and (11-2), where the shear strength V_c shall be in accordance with 11.8.6 or 11.8.7 and the shear strength V_s shall be in accordance with 11.8.8.

11.8.3 — The design of continuous deep beams for shear shall be based on 11.1 through 11.5 with 11.8.5 substituted for 11.1.3, or on methods satisfying equilibrium and strength requirements. In either case the design shall also satisfy 11.8.4, 11.8.9, and 11.8.10.

11.8.4 — Shear strength V_n for deep beams shall not be taken greater than $8\sqrt{f_c'}\,b_w d$ when ℓ_n/d is less than 2. When ℓ_n/d is between 2 and 5

$$V_n = \frac{2}{3}\left(10 + \frac{\ell_n}{d}\right)\sqrt{f_c'}\,b_w d \qquad (11\text{-}27)$$

11.8.5 — Critical section for shear measured from face of support shall be taken at a distance $0.15\ell_n$ for uniformly loaded beams and $0.50a$ for beams with concentrated loads, but not greater than d.

11.8.6 — Unless a more detailed calculation is made in accordance with 11.8.7

$$V_c = 2\sqrt{f_c'}\,b_w d \qquad (11\text{-}28)$$

COMMENTARY

R11.8 — Deep beams

R11.8.1 — The behavior of a deep beam is discussed in References 11.5 and 11.42. For a deep beam supporting gravity loads, this section applies if the loads are applied on the top of the beam and the beam is supported on its bottom face. If the loads are applied through the sides or bottom of such a member, the design for shear should be the same as for ordinary beams.

The longitudinal tension reinforcement in deep beams should be extended to the supports and adequately anchored by embedment, hooks, or welding to special devices. Bent-up bars are not recommended.

R11.8.3 — In a continuous beam, the critical section for shear defined in 11.8.5 occurs at a point where M_u approaches zero. As a result, the second term in Eq. (11-29) becomes large. For this reason, 11.8.3 requires continuous deep beams to be designed for shear according to the regular beam design procedures except that 11.8.5 is used to define the critical section for shear rather than 11.1.3. For a uniformly loaded beam, 11.1.3 allows one to design for the shear at d away from the support. This will frequently approach zero in a deep beam.

As an alternative to the regular beam design procedures, design methods satisfying equilibrium and strength requirements are allowed. Such methods are presented in References 11.42 and 11.43.

CODE

11.8.7 — Shear strength V_c shall be permitted to be computed by

$$V_c = \left(3.5 - 2.5\frac{M_u}{V_u d}\right)\left(1.9\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u}\right)b_w d \quad (11\text{-}29)$$

except that the term

$$\left(3.5 - 2.5\frac{M_u}{V_u d}\right)$$

shall not exceed 2.5, and V_c shall not be taken greater than $6\sqrt{f_c'}\, b_w d$. M_u is factored moment occurring simultaneously with V_u at the critical section defined in 11.8.5.

11.8.8 — Where factored shear force V_u exceeds shear strength ϕV_c, shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where shear strength V_s shall be computed by

$$V_s = \left[\frac{A_v}{s}\left(\frac{1 + \frac{\ell_n}{d}}{12}\right) + \frac{A_{vh}}{s_2}\left(\frac{11 - \frac{\ell_n}{d}}{12}\right)\right]f_y d \quad (11\text{-}30)$$

where A_v is area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s, and A_{vh} is area of shear reinforcement parallel to flexural reinforcement within a distance s_2.

11.8.9 — The area of shear reinforcement perpendicular to the span A_v shall not be less than $0.0025 b_w s$, and s shall not exceed $d/5$, nor 12 in.

11.8.10 — The area of shear reinforcement parallel to the span A_{vh} shall not be less than $0.0015 b_w s_2$, and s_2 shall not exceed $d/5$, nor 12 in.

11.8.11 — Shear reinforcement required at the critical section defined in 11.8.5 shall be used throughout the span.

11.9 — Special provisions for brackets and corbels

COMMENTARY

R11.8.7 — As the span-depth ratio of a member without web reinforcement decreases, its shear strength increases above the shear causing diagonal tension cracking. In Eq. (11-29), it is assumed that diagonal cracking occurs at the same shear strength as for ordinary beams, but that the shear strength carried by the concrete will be greater than the shear strength causing diagonal cracking.

Designers should note that shear in excess of the shear causing diagonal cracking may result in unsightly cracking unless shear reinforcement is provided.

R11.8.8 — The inclination of diagonal cracking may be greater than 45 deg; therefore, both horizontal and vertical shear reinforcement is required in deep flexural members.[11.44] The relative amounts of horizontal and vertical shear reinforcement that are selected from Eq. (11-30) may vary, as long as limits on the minimum amount and spacing are observed.

Special attention is directed to the importance of adequate anchorage for the shear reinforcement. Horizontal web reinforcement should be extended to the supports and anchored in the same manner as the tension reinforcement.

R11.8.9 and R11.8.10 — The relative amounts of horizontal and vertical shear reinforcement have been interchanged from those required in the 2001 code because tests[11.42,11.43,11.44] have shown that vertical shear reinforcement is more effective than horizontal shear reinforcement. The maximum spacing of bars is 12 in. because this steel is provided to restrain the width of the cracks.

R11.8.11 — Based on the analysis carried out at the critical sections specified in 11.8.5, it may be determined that the member either does not need shear reinforcement, or that shear reinforcement is required, in which case it must be used throughout the span.

R11.9 — Special provisions for brackets and corbels

Brackets and corbels are cantilevers having shear span-to-depth ratios not greater than unity, which tend to act as simple trusses or deep beams rather than flexural members designed for shear according to 11.3.

The corbel shown in Fig. R11.9.1 may fail by shearing along the interface between the column and the corbel, by yielding of the tension tie, by crushing or splitting of the compression strut, or by localized bearing or shearing failure under the loading plate. These failure modes are illustrated and are discussed more fully in Reference 11.1. The notation used in 11.9 is illustrated in Fig. R11.9.2.

CODE

11.9.1 — Provisions of 11.9 shall apply to brackets and corbels with a shear span-to-depth ratio ***a*/*d*** not greater than unity, and subject to a horizontal tensile force N_{uc} not larger than V_u. Distance ***d*** shall be measured at face of support.

11.9.2 — Depth at outside edge of bearing area shall not be less than **0.5*d***.

COMMENTARY

R11.9.1 — An upper limit of unity for ***a*/*d*** is specified for two reasons. First, for shear span-to-depth ratios exceeding unity, the diagonal tension cracks are less steeply inclined, and the use of horizontal stirrups alone as specified in 11.9.4 is not appropriate. Second, the specified method of design has only been validated experimentally for ***a*/*d*** of unity or less. An upper limit is specified for N_{uc} because this method of design has only been validated experimentally for N_{uc} less than or equal to V_u, including N_{uc} equal to zero.

R11.9.2 — A minimum depth is specified at the outside edge of the bearing area so that a premature failure will not occur due to a major diagonal tension crack propagating from below the bearing area to the outer sloping face of the corbel or bracket. Failures of this type have been observed[11.45] in corbels having depths at the outside edge of the bearing area less than specified in this section of the code.

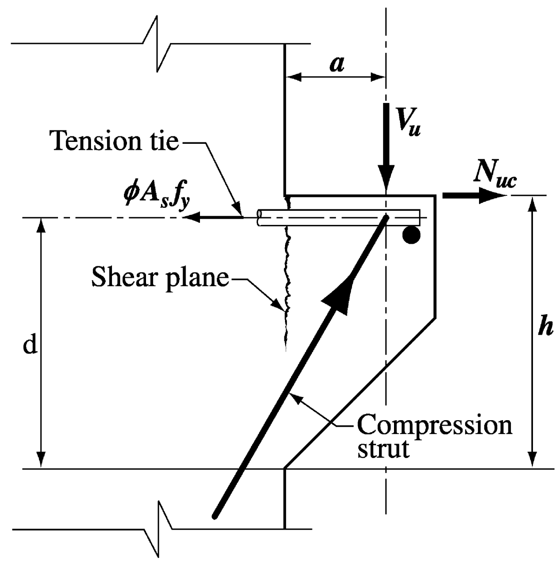

Fig. R11.9.1—Structural action of corbel.

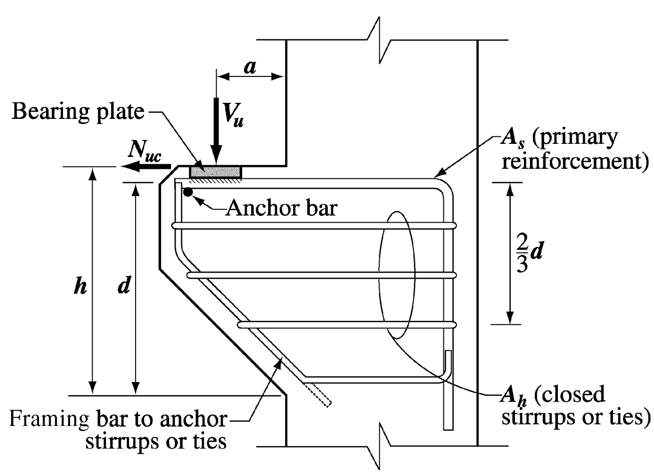

Fig. R11.9.2—Notation used in Section 11.9.

CODE

11.9.3 — Section at face of support shall be designed to resist simultaneously a shear V_u, a moment $[V_u a + N_{uc}(h - d)]$, and a horizontal tensile force N_{uc}.

11.9.3.1 — In all design calculations in accordance with 11.9, strength reduction factor ϕ shall be taken equal to 0.75.

11.9.3.2 — Design of shear-friction reinforcement A_{vf} to resist shear V_u shall be in accordance with 11.7.

11.9.3.2.1 — For normalweight concrete, shear strength V_n shall not be taken greater than $0.2f_c'b_wd$ nor $800b_wd$ in lb.

11.9.3.2.2 — For "all-lightweight" or "sand-lightweight" concrete, shear strength V_n shall not be taken greater than $(0.2 - 0.07a/d)f_c'b_wd$ nor $(800 - 280a/d)b_wd$ in lb.

11.9.3.3 — Reinforcement A_f to resist moment $[V_u a + N_{uc}(h - d)]$ shall be computed in accordance with 10.2 and 10.3.

11.9.3.4 — Reinforcement A_n to resist tensile force N_{uc} shall be determined from $N_{uc} \le \phi A_n f_y$. Tensile force N_{uc} shall not be taken less than $0.2V_u$ unless special provisions are made to avoid tensile forces. Tensile force N_{uc} shall be regarded as a live load even when tension results from creep, shrinkage, or temperature change.

11.9.3.5 — Area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$ or $(2A_{vf}/3 + A_n)$.

11.9.4 — Closed stirrups or ties parallel to A_s, with a total area A_h not less than $0.5(A_s - A_n)$, shall be uniformly distributed within two-thirds of the effective depth adjacent to A_s.

COMMENTARY

R11.9.3.1 — Corbel and bracket behavior is predominantly controlled by shear; therefore, a single value of $\phi = 0.75$ is specified for all design conditions.

R11.9.3.2.2 — Tests[11.46] have shown that the maximum shear strength of lightweight concrete corbels or brackets is a function of both f_c' and a/d. No data are available for corbels or brackets made of sand-lightweight concrete. As a result, the same limitations have been placed on both all-lightweight and sand-lightweight brackets and corbels.

R11.9.3.3 — Reinforcement required to resist moment can be calculated using ordinary flexural theory. The factored moment is calculated by summing moments about the flexural reinforcement at the face of support.

R11.9.3.4 — Because the magnitude of horizontal forces acting on corbels or brackets cannot usually be determined with great accuracy, it is specified that N_{uc} be regarded as a live load.

R11.9.3.5 — Tests[11.46] suggest that the total amount of reinforcement $(A_s + A_h)$ required to cross the face of support should be the greater of:

(a) The sum of A_{vf} calculated according to 11.9.3.2 and A_n calculated according to 11.9.3.4; or

(b) The sum of **3/2** times A_f calculated according to 11.9.3.3 and A_n calculated according to 11.9.3.4.

If (a) controls, $A_s = (2A_{vf}/3 + A_n)$ is required as primary tensile reinforcement, and the remaining $A_{vf}/3$ should be provided as closed stirrups parallel to A_s and distributed within $(2/3)d$, adjacent to A_s. Section 11.9.4 satisfies this by requiring $A_h = 0.5(2A_{vf}/3)$.

If (b) controls, $A_s = (A_f + A_n)$ is required as primary tension reinforcement, and the remaining $A_f/2$ should be provided as closed stirrups parallel to A_s and distributed within $(2/3)d$, adjacent to A_s. Again 11.9.4 satisfies this requirement.

R11.9.4 — Closed stirrups parallel to the primary tension reinforcement are necessary to prevent a premature diagonal tension failure of the corbel or bracket. The required area of closed stirrups $A_h = 0.5(A_s - A_n)$ automatically yields the appropriate amounts, as discussed in R11.9.3.5 above.

CODE

11.9.5 — Ratio $\rho = A_s/bd$ shall not be less than $0.04(f_c'/f_y)$.

11.9.6 — At front face of bracket or corbel, primary tension reinforcement A_s shall be anchored by one of the following:

(a) By a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars;

(b) By bending primary tension bars A_s back to form a horizontal loop; or

(c) By some other means of positive anchorage.

11.9.7 — Bearing area of load on bracket or corbel shall not project beyond straight portion of primary tension bars A_s, nor project beyond interior face of transverse anchor bar (if one is provided).

11.10 — Special provisions for walls

11.10.1 — Design for shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in 11.12. Design for horizontal in-plane shear forces in a wall shall be in accordance with 11.10.2 through 11.10.9.

11.10.2 — Design of horizontal section for shear in plane of wall shall be based on Eq. (11-1) and (11-2), where shear strength V_c shall be in accordance with 11.10.5 or 11.10.6 and shear strength V_s shall be in accordance with 11.10.9.

COMMENTARY

R11.9.5 — A minimum amount of reinforcement is specified to prevent the possibility of sudden failure should the bracket or corbel concrete crack under the action of flexural moment and outward tensile force N_{uc}.

R11.9.6 — Because the horizontal component of the inclined concrete compression strut (see Fig. R11.9.1) is transferred to the primary tension reinforcement at the location of the vertical load, the reinforcement A_s is essentially uniformly stressed from the face of the support to the point where the vertical load is applied. It should, therefore, be anchored at its outer end and in the supporting column, so as to be able to develop its yield strength from the face of support to the vertical load. Satisfactory anchorage at the outer end can be obtained by bending the A_s bars in a horizontal loop as specified in (b), or by welding a bar of equal diameter or a suitably sized angle across the ends of the A_s bars. The welds should be designed to develop the yield strength of the reinforcement A_s. The weld detail used successfully in the corbel tests reported in Reference 11.45 is shown in Fig. R11.9.6. The reinforcement A_s should be anchored within the supporting column in accordance with the requirements of Chapter 12. See additional discussion on end anchorage in R12.10.6.

R11.9.7 — The restriction on the location of the bearing area is necessary to ensure development of the yield strength of the reinforcement A_s near the load. When corbels are designed to resist horizontal forces, the bearing plate should be welded to the tension reinforcement A_s.

R11.10 — Special provisions for walls

R11.10.1 — Shear in the plane of the wall is primarily of importance for shearwalls with a small height-to-length ratio. The design of higher walls, particularly walls with uniformly distributed reinforcement, will probably be controlled by flexural considerations.

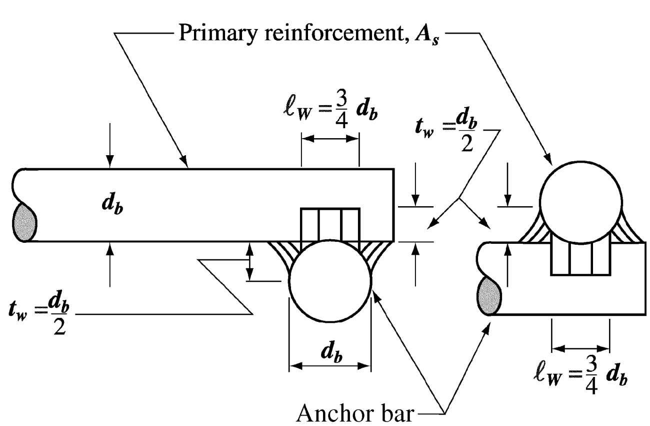

Fig. R11.9.6—Weld details used in tests of Reference 11.45.

CODE

11.10.3 — Shear strength V_n at any horizontal section for shear in plane of wall shall not be taken greater than $10\sqrt{f_c'}\,hd$.

11.10.4 — For design for horizontal shear forces in plane of wall, d shall be taken equal to $0.8\ell_w$. A larger value of d, equal to the distance from extreme compression fiber to center of force of all reinforcement in tension, shall be permitted to be used when determined by a strain compatibility analysis.

11.10.5 — Unless a more detailed calculation is made in accordance with 11.10.6, shear strength V_c shall not be taken greater than $2\sqrt{f_c'}\,hd$ for walls subject to N_u in compression, or V_c shall not be taken greater than the value given in 11.3.2.3 for walls subject to N_u in tension.

11.10.6 — Shear strength V_c shall be permitted to be computed by Eq. (11-31) and (11-32), where V_c shall be the lesser of Eq. (11-31) or (11-32)

$$V_c = 3.3\sqrt{f_c'}\,hd + \frac{N_u d}{4\ell_w} \tag{11-31}$$

or

$$V_c = \left[0.6\sqrt{f_c'} + \frac{\ell_w\left(1.25\sqrt{f_c'} + 0.2\dfrac{N_u}{\ell_w h}\right)}{\dfrac{M_u}{V_u} - \dfrac{\ell_w}{2}}\right] hd \tag{11-32}$$

where N_u is negative for tension. When $(M_u/V_u - \ell_w/2)$ is negative, Eq. (11-32) shall not apply.

11.10.7 — Sections located closer to wall base than a distance $\ell_w/2$ or one-half the wall height, whichever is less, shall be permitted to be designed for the same V_c as that computed at a distance $\ell_w/2$ or one-half the height.

11.10.8 — When factored shear force V_u is less than $\phi V_c/2$, reinforcement shall be provided in accordance with 11.10.9 or in accordance with Chapter 14. When V_u exceeds $\phi V_c/2$, wall reinforcement for resisting shear shall be provided in accordance with 11.10.9.

11.10.9 — Design of shear reinforcement for walls

COMMENTARY

R11.10.3 — Although the width-to-depth ratio of shearwalls is less than that for ordinary beams, tests[11.47] on shearwalls with a thickness equal to $\ell_w/25$ have indicated that ultimate shear stresses in excess of $10\sqrt{f_c'}$ can be obtained.

R11.10.5 and R11.10.6 — Equation (11-31) and (11-32) may be used to determine the inclined cracking strength at any section through a shearwall. Equation (11-31) corresponds to the occurrence of a principal tensile stress of approximately $4\sqrt{f_c'}$ at the centroid of the shearwall cross section. Equation (11-32) corresponds approximately to the occurrence of a flexural tensile stress of $6\sqrt{f_c'}$ at a section $\ell_w/2$ above the section being investigated. As the term

$$\left(\frac{M_u}{V_u} - \frac{\ell_w}{2}\right)$$

decreases, Eq. (11-31) will control before this term becomes negative. When this term becomes negative, Eq. (11-31) should be used.

R11.10.7 — The values of V_c computed from Eq. (11-31) and (11-32) at a section located a distance $\ell_w/2$ or $h_w/2$ (whichever is less) above the base apply to that and all sections between this section and the base. However, the maximum factored shear force V_u at any section, including the base of the wall, is limited to ϕV_n in accordance with 11.10.3.

R11.10.9 — Design of shear reinforcement for walls

Both horizontal and vertical shear reinforcement are required for all walls. For low walls, test data[11.48] indicate that horizontal shear reinforcement becomes less effective with vertical reinforcement becoming more effective. This change in effectiveness of the horizontal versus vertical reinforcement is recognized in Eq. (11-34); when h_w/ℓ_w is less than 0.5, the amount of vertical reinforcement is equal to the amount of horizontal reinforcement. When h_w/ℓ_w is greater than 2.5, only a minimum amount of vertical reinforcement is required ($\mathbf{0.0025}s_1h$).

Equation (11-33) is presented in terms of shear strength V_s provided by the horizontal shear reinforcement for direct application in Eq. (11-1) and (11-2).

Vertical shear reinforcement also must be provided in accordance with 11.10.9.4 within the spacing limitation of 11.10.9.5.

11.10.9.1 — Where factored shear force V_u exceeds shear strength ϕV_c, horizontal shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where shear strength V_s shall be computed by

$$V_s = \frac{A_v f_y d}{s_2} \tag{11-33}$$

where A_v is area of horizontal shear reinforcement within a distance s_2 and distance d is in accordance with 11.10.4. Vertical shear reinforcement shall be provided in accordance with 11.10.9.4.

11.10.9.2 — Ratio ρ_h of horizontal shear reinforcement area to gross concrete area of vertical section shall not be less than 0.0025.

11.10.9.3 — Spacing of horizontal shear reinforcement s_2 shall not exceed $\ell_w/5$, nor 12 in.

R11.10.9.3 — Maximum spacing of 12 in. for reinforcement is for crack control.

11.10.9.4 — Ratio ρ_n of vertical shear reinforcement area to gross concrete area of horizontal section shall not be less than

$$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_h - 0.0025) \tag{11-34}$$

nor 0.0025, but need not be greater than the required horizontal shear reinforcement.

11.10.9.5 — Spacing of vertical shear reinforcement s_1 shall not exceed $\ell_w/3$, nor 12 in.

R11.10.9.5 — Maximum spacing of 12 in. for reinforcement is for crack control.

11.11 — Transfer of moments to columns

11.11.1 — When gravity load, wind, earthquake, or other lateral forces cause transfer of moment at connections of framing elements to columns, the shear resulting from moment transfer shall be considered in the design of lateral reinforcement in the columns.

R11.11 — Transfer of moments to columns

R11.11.1 — Tests[11.49] have shown that the joint region of a beam-to-column connection in the interior of a building does not require shear reinforcement if the joint is confined on four sides by beams of approximately equal depth. Joints without lateral confinement, however, such as at the exterior of a building, need shear reinforcement to prevent deterioration due to shear cracking.[11.50]

For regions where strong earthquakes may occur, joints may be required to withstand several reversals of loading that develop the flexural capacity of the adjoining beams. See Chapter 21 for special provisions for seismic design.

CODE

11.11.2 — Except for connections not part of a primary seismic load-resisting system that are restrained on four sides by beams or slabs of approximately equal depth, connections shall have lateral reinforcement not less than that required by Eq. (11-13) within the column for a depth not less than that of the deepest connection of framing elements to the columns. See also 7.9.

11.12 — Special provisions for slabs and footings

11.12.1 — The shear strength of slabs and footings in the vicinity of columns, concentrated loads, or reactions is governed by the more severe of two conditions:

11.12.1.1 — Beam action where each critical section to be investigated extends in a plane across the entire width. For beam action, the slab or footing shall be designed in accordance with 11.1 through 11.5.

11.12.1.2 — Two-way action where each of the critical sections to be investigated shall be located so that its perimeter b_o is a minimum but need not approach closer than $d/2$ to

(a) Edges or corners of columns, concentrated loads, or reaction areas; or

(b) Changes in slab thickness such as edges of capitals or drop panels.

For two-way action, the slab or footing shall be designed in accordance with 11.12.2 through 11.12.6.

11.12.1.3 — For square or rectangular columns, concentrated loads, or reaction areas, the critical sections with four straight sides shall be permitted.

11.12.2 — The design of a slab or footing for two-way action is based on Eq. (11-1) and (11-2). V_c shall be computed in accordance with 11.12.2.1, 11.12.2.2, or 11.12.3.1. V_s shall be computed in accordance with 11.12.3. For slabs with shearheads, V_n shall be in accordance with 11.12.4. When moment is transferred between a slab and a column, 11.12.6 shall apply.

COMMENTARY

R11.12 — Special provisions for slabs and footings

R11.12.1 — Differentiation must be made between a long and narrow slab or footing acting as a beam, and a slab or footing subject to two-way action where failure may occur by "punching" along a truncated cone or pyramid around a concentrated load or reaction area.

R11.12.1.2 — The critical section for shear in slabs subjected to bending in two directions follows the perimeter at the edge of the loaded area.[11.3] The shear stress acting on this section at factored loads is a function of $\sqrt{f_c'}$ and the ratio of the side dimension of the column to the effective slab depth. A much simpler design equation results by assuming a pseudocritical section located at a distance $d/2$ from the periphery of the concentrated load. When this is done, the shear strength is almost independent of the ratio of column size to slab depth. For rectangular columns, this critical section was originally defined by straight lines drawn parallel to and at a distance $d/2$ from the edges of the loaded area. Section 11.12.1.3 allows the use of a rectangular critical section.

For slabs of uniform thickness, it is sufficient to check shear on one section. For slabs with changes in thickness as happens, for example at the edge of drop panels, it is necessary to check shear at several sections.

For edge columns at points where the slab cantilevers beyond the column, the critical perimeter will either be three-sided or four-sided.

CODE

11.12.2.1 — For nonprestressed slabs and footings, V_c shall be the smallest of (a), (b), and (c):

(a) $$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f_c'}\,b_o d \quad (11\text{-}35)$$

where β_c is the ratio of long side to short side of the column, concentrated load or reaction area

(b) $$V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f_c'}\,b_o d \quad (11\text{-}36)$$

where α_s is 40 for interior columns, 30 for edge columns, 20 for corner columns, and

(c) $$V_c = 4\sqrt{f_c'}\,b_o d \quad (11\text{-}37)$$

11.12.2.2 — At columns of two-way prestressed slabs and footings that meet the requirements of 18.9.3

$$V_c = (\beta_p\sqrt{f_c'} + 0.3f_{pc})b_o d + V_p \quad (11\text{-}38)$$

where β_p is the smaller of 3.5 or $(\alpha_s d/b_o + 1.5)$, α_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns, b_o is perimeter of critical section defined in 11.12.1.2, f_{pc} is the average value of f_{pc} for the two directions, and V_p is the vertical component of all effective prestress forces crossing the critical section. V_c shall be permitted to be computed by Eq. (11-38) if the following are satisfied; otherwise, 11.12.2.1 shall apply:

(a) no portion of the column cross section shall be closer to a discontinuous edge than 4 times the slab thickness; and

(b) f_c' in Eq. (11-38) shall not be taken greater than 5000 psi; and

(c) f_{pc} in each direction shall not be less than 125 psi, nor be taken greater than 500 psi.

11.12.3 — Shear reinforcement consisting of bars or wires and single- or multiple-leg stirrups shall be permitted in slabs and footings with an effective depth d greater than or equal to 6 in., but not less than 16 times

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R11.12.2.1 — For square columns, the shear stress due to ultimate loads in slabs subjected to bending in two directions is limited to $4\sqrt{f_c'}$. Tests,[11.51] however, have indicated that the value of $4\sqrt{f_c'}$ is unconservative when the ratio β_c of the lengths of the long and short sides of a rectangular column or loaded area is larger than 2.0. In such cases, the actual shear stress on the critical section at punching shear failure varies from a maximum of about $4\sqrt{f_c'}$ around the corners of the column or loaded area, down to $2\sqrt{f_c'}$ or less along the long sides between the two end sections. Other tests[11.52] indicate that v_c decreases as the ratio b_o/d increases. Equation (11-35) and (11-36) were developed to account for these two effects. The words "interior, edge, and corner columns" in 11.12.2.1(b) refer to critical sections with four, three, or two sides, respectively.

For shapes other than rectangular, β_c is taken to be the ratio of the longest overall dimension of the effective loaded area to the largest overall dimension of the effective loaded area measured perpendicular thereto, as illustrated for an L-shaped reaction area in Fig. R11.12.2. The effective loaded area is that area totally enclosing the actual loaded area, for which the perimeter is a minimum.

R11.12.2.2 — For prestressed slabs and footings, a modified form of Eq. (11-35) and (11-36) is specified for two-way action shear strength. Research[11.53,11.54] indicates that the shear strength of two-way prestressed slabs around interior columns is conservatively predicted by Eq. (11-38). V_c from Eq. (11-36) corresponds to a diagonal tension failure of the concrete initiating at the critical section defined in 11.12.1.2. The mode of failure differs from a punching shear failure of the concrete compression zone around the perimeter of the loaded area predicted by Eq. (11-35). Consequently, the term β_c does not enter into Eq. (11-38). Design values for f_c' and f_{pc} are restricted due to limited test data available for higher values. When computing f_{pc}, loss of prestress due to restraint of the slab by shearwalls and other structural elements must be taken into account.

In a prestressed slab with distributed tendons, the V_p term in Eq. (11-38) contributes only a small amount to the shear strength; therefore, it may be conservatively taken as zero. If V_p is to be included, the tendon profile assumed in the calculations must be specified.

For an exterior column support where the distance from the outside of the column to the edge of the slab is less than four times the slab thickness, the prestress is not fully effective around the total perimeter b_o of the critical section. Shear strength in this case is therefore conservatively taken the same as for a nonprestressed slab.

R11.12.3 — Research[11.55-11.59] has shown that shear reinforcement consisting of properly anchored bars or wires and single- or multiple-leg stirrups, or closed stirrups, can increase the punching shear resistance of slabs. The spacing

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the shear reinforcement diameter. Shear reinforcement shall be in accordance with 11.12.3.1 through 11.12.3.4.

11.12.3.1 — V_n shall be computed by Eq. (11-2), where V_c shall not be taken greater than $2\sqrt{f_c'}\,b_o d$, and the strength of shear reinforcement V_s shall be calculated in accordance with 11.5. The area of shear reinforcement A_v used in Eq. (11-15) is the cross-sectional area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the column section.

11.12.3.2 — V_n shall not be taken greater than $6\sqrt{f_c'}\,b_o d$.

11.12.3.3 — The distance between the column face and the first line of stirrup legs that surround the column shall not exceed $d/2$ nor 12 in. The spacing between adjacent stirrup legs in the first line of shear reinforcement shall not exceed $2d$ nor 12 in. measured in a direction parallel to the column face. The spacing between successive lines of shear reinforcement that surround the column shall not exceed $d/2$ nor 12 in. measured in a direction perpendicular to the column face.

11.12.3.4 — Slab shear reinforcement shall satisfy the anchorage requirements of 12.13 and shall engage the longitudinal flexural reinforcement in the direction being considered.

11.12.4 — Shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) shall be permitted in slabs. The provisions of 11.12.4.1 through 11.12.4.9 shall apply where shear due to gravity load is transferred at interior column supports. Where moment is transferred to columns, 11.12.6.3 shall apply.

11.12.4.1 — Each shearhead shall consist of steel shapes fabricated by welding with a full penetration weld into identical arms at right angles. Shearhead arms shall not be interrupted within the column section.

11.12.4.2 — A shearhead shall not be deeper than 70 times the web thickness of the steel shape.

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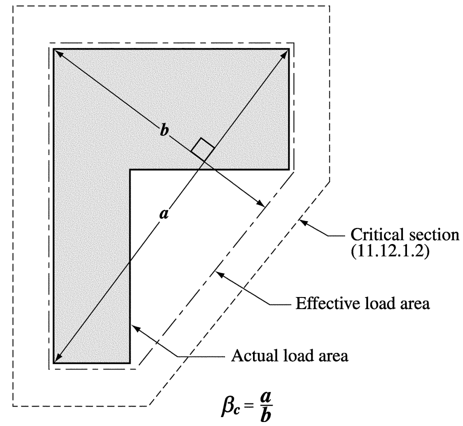

Fig. R11.12.2—Value of β_c for nonrectangular loaded area.

limits given in 11.12.3.3 correspond to slab shear reinforcement details that have been shown to be effective. Sections 12.13.2 and 12.13.3 give anchorage requirements for stirrup-type shear reinforcement that should also be applied for bars or wires used as slab shear reinforcement. It is essential that this shear reinforcement engage longitudinal reinforcement at both the top and bottom of the slab, as shown for topical details in Fig. R11.12.3(a) to (c). Anchorage of shear reinforcement according to the requirements of 12.13 is difficult in slabs thinner than 10 in. Shear reinforcement consisting of vertical bars mechanically anchored at each end by a plate or head capable of developing the yield strength of the bars has been used successfully.[11.59]

In a slab-column connection for which the moment transfer is negligible, the shear reinforcement should be symmetrical about the centroid of the critical section (Fig. R11.12.3(d)). Spacing limits defined in 11.12.3.3 are also shown in Fig. R11.12.3(d) and (e). At edge columns or for interior connections where moment transfer is significant, closed stirrups are recommended in a pattern as symmetrical as possible. Although the average shear stresses on faces *AD* and *BC* of the exterior column in Fig. R11.12.3(e) are lower than on face *AB*, the closed stirrups extending from faces *AD* and *BC* provide some torsional capacity along the edge of the slab.

R11.12.4 — Based on reported test data,[11.60] design procedures are presented for shearhead reinforcement consisting of structural steel shapes. For a column connection transferring moment, the design of shearheads is given in 11.12.6.3.

Three basic criteria must be considered in the design of shearhead reinforcement for connections transferring shear due to gravity load. First, a minimum flexural strength must be provided to assure that the required shear strength of the slab is reached before the flexural strength of the shearhead is exceeded. Second, the shear stress in the slab at the end of the shearhead reinforcement must be limited. Third, after these two requirements are satisfied, the designer can reduce the negative slab reinforcement in proportion to the moment contribution of the shearhead at the design section.

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11.12.4.3 — The ends of each shearhead arm shall be permitted to be cut at angles not less than 30 deg with the horizontal, provided the plastic moment strength of the remaining tapered section is adequate to resist the shear force attributed to that arm of the shearhead.

11.12.4.4 — All compression flanges of steel shapes shall be located within **0.3*d*** of compression surface of slab.

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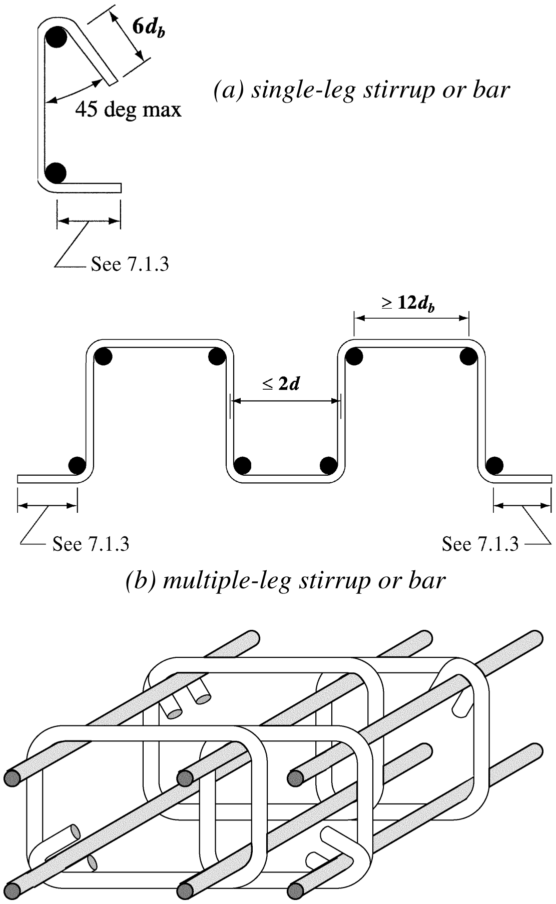

Fig. R11.12.3(a)-(c)—Single- or multiple-leg stirrup-type slab shear reinforcement.

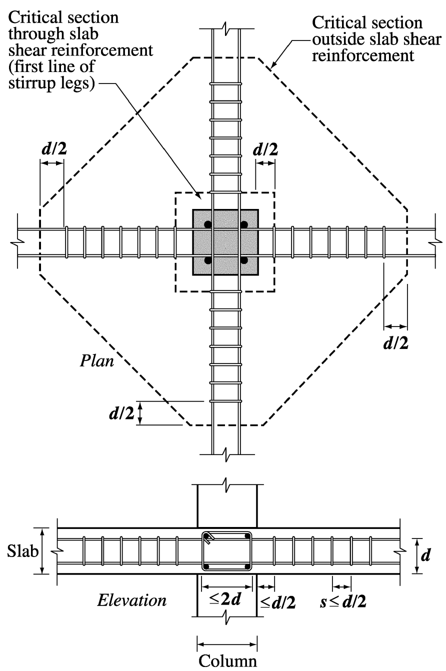

Fig. R11.12.3(d)—Arrangement of stirrup shear reinforcement, interior column.

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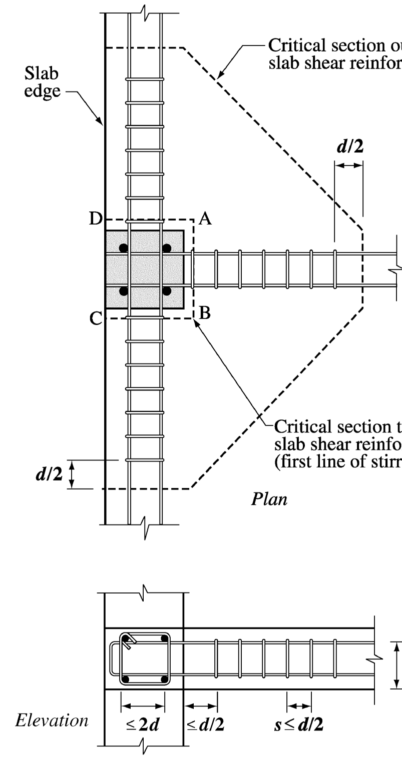

Fig. R11.12.3(e)—Arrangement of stirrup shear reinforcement, edge column.

11.12.4.5 — The ratio α_v between the flexural stiffness of each shearhead arm and that of the surrounding composite cracked slab section of width $(c_2 + d)$ shall not be less than 0.15.

11.12.4.6 — The plastic moment strength M_p required for each arm of the shearhead shall be computed by

$$M_p = \frac{V_u}{2\phi\eta}\left[h_v + \alpha_v\left(\ell_v - \frac{c_1}{2}\right)\right] \qquad (11\text{-}39)$$

where ϕ is the strength reduction factor for flexure, η is the number of arms, and ℓ_v is the minimum length of each shearhead arm required to comply with requirements of 11.12.4.7 and 11.12.4.8.

R11.12.4.5 and R11.12.4.6 — The assumed idealized shear distribution along an arm of a shearhead at an interior column is shown in Fig. R11.12.4.5. The shear along each of the arms is taken as $\alpha_v V_c/\eta$, where α_v and η are defined in 11.12.4.5 and 11.12.4.6, and V_c is defined in 11.12.2.1. However, the peak shear at the face of the column is taken as the total shear considered per arm $V_u/\phi\eta$ minus the shear considered carried to the column by the concrete compression zone of the slab. The latter term is expressed as $(V_c/\eta)(1 - \alpha_v)$, so that it approaches zero for a heavy shearhead and approaches $V_u/\phi\eta$ when a light shearhead is used. Equation (11-39) then follows from the assumption that the inclined cracking shear force V_c is about one-half the factored shear force V_u. In this equation, M_p is the required plastic moment strength of each shearhead arm necessary to assure that factored shear V_u is attained as the moment strength of the shearhead is reached. The quantity ℓ_v is the length from the center of the column to the point at which the shearhead is no longer required, and the distance $c_1/2$ is one-half the dimension of the column in the direction considered.

11.12.4.7 — The critical slab section for shear shall be perpendicular to the plane of the slab and shall cross each shearhead arm at three-quarters the distance $[\ell_v - (c_1/2)]$ from the column face to the end of the shearhead arm. The critical section shall be located so that its perimeter b_o is a minimum, but need not be closer than the perimeter defined in 11.12.1.2(a).

R11.12.4.7 — The test results indicated that slabs containing "underreinforcing" shearheads failed at a shear stress on a critical section at the end of the shearhead reinforcement less than $4\sqrt{f_c'}$. Although the use of "overreinforcing" shearheads brought the shear strength back to about the equivalent of $4\sqrt{f_c'}$, the limited test data suggest that a conservative design is desirable. Therefore, the shear

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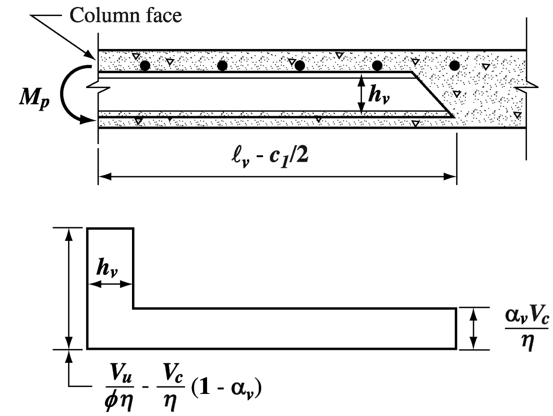

Fig. R11.12.4.5—Idealized shear acting on shearhead.

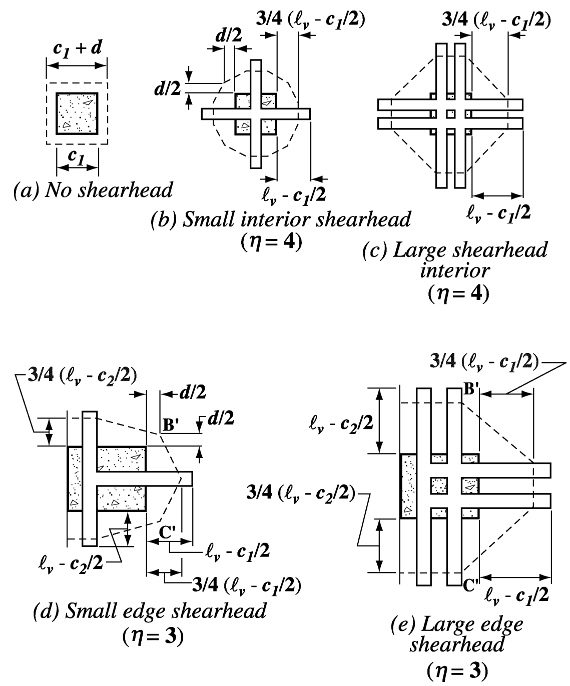

Fig. R11.12.4.7—Location of critical section defined in 11.12.4.7.

strength is calculated as $4\sqrt{f'_c}$ on an assumed critical section located inside the end of the shearhead reinforcement.

The critical section is taken through the shearhead arms three-fourths of the distance $[\ell_v - (c_1/2)]$ from the face of the column to the end of the shearhead. However, this assumed critical section need not be taken closer than $d/2$ to the column. See Fig. R11.12.4.7.

11.12.4.8 — V_n shall not be taken greater than $4\sqrt{f'_c}b_od$ on the critical section defined in 11.12.4.7. When shearhead reinforcement is provided, V_n shall not be taken greater than $7\sqrt{f'_c}b_od$ on the critical section defined in 11.12.1.2(a).

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11.12.4.9 — The moment resistance M_v contributed to each slab column strip by a shearhead shall not be taken greater than

$$M_v = \frac{\phi \alpha_v V_u}{2\eta}\left(\ell_v - \frac{c_1}{2}\right) \quad (11\text{-}40)$$

where ϕ is the strength reduction factor for flexure, η is the number of arms, and ℓ_v is the length of each shearhead arm actually provided. However, M_v shall not be taken larger than the smaller of:

(a) 30 percent of the total factored moment required for each slab column strip;

(b) The change in column strip moment over the length ℓ_v;

(c) The value of M_p computed by Eq. (11-39).

11.12.4.10 — When unbalanced moments are considered, the shearhead must have adequate anchorage to transmit M_p to column.

11.12.5 — Openings in slabs

When openings in slabs are located at a distance less than 10 times the slab thickness from a concentrated load or reaction area, or when openings in flat slabs are located within column strips as defined in Chapter 13, the critical slab sections for shear defined in 11.12.1.2 and 11.12.4.7 shall be modified as follows:

11.12.5.1 — For slabs without shearheads, that part of the perimeter of the critical section that is enclosed by straight lines projecting from the centroid of the column, concentrated load, or reaction area and tangent to the boundaries of the openings shall be considered ineffective.

11.12.5.2 — For slabs with shearheads, the ineffective portion of the perimeter shall be one-half of that defined in 11.12.5.1.

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R11.12.4.9 — If the peak shear at the face of the column is neglected, and the cracking load V_c is again assumed to be about one-half of V_u, the moment contribution of the shearhead M_v can be conservatively computed from Eq. (11-40), in which ϕ is the factor for flexure.

R11.12.4.10 — See R11.12.6.3.

R11.12.5 — Openings in slabs

Provisions for design of openings in slabs (and footings) were developed in Reference 11.3. The locations of the effective portions of the critical section near typical openings and free edges are shown by the dashed lines in Fig. R11.12.5. Additional research[11.51] has confirmed that these provisions are conservative.

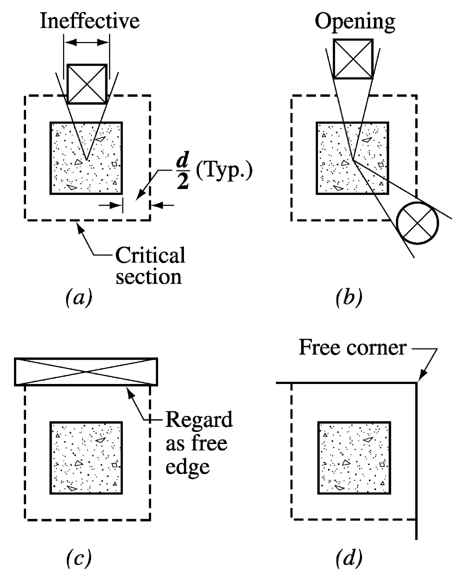

Fig. R11.12.5—Effect of openings and free edges (effective perimeter shown with dashed lines.

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11.12.6 — Transfer of moment in slab-column connections

11.12.6.1 — When gravity load, wind, earthquake, or other lateral forces cause transfer of unbalanced moment M_u between a slab and a column, a fraction $\gamma_f M_u$ of the unbalanced moment shall be transferred by flexure in accordance with 13.5.3. The remainder of the unbalanced moment given by $\gamma_v M_u$ shall be considered to be transferred by eccentricity of shear about the centroid of the critical section defined in 11.12.1.2 where

$$\gamma_v = (1 - \gamma_f) \quad (11\text{-}41)$$

11.12.6.2 — The shear stress resulting from moment transfer by eccentricity of shear shall be assumed to vary linearly about the centroid of the critical sections defined in 11.12.1.2. The maximum shear stress due to the factored shear force and moment shall not exceed ϕv_n:

(a) For members without shear reinforcement

$$\phi v_n = \phi V_c/(b_o d) \quad (11\text{-}42)$$

where V_c is as defined in 11.12.2.1 or 11.12.2.2.

(b) For members with shear reinforcement other than shearheads

$$\phi v_n = \phi(V_c + V_s)/(b_o d) \quad (11\text{-}43)$$

where V_c and V_s are defined in 11.12.3.1. The design shall take into account the variation of shear stress around the column. The shear stress due to factored shear force and moment shall not exceed $2\sqrt{f_c'}$ at the critical section located $d/2$ outside the outermost line of stirrup legs that surround the column.

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R11.12.6 — Transfer of moment in slab-column connections

R11.12.6.1 — In Reference 11.61, it was found that where moment is transferred between a column and a slab, 60 percent of the moment should be considered transferred by flexure across the perimeter of the critical section defined in 11.12.1.2, and 40 percent by eccentricity of the shear about the centroid of the critical section. For rectangular columns, it has been assumed that the portion of the moment transferred by flexure increases as the width of the face of the critical section resisting the moment increases as given by Eq. (13-1).

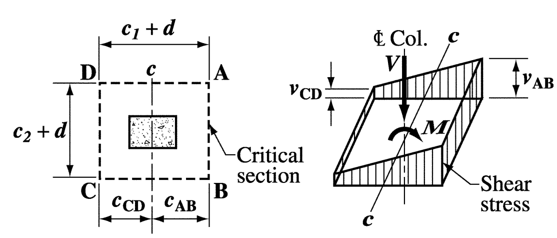
Most of the data in Reference 11.61 were obtained from tests of square columns, and little information is available for round columns. These can be approximated as square columns. Figure R13.6.2.5 shows square supports having the same area as some nonrectangular members.

R11.12.6.2 — The stress distribution is assumed as illustrated in Fig. R11.12.6.2 for an interior or exterior column. The perimeter of the critical section, *ABCD*, is determined in accordance with 11.12.1.2. The factored shear force V_u and unbalanced moment M_u are determined at the centroidal axis *c-c* of the critical section. The maximum factored shear stress may be calculated from

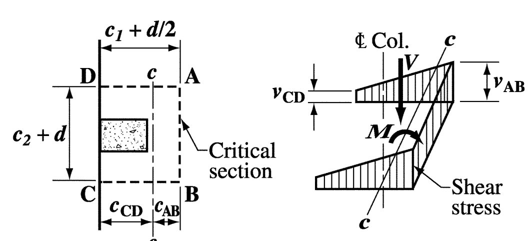
$$v_{u(AB)} = \frac{V_u}{A_c} + \frac{\gamma_v M_u c_{AB}}{J_c}$$

or

$$v_{u(CD)} = \frac{V_u}{A_c} - \frac{\gamma_v M_u c_{CD}}{J_c}$$



(a) Interior column



(b) Edge column

Fig. R11.12.6.2—Assumed distribution of shear stress.

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11.12.6.3 — When shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) is provided, the sum of the shear stresses due to vertical load acting on the critical section defined by 11.12.4.7 and the shear stresses resulting from moment transferred by eccentricity of shear about the centroid of the critical section defined in 11.12.1.2(a) and 11.12.1.3 shall not exceed $\phi 4\sqrt{f_c'}$.

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where γ_v is given by Eq. (11-41). For an interior column, A_c and J_c may be calculated by

A_c = area of concrete of assumed critical section
= $2d(c_1 + c_2 + 2d)$

J_c = property of assumed critical section analogous to polar moment of inertia

$$= \frac{d(c_1+d)^3}{6} + \frac{(c_1+d)d^3}{6} + \frac{d(c_2+d)(c_1+d)^2}{2}$$

Similar equations may be developed for A_c and J_c for columns located at the edge or corner of a slab.

The fraction of the unbalanced moment between slab and column not transferred by eccentricity of the shear must be transferred by flexure in accordance with 13.5.3. A conservative method assigns the fraction transferred by flexure over an effective slab width defined in 13.5.3.2. Often, designers concentrate column strip reinforcement near the column to accommodate this unbalanced moment. Available test data seem to indicate that this practice does not increase shear strength, but may be desirable to increase the stiffness of the slab-column junction.

Test data[11.62] indicate that the moment transfer capacity of a prestressed slab to column connection can be calculated using the procedures of 11.12.6 and 13.5.3.

Where shear reinforcement has been used, the critical section beyond the shear reinforcement generally has a polygonal shape (Fig. R11.12.3(d) and (e)). Equations for calculating shear stresses on such sections are given in Reference 11.58.

R11.12.6.3 — Tests[11.63] indicate that the critical section defined in 11.12.1.2(a) and 11.12.1.3 and are appropriate for calculations of shear stresses caused by transfer of moments even when shearheads are used. Then, even though the critical sections for direct shear and shear due to moment transfer differ, they coincide or are in close proximity at the column corners where the failures initiate. Because a shearhead attracts most of the shear as it funnels toward the column, it is conservative to take the maximum shear stress as the sum of the two components.

Section 11.12.4.10 requires the moment M_p transferred to the column in shearhead connections transferring unbalanced moments. This may be done by bearing within the column or positive mechanical anchorage.

CHAPTER 12 — DEVELOPMENT AND SPLICES OF REINFORCEMENT

CODE

12.0 — Notation

a = depth of equivalent rectangular stress block as defined in 10.2.7.1, in.

A_b = area of an individual bar, in.^{2}

A_s = area of nonprestressed tension reinforcement, in.^{2}

A_{tr} = total cross-sectional area of all transverse reinforcement that is within the spacing s and that crosses the potential plane of splitting through the reinforcement being developed, in.^{2}

A_v = area of shear reinforcement within a distance s, in.^{2}

A_w = area of an individual wire to be developed or spliced, in.^{2}

b_w = web width, or diameter of circular section, in.

c = spacing or cover dimension, in. See 12.2.4.

d = distance from extreme compression fiber to centroid of tension reinforcement, in.

d_b = nominal diameter of bar, wire, or prestressing strand, in.

f'_c = specified compressive strength of concrete, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi

f_{ps} = stress in prestressed reinforcement at nominal strength, ksi

f_{se} = effective stress in prestressed reinforcement (after allowance for all prestress losses), ksi

f_y = specified yield strength of nonprestressed reinforcement, psi

f_{yt} = specified yield strength of transverse reinforcement, psi

h = overall thickness of member, in.

K_{tr} = transverse reinforcement index

= $\dfrac{A_{tr} f_{yt}}{1500 sn}$ (constant 1500 carries the unit lb/in.^{2})

ℓ_a = additional embedment length at support or at point of inflection, in.

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The development length concept for anchorage of reinforcement was first introduced in the 1971 ACI Building Code, to replace the dual requirements for flexural bond and anchorage bond contained in earlier editions of the ACI Building Code. It is no longer necessary to consider the flexural bond concept that placed emphasis on the computation of nominal peak bond stresses. Consideration of an average bond resistance over a full development length of the reinforcement is more meaningful, partially because all bond tests consider an average bond resistance over a length of embedment of the reinforcement, and partially because uncalculated extreme variations in local bond stresses exist near flexural cracks.[12.1]

The development length concept is based on the attainable average bond stress over the length of embedment of the reinforcement. Development lengths are required because of the tendency of highly stressed bars to split relatively thin sections of restraining concrete. A single bar embedded in a mass of concrete should not require as great a development length; although a row of bars, even in mass concrete, can create a weakened plane, with longitudinal splitting along the plane of the bars.

In application, the development length concept requires minimum lengths or extensions of reinforcement beyond all points of peak stress in the reinforcement. Such peak stresses generally occur at the points specified in 12.10.2.

The strength reduction factor ϕ is not used in the development length and lap splice equations. An allowance for strength reduction is already included in the expressions for determining development and splice lengths.

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip.

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ℓ_d = development length of deformed bars and deformed wire in tension, in.

ℓ_{dc} = development length of deformed bars and deformed wire in compression, in.

ℓ_{dh} = development length of standard hook in tension, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus radius of bend and one bar diameter), in.

ℓ_{hb} = basic development length of standard hook in tension, in.

M_n = nominal moment strength at section, in.-lb
= $A_s f_y (d - a/2)$

n = number of bars or wires being spliced or developed along the plane of splitting

s = maximum center-to-center spacing of transverse reinforcement within ℓ_d, in.

s_w = spacing of wire to be developed or spliced, in.

V_n = nominal shear strength, lb

V_u = factored shear force at section, lb

α = reinforcement location factor. See 12.2.4

β = coating factor. See 12.2.4

β_b = ratio of area of reinforcement cut off to total area of tension reinforcement at section

γ = reinforcement size factor. See 12.2.4

λ = lightweight aggregate concrete factor. See 12.2.4

ϕ = strength reduction factor. See 9.3

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12.1 — Development of reinforcement—General

12.1.1 — Calculated tension or compression in reinforcement at each section of structural concrete members shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination thereof. Hooks shall not be used to develop bars in compression.

12.1.2 — The values of $\sqrt{f'_c}$ used in this chapter shall not exceed 100 psi.

R12.1 — Development of reinforcement—General

From a point of peak stress in reinforcement, some length of reinforcement or anchorage is necessary to develop the stress. This development length or anchorage is necessary on both sides of such peak stress points. Often the reinforcement continues for a considerable distance on one side of a critical stress point so that calculations need involve only the other side, for example, the negative moment reinforcement continuing through a support to the middle of the next span.

12.2 — Development of deformed bars and deformed wire in tension

R12.2 — Development of deformed bars and deformed wire in tension

The general development length equation (Eq. (12-1)) is given in 12.2.3. The equation is based on the expression for development length previously endorsed by Committee 408.[12.2,12.3] In Eq. (12-1), c is a factor that represents the

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smallest of the side cover, the cover over the bar or wire (in both cases measured to the center of the bar or wire), or one-half the center-to-center spacing of the bars or wires. K_{tr} is a factor that represents the contribution of confining reinforcement across potential splitting planes. α is the traditional reinforcement location factor to reflect the adverse effects of the top reinforcement casting position. β is a coating factor reflecting the effects of epoxy coating. There is a limit on the product $\alpha\beta$. γ is a reinforcement size factor that reflects the more favorable performance of smaller diameter reinforcement. λ is a factor reflecting the lower tensile strength of lightweight concrete and the resulting reduction of the splitting resistance, which increases the development length in lightweight concrete. A limit of 2.5 is placed on the term $(c + K_{tr})/d_b$. When $(c + K_{tr})/d_b$ is less than 2.5, splitting failures are likely to occur. For values above 2.5, a pullout failure is expected, and an increase in cover or transverse reinforcement is unlikely to increase the anchorage capacity.

Equation (12-1) allows the designer to see the effect of all variables controlling the development length. The designer is permitted to disregard terms when such omission results in longer and hence, more conservative, development lengths.

The provisions of 12.2.2 and 12.2.3 give a two-tier approach. The user can either calculate ℓ_d based on the actual $(c + K_{tr})/d_b$ (12.2.3) or calculate ℓ_d using 12.2.2, which is based on two preselected values of $(c + K_{tr})/d_b$.

Section 12.2.2 recognizes that many current practical construction cases utilize spacing and cover values along with confining reinforcement, such as stirrups or ties, that result in a value of $(c + K_{tr})/d_b$ of at least 1.5. Examples include a minimum clear cover of d_b along with either minimum clear spacing of $2d_b$, or a combination of minimum clear spacing of d_b and minimum ties or stirrups. For these frequently occurring cases, the development length for larger bars can be taken as $\ell_d = [f_y\alpha\beta\lambda/(20\sqrt{f_c'})]d_b$. In the development of ACI 318-95, a comparison with past provisions and a check of a database of experimental results maintained by ACI Committee 408[12.2] indicated that for No. 6 deformed bars and smaller, as well as for deformed wire, the development lengths could be reduced 20 percent using $\gamma = 0.80$. This is the basis for the middle column of the table in 12.2.2. With less cover and in the absence of minimum ties or stirrups, the minimum clear spacing limits of 7.6.1 and the minimum concrete cover requirements of 7.7 result in minimum values of c of d_b. Thus, for "other cases," the values are based on using $(c + K_{tr})/d_b = 1.0$ in Eq. (12-1).

The user may easily construct simple, useful expressions. For example, in all structures with normalweight concrete ($\lambda = 1.0$), uncoated reinforcement ($\beta = 1.0$), No. 7 or larger bottom bars ($\alpha = 1.0$) with $f_c' = 4$ ksi and Grade 60 reinforcement, the equations reduce to

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$$\ell_d = \frac{(60{,}000)(1.0)(1.0)(1.0)}{20\sqrt{4000}}d_b = 47d_b$$

or

$$\ell_d = \frac{3(60{,}000)(1.0)(1.0)(1.0)}{40\sqrt{4000}}d_b = 71d_b$$

Thus, as long as minimum cover of d_b is provided along with a minimum clear spacing of $2d_b$, or a minimum clear cover of d_b and a minimum clear spacing of d_b are provided along with minimum ties or stirrups, a designer knows that $\ell_d = 47d_b$. The penalty for spacing bars closer or providing less cover is the requirement that $\ell_d = 71d_b$.

Many practical combinations of side cover, clear cover, and confining reinforcement can be used with 12.2.3 to produce significantly shorter development lengths than allowed by 12.2.2. For example, bars or wires with minimum clear cover not less than $2d_b$ and minimum clear spacing not less than $4d_b$ and without any confining reinforcement would have a $(c + K_{tr})/d_b$ value of 2.5, and would require a development length of only $28d_b$ for the example above.

12.2.1 — Development length ℓ_d, in inches for deformed bars and deformed wire in tension shall be determined from either 12.2.2 or 12.2.3, but ℓ_d shall not be less than 12 in.

12.2.2 — For deformed bars or deformed wire, ℓ_d shall be as follows:

	No. 6 and smaller bars and deformed wires	No. 7 and larger bars
Clear spacing of bars being developed or spliced not less than d_b, clear cover not less than d_b, and stirrups or ties throughout ℓ_d not less than the code minimum or Clear spacing of bars being developed or spliced not less than $2d_b$ and clear cover not less than d_b	$\left(\frac{f_y\alpha\beta\lambda}{25\sqrt{f'_c}}\right)d_b$	$\left(\frac{f_y\alpha\beta\lambda}{20\sqrt{f'_c}}\right)d_b$
Other cases	$\left(\frac{3f_y\alpha\beta\lambda}{50\sqrt{f'_c}}\right)d_b$	$\left(\frac{3f_y\alpha\beta\lambda}{40\sqrt{f'_c}}\right)d_b$

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12.2.3 — For deformed bars or deformed wire, ℓ_d shall be

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\sqrt{f'_c}} \frac{\alpha\beta\gamma\lambda}{\left(\frac{c + K_{tr}}{d_b}\right)} \right) d_b \quad (12\text{-}1)$$

in which the term $(c + K_{tr})/d_b$ shall not be taken greater than 2.5.

12.2.4 — The factors for use in the expressions for development of deformed bars and deformed wires in tension in Chapter 12 are as follows:

α = reinforcement location factor
Horizontal reinforcement so placed that more than 12 in. of fresh concrete is cast in the member below the development length or splice 1.3
Other reinforcement... 1.0

β = coating factor
Epoxy-coated bars or wires with cover less than $3d_b$, or clear spacing less than $6d_b$............................. 1.5
All other epoxy-coated bars or wires 1.2
Uncoated reinforcement 1.0

However, the product of $\alpha\beta$ need not be taken greater than 1.7.

γ = reinforcement size factor
No. 6 and smaller bars and deformed wires......... 0.8
No. 7 and larger bars... 1.0

λ = lightweight aggregate concrete factor
When lightweight aggregate concrete is used...... 1.3

However, when f_{ct} is specified, λ shall be permitted to be taken as $6.7\sqrt{f'_c}/f_{ct}$
but not less than 1.0
When normalweight concrete is used 1.0

c = spacing or cover dimension, in.

Use the smaller of either the distance from the center of the bar or wire to the nearest concrete surface or one-half the center-to-center spacing of the bars or wires being developed.

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1500 sn}$$

It shall be permitted to use $K_{tr} = 0$ as a design simplification even if transverse reinforcement is present.

12.2.5 — Excess reinforcement

Reduction in development length shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage

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R12.2.4 — The reinforcement location factor α accounts for position of the reinforcement in freshly placed concrete. The factor was reduced to 1.3 in the 318-89 code to reflect recent research.[12.4,12.5]

The factor λ for lightweight aggregate concrete was made the same for all types of lightweight aggregates in the 318-89 code. Research on hooked bar anchorages did not support the variations in previous codes for all-lightweight and sand-lightweight concrete and a single value, 1.3, was selected. Section 12.2.4 allows a lower factor to be used when the splitting tensile strength of the lightweight concrete is specified. See ACI 318-02 Code 5.1.4.

Studies[12.6-12.8] of the anchorage of epoxy-coated bars show that bond strength is reduced because the coating prevents adhesion and friction between the bar and the concrete. The factors reflect the type of anchorage failure likely to occur. When the cover or spacing is small, a splitting failure can occur, and the anchorage or bond strength is substantially reduced. If the cover and spacing between bars is large, a splitting failure is precluded, and the effect of the epoxy coating on anchorage strength is not as large. Studies[12.9] have shown that although the cover or spacing may be small, the anchorage strength may be increased by adding transverse steel crossing the plane of splitting, and restraining the splitting crack.

Because the bond of epoxy-coated bars is already reduced due to the loss of adhesion between the bar and the concrete, an upper limit of 1.7 is established for the product of the top reinforcement and epoxy-coated reinforcement factors.

Although there is no requirement for transverse reinforcement along the tension development or splice length, recent research[12.10,12.11] indicates that in concrete with very high compressive strength, brittle anchorage failure occurred in bars with inadequate transverse reinforcement. In splice tests of No. 8 and No. 11 bars in concrete with an f'_c of approximately 15,000 psi, transverse reinforcement improved ductile anchorage behavior.

R12.2.5 — Excess reinforcement

The reduction factor based on area is not to be used in those cases where anchorage development for full f_y is required. For example, the excess reinforcement factor does not apply

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or development for f_y is specifically required or the reinforcement is designed under provisions of 11.7.4 or 21.2.1.5

$(A_s \text{ required})/(A_s \text{ provided})$

12.3 — Development of deformed bars and deformed wire in compression

12.3.1 — Development length ℓ_{dc}, in inches, for deformed bars and deformed wire in compression shall be determined from 12.3.2 and applicable modification factors of 12.3.3, but ℓ_{dc} shall not be less than 8 in.

12.3.2 — For deformed bars and deformed wire, ℓ_{dc} shall be taken as the larger of $(0.02 f_y / \sqrt{f_c'}) d_b$ and $(0.0003 f_y) d_b$, where the constant 0.0003 carries the unit of in.^{2}/lb.

12.3.3 — The length ℓ_{dc} in 12.3.2 shall be permitted to be multiplied by the applicable factors for:

(a) Reinforcement in excess of that required by analysis .... $(A_s \text{ required})/(A_s \text{ provided})$

(b) Reinforcement enclosed within spiral reinforcement not less than 1/4 in. diameter and not more than 4 in. pitch or within No. 4 ties in conformance with 7.10.5 and spaced at not more than 4 in. on center 0.75

12.4 — Development of bundled bars

12.4.1 — Development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased 20 percent for three-bar bundle, and 33 percent for four-bar bundle.

12.4.2 — For determining the appropriate factors in 12.2, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

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for development of positive moment reinforcement at supports according to 12.11.2, for development of shrinkage and temperature reinforcement according to 7.12.2.3, or for development of reinforcement provided according to 7.13 and 13.3.8.5.

R12.3 — Development of deformed bars and deformed wire in compression

The weakening effect of flexural tension cracks is not present for bars and wire in compression and usually end bearing of the bars on the concrete is beneficial. Therefore, shorter development lengths are specified for compression than for tension. The development length may be reduced 25 percent when the reinforcement is enclosed within spirals or ties. A reduction in development length is also permitted if excess reinforcement is provided.

R12.4 — Development of bundled bars

R12.4.1 — An increased development length for individual bars is required when three or four bars are bundled together. The extra extension is needed because the grouping makes it more difficult to mobilize bond resistance from the core between the bars.

The designer should also note 7.6.6.4 relating to the cutoff points of individual bars within a bundle and 12.14.2.2 relating to splices of bundled bars. The increases in development length of 12.4 do apply when computing splice lengths of bundled bars in accordance with 12.14.2.2. The development of bundled bars by a standard hook of the bundle is not covered by the provisions of 12.5.

R12.4.2 — Although splice and development lengths of bundled bars are based on the diameter of individual bars increased by 20 or 33 percent as appropriate, it is necessary to use an equivalent diameter of the entire bundle derived from the equivalent total area of bars when determining factors in 12.2, which considers cover and clear spacing and represents the tendency of concrete to split.

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12.5 — Development of standard hooks in tension

12.5.1 — Development length ℓ_{dh}, in inches, for deformed bars in tension terminating in a standard hook (see 7.1) shall be determined from 12.5.2 and the applicable modification factors of 12.5.3, but ℓ_{dh} shall not be less than $8d_b$ nor less than 6 in.

12.5.2 — For deformed bars, ℓ_{dh} shall be $(0.02\beta\lambda f_y/\sqrt{f'_c})d_b$ with β taken as 1.2 for epoxy-coated reinforcement, and λ taken as 1.3 for lightweight aggregate concrete. For other cases, β and λ shall be taken as 1.0.

12.5.3 — The length ℓ_{dh} in 12.5.2 shall be permitted to be multiplied by the following applicable factors:

(a) For No. 11 bar and smaller hooks with side cover (normal to plane of hook) not less than 2-1/2 in., and for 90-deg hook with cover on bar extension beyond hook not less than 2 in. 0.7

(b) For 90-deg hooks of No. 11 and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length ℓ_{dh} of the hook; or enclosed within ties or stirrups parallel to the bar being developed, spaced not greater than $3d_b$ along the length of the tail extension of the hook plus bend 0.8

(c) For 180-deg hooks of No. 11 and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length ℓ_{dh}, of the hook 0.8

(d) Where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis........ (A_s required)/(A_s provided)

In 12.5.3(b) and 12.5.3(c), d_b is the diameter of the hooked bar, and the first tie or stirrup shall enclose the bent portion of the hook, within $2d_b$ of the outside of the bend.

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R12.5 — Development of standard hooks in tension

The provisions for hooked bar anchorage were extensively revised in the ACI 318-83 code. Study of failures of hooked bars indicate that splitting of the concrete cover in the plane of the hook is the primary cause of failure, and that splitting originates at the inside of the hook where the local stress concentrations are very high. Thus, hook development is a direct function of bar diameter d_b that governs the magnitude of compressive stresses on the inside of the hook. Only standard hooks (see 7.1) are considered and the influence of larger radius of bend cannot be evaluated by 12.5.

The hooked bar anchorage provisions give the total hooked bar embedment length as shown in Fig. R12.5. The development length ℓ_{dh} is measured from the critical section to the outside end (or edge) of the hook.

The development length for standard hooks ℓ_{hb} of 12.5.2 can be reduced by all applicable modification factors of 12.5.3. As an example, if the conditions of both 12.5.3(a) and (c) are met, both factors may be applied.

The effects of bar yield strength, excess reinforcement, lightweight concrete, and factors to reflect the resistance to splitting provided from confinement by concrete and transverse ties or stirrups are based on recommendations from References 12.2 and 12.3.

Tests[12.12] indicate that closely spaced ties at or near the bend portion of a hooked bar are most effective in confining the hooked bar. For construction purposes, this is not always practicable. The cases where the modification factor of 12.5.3(b) may be used are illustrated in Fig. R12.5.3(a) and (b). Figure R12.5.3(a) shows placement of ties or stirrups perpendicular to the bar being developed, spaced along the development length, ℓ_{dh}, of the hook. Figure R12.5.3(b) shows placement of ties or stirrups parallel to the bar being

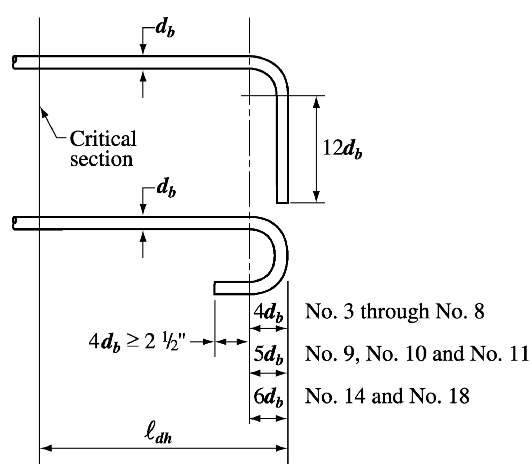

Fig. R12.5—Hooked bar details for development of standard hooks.

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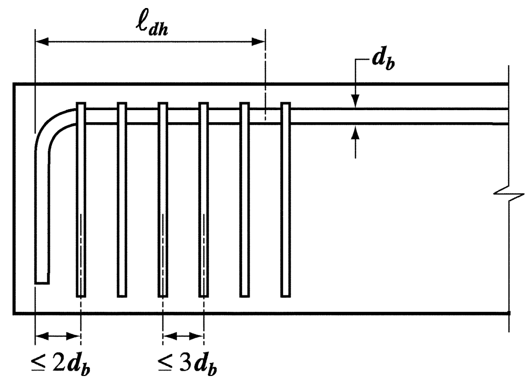

Fig. R12.5.3(a)—Ties or stirrups placed perpendicular to the bar being developed, spaced along the development length ℓ_{dh}.

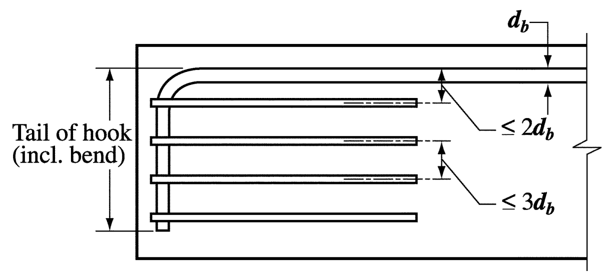

Fig. R12.5.3(b)—Ties or stirrups placed parallel to the bar being developed, spaced along the length of the tail extension of the hook plus bend.

developed along the length of the tail extension of the hook plus bend. The latter configuration would be typical in a beam column joint.

The factor for excess reinforcement in 12.5.3(d) applies only where anchorage or development for full f_y is not specifically required. The λ factor for lightweight concrete is a simplification over the procedure in 12.2.3.3 of ACI 318-83 in which the increase varies from 18 to 33 percent, depending on the amount of lightweight aggregate used. Unlike straight bar development, no distinction is made between top bars and other bars; such a distinction is difficult for hooked bars in any case. A minimum value of ℓ_{dh} is specified to prevent failure by direct pullout in cases where a hook may be located very near the critical section. Hooks cannot be considered effective in compression.

Tests[12.13] indicate that the development length for hooked bars should be increased by 20 percent to account for reduced bond when reinforcement is epoxy coated.

12.5.4 — For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 2-1/2 in., the hooked bar shall be enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length ℓ_{dh} of the hook. The first tie or stirrup shall enclose the bent portion of the hook, within $2d_b$ of the

R12.5.4 — Bar hooks are especially susceptible to a concrete splitting failure if both side cover (normal to plane of hook) and top or bottom cover (in plane of hook) are small. See Fig. R12.5.4. With minimum confinement provided by concrete, additional confinement provided by ties or stirrups is essential, especially if full bar strength should be developed by a hooked bar with such small cover. Cases where hooks may require ties or stirrups for confine-

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outside of the bend, where d_b is the diameter of the hooked bar. For this case, the factors of 12.5.3(b) and (c) shall not apply.

12.5.5 — Hooks shall not be considered effective in developing bars in compression.

12.6 — Mechanical anchorage

12.6.1 — Any mechanical device capable of developing the strength of reinforcement without damage to concrete is allowed as anchorage.

12.6.2 — Test results showing adequacy of such mechanical devices shall be presented to the building official.

12.6.3 — Development of reinforcement shall be permitted to consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between the point of maximum bar stress and the mechanical anchorage.

12.7 — Development of welded deformed wire fabric in tension

12.7.1 — Development length ℓ_d, in inches, of welded deformed wire fabric measured from the point of critical section to the end of wire shall be computed as the product of the development length ℓ_d, from 12.2.2 or 12.2.3 times a wire fabric factor from 12.7.2 or 12.7.3. It shall be permitted to reduce the development length in accordance with 12.2.5 when applicable, but ℓ_d shall not be less than 8 in. except in computation of lap splices by 12.18. When using the wire fabric factor from 12.7.2, it shall be permitted to use an epoxy-coating factor β of 1.0 for epoxy-coated welded wire fabric in 12.2.2 and 12.2.3.

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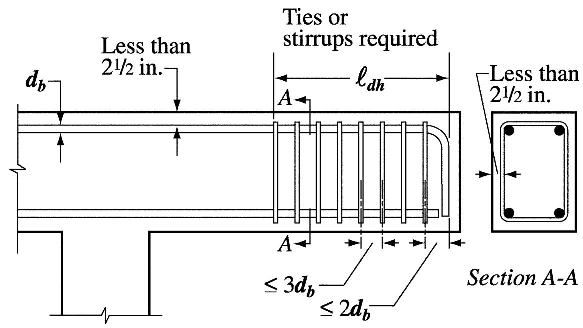

Fig. R12.5.4—Concrete cover per 12.5.4.

ment are at ends of simply supported beams, at free end of cantilevers, and at ends of members framing into a joint where members do not extend beyond the joint. In contrast, if calculated bar stress is so low that the hook is not needed for bar anchorage, the ties or stirrups are not necessary. Also, provisions of 12.5.4 do not apply for hooked bars at discontinuous ends of slabs with confinement provided by the slab continuous on both sides normal to the plane of the hook.

R12.5.5 — In compression, hooks are ineffective and may not be used as anchorage.

R12.6 — Mechanical anchorage

R12.6.1 — Mechanical anchorage can be made adequate for strength both for tendons and for bar reinforcement.

R12.6.3 — Total development of a bar consists of the sum of all the parts that contribute to anchorage. When a mechanical anchorage is not capable of developing the required design strength of the reinforcement, additional embedment length of reinforcement should be provided between the mechanical anchorage and the critical section.

R12.7 — Development of welded deformed wire fabric in tension

Figure R12.7 shows the development requirements for deformed wire fabric with one cross wire within the development length. ASTM A 497 for deformed wire fabric requires the same strength of the weld as required for plain wire fabric (ASTM A 185). Some of the development is assigned to welds, and some is assigned to the length of deformed wire. The development computations are simplified from earlier code provisions for wire development by assuming that only one cross wire is contained in the development length. The factors in 12.7.2 are applied to the deformed wire development length computed from 12.2, but with an absolute minimum of 8 in. The explicit statement

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12.7.2 — For welded deformed wire fabric with at least one cross wire within the development length and not less than 2 in. from the point of the critical section, the wire fabric factor shall be the greater of

$$\left(\frac{f_y - 35{,}000}{f_y}\right)$$

or

$$\left(\frac{5d_b}{s_w}\right)$$

but need not be greater than 1.

12.7.3 — For welded deformed wire fabric with no cross wires within the development length or with a single cross wire less than 2 in. from the point of the critical section, the wire fabric factor shall be taken as 1, and the development length shall be determined as for deformed wire.

12.7.4 — When any plain wires are present in the deformed wire fabric in the direction of the development length, the fabric shall be developed in accordance with 12.8.

12.8 — Development of welded plain wire fabric in tension

Yield strength of welded plain wire fabric shall be considered developed by embedment of two cross wires with the closer cross wire not less than 2 in. from the point of the critical section. However, the development length ℓ_d, in inches, measured from the point of the critical section to the outermost cross wire shall not be less than

$$0.27\frac{A_w}{s_w}\left(\frac{f_y}{\sqrt{f'_c}}\right)\lambda$$

except that when reinforcement provided is in excess of that required, this length shall be permitted to be reduced in accordance with 12.2.5. ℓ_d shall not be less than 6 in. except in computation of lap splices by 12.19.

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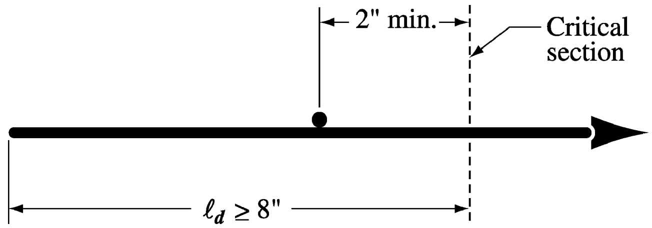

Fig. R12.7—Development of welded deformed wire fabric.

that the mesh multiplier not be taken greater than 1 corrects an oversight in earlier codes. The multipliers were derived using the general relationships between deformed wire mesh and deformed wires in the ℓ_{db} values of ACI 318-83.

Tests[12.14] have indicated that epoxy-coated welded wire fabric has essentially the same development and splice strengths as uncoated fabric because the cross wires provide the primary anchorage for the wire. Therefore, an epoxy-coating factor of 1.0 is used for development and splice lengths of epoxy-coated welded wire fabric with cross wires within the splice or development length.

R12.8 — Development of welded plain wire fabric in tension

Figure R12.8 shows the development requirements for plain wire fabric with development primarily dependent on the location of cross wires. For fabrics made with the smaller wires, an embedment of at least two cross wires 2 in. or more beyond the point of critical section is adequate to develop the full yield strength of the anchored wires. For fabrics made with larger closely spaced wires, however, a longer embedment is required, and a minimum development length is provided for these fabrics.

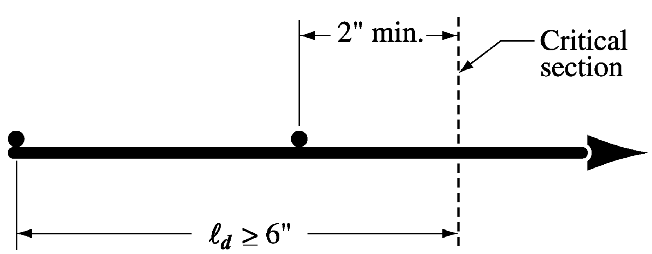

Fig. R12.8—Development of welded plain wire fabric.

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12.9 — Development of prestressing strand

12.9.1 — Except as provided in 12.9.1.1, seven-wire strand shall be bonded beyond the critical section for a development length ℓ_d, in inches, not less than

$$\ell_d = \left(\frac{f_{se}}{3}\right) d_b + (f_{ps} - f_{se}) d_b \qquad (12\text{-}2)$$

where d_b is strand diameter in inches, and f_{ps} and f_{se} are expressed in kips/in.^{2} The expressions in parenthesis are used as a constant without units.

12.9.1.1 — Embedment less than the development length shall be permitted at a section of a member provided the design strand stress at that section does not exceed values obtained from the bilinear relationship defined by Eq. (12-2).

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R12.9 — Development of prestressing strand

The development requirements for prestressing strand are intended to provide bond integrity for the strength of the member. The provisions are based on tests performed on normalweight concrete members with a minimum cover of 2 in. These tests may not represent the behavior of strand in low water-cementitious materials ratio, no-slump concrete. Fabrication methods should ensure consolidation of concrete around the strand with complete contact between the steel and concrete. Extra precautions should be exercised when low water-cementitious materials ratio, no-slump concrete is used. In general, this section will control only for the design of cantilever and short-span members.

The first term in Eq. (12-2) represents the transfer length of the strand, that is, the distance over which the strand should be bonded to the concrete to develop the prestress f_{se} in the strand. The second term represents the additional length over which the strand should be bonded so that a stress f_{ps} may develop in the strand at nominal strength of the member.

The bond of strand is a function of a number of factors, including the configuration and surface condition of the steel, the stress in the steel, the depth of concrete beneath the strand, and the method used to transfer the force in the strand to the concrete. For bonded applications, quality assurance procedures should be used to confirm that the strand is capable of adequate bond.[12.15,12.16] The precast concrete manufacturer may rely on certification from the strand manufacturer that the strand has bond characteristics that comply with this section. Strand with a slightly rusted surface can have an appreciably shorter transfer length than clean strand. Gentle release of the strand will permit a shorter transfer length than abruptly cutting the strands.

The provisions of 12.9 do not apply to plain wires or to end anchored tendons. The length for smooth wire could be expected to be considerably greater due to the absence of mechanical interlock. Flexural bond failure would occur with plain wire when first slip occurred.

R12.9.1.1 — Figure R12.9 shows the relationship between steel stress and the distance over which the strand is bonded to the concrete represented by Eq. (12-2). This idealized variation of strand stress may be used for analyzing sections within the development region.[12.17,12.18]

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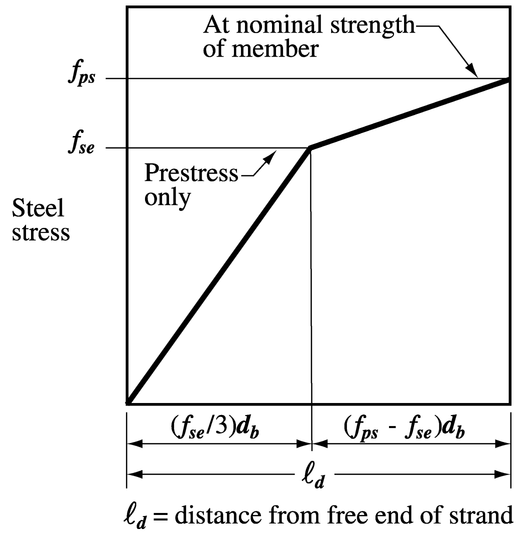

Fig. R12.9—Idealized bilinear relationship between steel stress and distance from the free end of the strand.

The expressions for transfer length, and for the additional bonded length necessary to develop an increase in stress of $(f_{ps} - f_{se})$, are based on tests of members prestressed with clean, 1/4, 3/8, and 1/2 in. diameter strands for which the maximum value of f_{ps} was 275 kips/in.^{2} See References 12.19, 12.20, and 12.21.

12.9.2 — Limiting the investigation to cross sections nearest each end of the member that are required to develop full design strength under specified factored loads shall be permitted except where bonding of one or more strands does not extend to the end of the member, or where concentrated loads are applied in the strand development length.

R12.9.2 — Where bonding of one or more strands does not extend to the end of the member, critical sections may be at locations other than where full design strength is required to be developed, and detailed analysis may be required. References 12.17 and 12.18 show a method that may be used in the case of strands with different points of full development. Conservatively, only the strands that are fully developed at a section may be considered effective at that section. If critical sections occur in the transfer region, special considerations may be necessary. Some loading conditions, such as where heavy concentrated loads occur within the strand development length, may cause critical sections to occur away from the section that is required to develop full design strength.

12.9.3 — Where bonding of a strand does not extend to end of member, and design includes tension at service load in precompressed tensile zone as permitted by 18.4.2, development length specified in 12.9.1 shall be doubled.

R12.9.3 — Exploratory tests conducted in 1965[12.19] that studied the effect of debonded strand (bond not permitted to extend to the ends of members) on performance of pretensioned girders indicated that the performance of these girders with embedment lengths twice those required by 12.9.1 closely matched the flexural performance of similar pretensioned girders with strand fully bonded to ends of girders. Accordingly, doubled development length is required for strand not bonded through to the end of a member. Subsequent tests[12.22] indicated that in pretensioned members designed for zero tension in the concrete under service load conditions (see 18.4.2), the development length for debonded strands need not be doubled. For analysis of sections with debonded strands at locations where strand is not fully developed, it is usually assumed that both the transfer length and development length are doubled.

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12.10 — Development of flexural reinforcement—General

12.10.1 — Development of tension reinforcement by bending across the web to be anchored or made continuous with reinforcement on the opposite face of member shall be permitted.

12.10.2 — Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent. Provisions of 12.11.3 must be satisfied.

12.10.3 — Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of member or $12d_b$, whichever is greater, except at supports of simple spans and at free end of cantilevers.

COMMENTARY

R12.10 — Development of flexural reinforcement—General

R12.10.2 — Critical sections for a typical continuous beam are indicated with a "c" or an "x" in Fig. R12.10.2. For uniform loading, the positive reinforcement extending into the support is more apt to be governed by the requirements of 12.11.3 rather than by development length measured from a point of maximum moment or bar cutoff.

R12.10.3 — The moment diagrams customarily used in design are approximate; some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, lateral loads, or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location of the calculated tensile stress approximately a distance d towards a point of zero moment. When stirrups are provided, this effect is less severe, although still present to some extent.

To provide for shifts in the location of maximum moments, the code requires the extension of reinforcement a distance

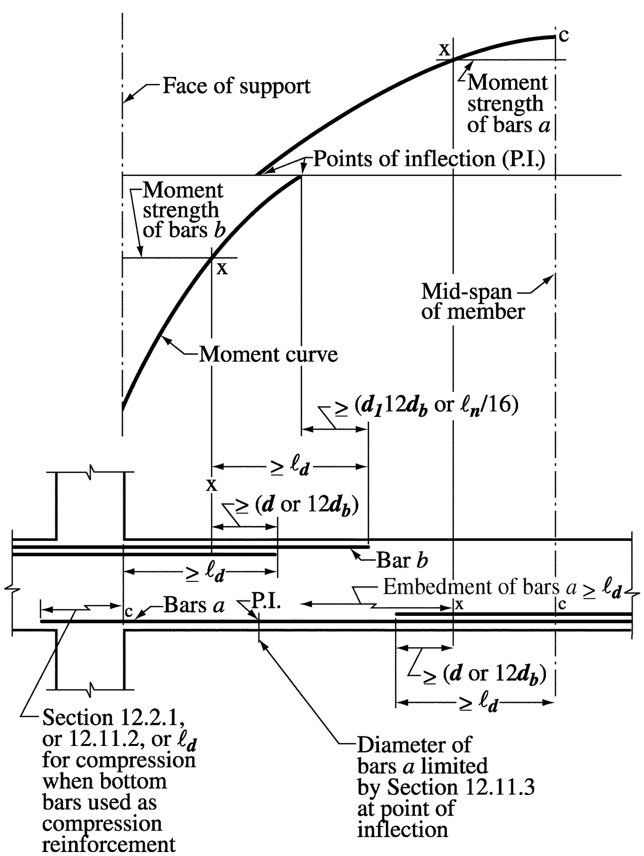


Fig. R12.10.2—Development of flexural reinforcement in a typical continuous beam.

CODE

COMMENTARY

d or $12d_b$ beyond the point at which it is theoretically no longer required to resist flexure, except as noted. Cutoff points of bars to meet this requirement are illustrated in Fig. R12.10.2.

When bars of different sizes are used, the extension should be in accordance with the diameter of bar being terminated. A bar bent to the far face of a beam and continued there may logically be considered effective, in satisfying this section, to the point where the bar crosses the mid-depth of the member.

12.10.4 — Continuing reinforcement shall have an embedment length not less than the development length ℓ_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

R12.10.4 — Peak stresses exist in the remaining bars wherever adjacent bars are cut off, or bent, in tension regions. In Fig. R12.10.2, an "x" is used to indicate the peak stress points remaining in continuing bars after part of the bars have been cut off. If bars are cut off as short as the moment diagrams allow, these peak stresses become the full f_y, which requires a full ℓ_d extension as indicated. This extension may exceed the length required for flexure.

12.10.5 — Flexural reinforcement shall not be terminated in a tension zone unless 12.10.5.1, 12.10.5.2, or 12.10.5.3 is satisfied:

12.10.5.1 — Factored shear at the cutoff point does not exceed two-thirds of the design shear strength ϕV_n.

12.10.5.2 — Stirrup area in excess of that required for shear and torsion is provided along each terminated bar or wire over a distance from the termination point equal to three-fourths the effective depth of member. Excess stirrup area A_v shall be not less than $60b_ws/f_y$. Spacing s shall not exceed $d/8\beta_b$.

12.10.5.3 — For No. 11 bars and smaller, continuing reinforcement provides double the area required for flexure at the cutoff point and factored shear does not exceed three-fourths the design shear strength ϕV_n.

R12.10.5 — Reduced shear strength and loss of ductility when bars are cut off in a tension zone, as in Fig. R12.10.2, have been reported. The code does not permit flexural reinforcement to be terminated in a tension zone unless special conditions are satisfied. Flexure cracks tend to open early wherever any reinforcement is terminated in a tension zone. If the steel stress in the continuing reinforcement and the shear strength are each near their limiting values, diagonal tension cracking tends to develop prematurely from these flexure cracks. Diagonal cracks are less likely to form where shear stress is low (see 12.10.5.1). Diagonal cracks can be restrained by closely spaced stirrups (see 12.10.5.2). A lower steel stress reduces the probability of such diagonal cracking (see 12.10.5.3). These requirements are not intended to apply to tension splices that are covered by 12.2, 12.13.5, and the related 12.15.

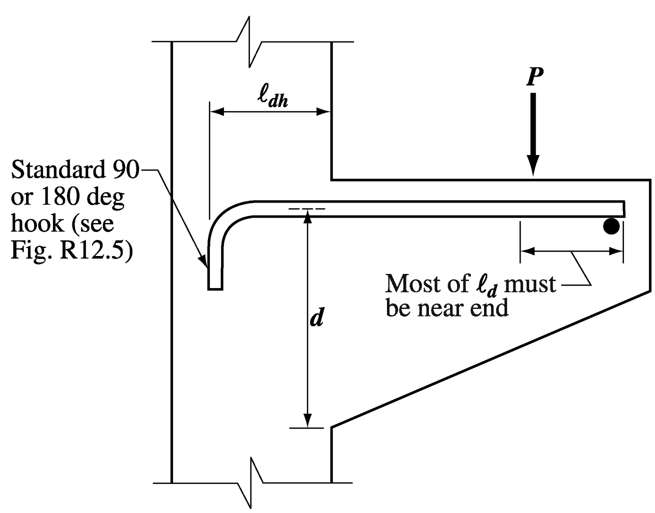


Fig. R12.10.6 — Special member largely dependent on end anchorage.

CODE

12.10.6 — Adequate anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets; deep flexural members; or members in which tension reinforcement is not parallel to compression face. See 12.11.4 and 12.12.4 for deep flexural members.

12.11 — Development of positive moment reinforcement

12.11.1 — At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 6 in.

12.11.2 — When a flexural member is part of a primary lateral load resisting system, positive moment reinforcement required to be extended into the support by 12.11.1 shall be anchored to develop the specified yield strength f_y in tension at the face of support.

12.11.3 — At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that ℓ_d computed for f_y by 12.2 satisfies Eq. (12-3); except, Eq. (12-3) need not be satisfied for reinforcement terminating beyond centerline of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook

$$\ell_d \leq \frac{M_n}{V_u} + \ell_a \qquad (12\text{-}3)$$

where M_n is nominal moment strength assuming all reinforcement at the section to be stressed to the specified yield strength f_y; V_u is factored shear force at the section; ℓ_a at a support shall be the embedment length beyond center of support; or ℓ_a at a point of inflection shall be limited to the effective depth of member or $12d_b$, whichever is greater.

COMMENTARY

R12.10.6 — Brackets, members of variable depth, and other members where steel stress f_s does not decrease linearly in proportion to a decreasing moment require special consideration for proper development of the flexural reinforcement. For the bracket shown in Fig. R12.10.6, the stress at ultimate in the reinforcement is almost constant at approximately f_y from the face of support to the load point. In such a case, development of the flexural reinforcement depends largely on the end anchorage provided at the loaded end. Reference 12.1 suggests a welded cross bar of equal diameter as a means of providing effective end anchorage. An end hook in the vertical plane, with the minimum diameter bend, is not totally effective because an essentially plain concrete corner will exist near loads applied close to the corner. For wide brackets (perpendicular to the plane of the figure) and loads not applied close to the corners, U-shaped bars in a horizontal plane provide effective end hooks.

R12.11 — Development of positive moment reinforcement

R12.11.1 — Positive moment reinforcement is carried into the support to provide for some shifting of the moments due to changes in loading, settlement of supports, and lateral loads.

R12.11.2 — When a flexural member is part of a primary lateral-load-resisting system, loads greater than those anticipated in design may cause reversal of moment at supports; some positive reinforcement should be well anchored into the support. This anchorage is required to assure ductility of response in the event of serious overstress, such as from blast or earthquake. It is not sufficient to use more reinforcement at lower stresses.

R12.11.3 — At simple supports and points of inflection such as "PI" in Fig. R12.10.2, the diameter of the positive reinforcement should be small enough so that computed development length of the bar ℓ_d does not exceed $M_n/V_u + \ell_a$, or under favorable support conditions, $1.3M_n/V_u + \ell_a$. Figure R12.11.3(a) illustrates the use of the provision.

At the point of inflection, the value of ℓ_a should not exceed the actual bar extension used beyond the point of zero moment. The M_n/V_u portion of the available length is a theoretical quantity not generally associated with an obvious maximum stress point. M_n is the nominal strength of the cross section without the ϕ-factor, and is not the applied factored moment.

The length M_n/V_u corresponds to the development length for the maximum size bar obtained from the previously used flexural bond equation $\Sigma_o = V/ujd$, where u is bond stress, and jd is the moment arm. In the 1971 ACI Building Code, this anchorage requirement was relaxed from previous

CODE

An increase of 30 percent in the value of M_n/V_u shall be permitted when the ends of reinforcement are confined by a compressive reaction.

12.11.4 — At simple supports of deep beams, positive moment tension reinforcement shall be anchored to develop its specified yield strength, f_y, in tension at the face of support. At interior supports of deep beams, positive moment tension reinforcement shall be continuous or be spliced with that of the adjacent spans.

COMMENTARY

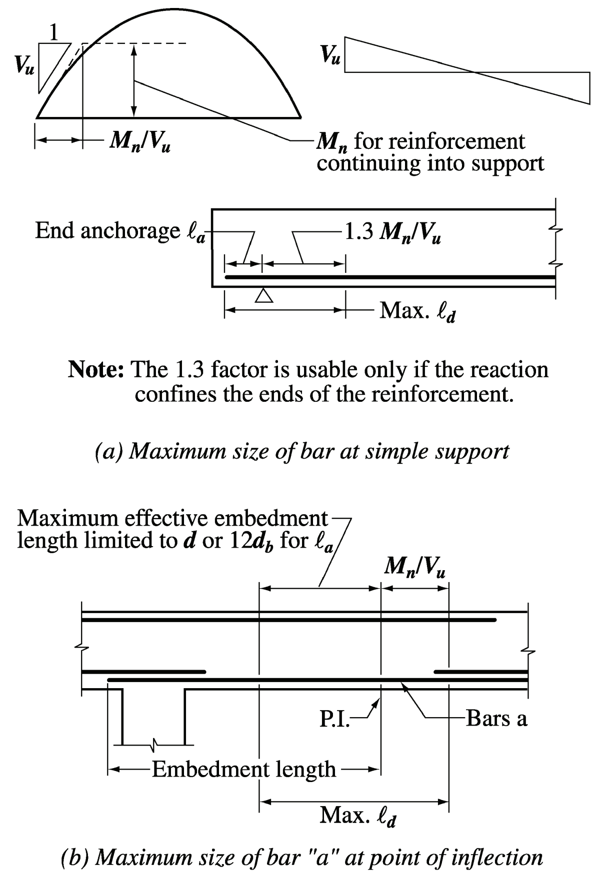


Fig. R12.11.3—Concept for determining maximum bar size per 12.11.3.

codes by crediting the available end anchorage length ℓ_a and by including a 30 percent increase for M_n/V_u when the ends of the reinforcement are confined by a compressive reaction.

For example, a bar size is provided at a simple support such that ℓ_d is computed in accordance with 12.2. The bar size provided is satisfactory only if computed ℓ_d does not exceed $1.3M_n/V_u + \ell_a$.

The ℓ_a to be used at points of inflection is limited to the effective depth of the member d or 12 bar diameters ($12d_b$), whichever is greater. Figure R12.11.3(b) illustrates this provision at points of inflection. The ℓ_a limitation is added because test data are not available to show that a long end anchorage length will be fully effective in developing a bar that has only a short length between a point of inflection and a point of maximum stress.

R12.11.4 — The use of the strut-and-tie model for the design of reinforced concrete deep flexural members clarifies that there is significant tension in the reinforcement at the face of the support. This requires the tension reinforcement to be continuous or be developed through and beyond the support.[12.23]

CODE

12.12 — Development of negative moment reinforcement

12.12.1 — Negative moment reinforcement in a continuous, restrained, or cantilever member, or in any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks, or mechanical anchorage.

12.12.2 — Negative moment reinforcement shall have an embedment length into the span as required by 12.1 and 12.10.3.

12.12.3 — At least one-third the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than effective depth of member, $12d_b$, or one-sixteenth the clear span, whichever is greater.

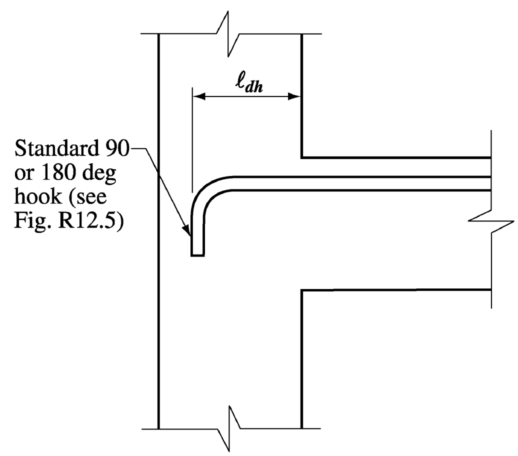
12.12.4 — At interior supports of deep flexural members, negative moment tension reinforcement shall be continuous with that of the adjacent spans.

COMMENTARY

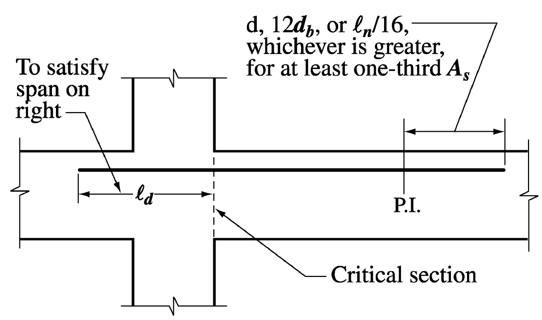
R12.12 — Development of negative moment reinforcement

Figure R12.12 illustrates two methods of satisfying requirements for anchorage of tension reinforcement beyond the face of support. For anchorage of reinforcement with hooks, see R12.5.

Section 12.12.3 provides for possible shifting of the moment diagram at a point of inflection, as discussed under R12.10.3. This requirement may exceed that of 12.10.3, and the more restrictive of the two provisions governs.



(a) Anchorage into exterior column



Note: Usually such anchorage becomes part of the adjacent beam reinforcement.

(b) Anchorage into adjacent beam

Fig. R12.12—Development of negative moment reinforcement.

CODE

12.13 — Development of web reinforcement

12.13.1 — Web reinforcement shall be as close to the compression and tension surfaces of the member as cover requirements and proximity of other reinforcement permits.

12.13.2 — Ends of single leg, simple U-, or multiple U-stirrups shall be anchored as required by 12.13.2.1 through 12.13.2.5:

12.13.2.1 — For No. 5 bar and D31 wire and smaller, and for No. 6, No. 7, and No. 8 bars with f_y of 40,000 psi or less, a standard hook around longitudinal reinforcement.

12.13.2.2 — For No. 6, No. 7, and No. 8 stirrups with f_y greater than 40,000 psi, a standard stirrup hook around a longitudinal bar plus an embedment between midheight of the member and the outside end of the hook equal to or greater than $0.014 d_b f_y / \sqrt{f'_c}$.

12.13.2.3 — For each leg of welded plain wire fabric forming simple U-stirrups, either:

(a) Two longitudinal wires spaced at a 2 in. spacing along the member at the top of the U; or

(b) One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 2 in. from the first wire. The second wire shall be permitted to be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than $8d_b$.

COMMENTARY

R12.13 — Development of web reinforcement

R12.13.1 — Stirrups should be carried as close to the compression face of the member as possible because near ultimate load, the flexural tension cracks penetrate deeply.

R12.13.2 — The anchorage or development requirements for stirrups composed of bars or deformed wire were changed in the 1989 code to simplify the requirements. The straight anchorage was deleted as this stirrup is difficult to hold in place during concrete placement and the lack of a hook may make the stirrup ineffective as it crosses shear cracks near the end of the stirrup.

R12.13.2.1 — For a No. 5 bar or smaller, anchorage is provided by a standard stirrup hook, as defined in 7.1.3, hooked around a longitudinal bar. The 1989 code eliminated the need for a calculated straight embedment length in addition to the hook for these small bars, but 12.13.1 requires a full depth stirrup. Likewise, larger stirrups with f_y equal to or less than 40,000 are sufficiently anchored with a standard stirrup hook around the longitudinal reinforcement.

R12.13.2.2 — Because it is not possible to bend a No. 6, No. 7, or No. 8 stirrup tightly around a longitudinal bar and due to the force in a bar with a design stress greater than 40,000 psi, stirrup anchorage depends on both the value of the hook and whatever development length is provided. A longitudinal bar within a stirrup hook limits the width of any flexural cracks, even in a tensile zone. Because such a stirrup hook cannot fail by splitting parallel to the plane of the hooked bar, the hook strength as utilized in 12.5.2 has been adjusted to reflect cover and confinement around the stirrup hook.

For stirrups with f_y of only 40,000 psi, a standard stirrup hook provides sufficient anchorage and these bars are covered in 12.13.2.1. For bars with higher strength, the embedment should be checked. A 135- or 180-deg hook is preferred, but a 90 deg hook may be used provided the free end of the 90-deg hook is extended the full 12 bar diameters as required in 7.1.3.

R12.13.2.3 — The requirements for anchorage of welded plain wire fabric stirrups are illustrated in Fig. R12.13.2.3.

CODE

12.13.2.4 — For each end of a single leg stirrup of welded plain or deformed wire fabric, two longitudinal wires at a minimum spacing of 2 in. and with the inner wire at least the greater of ***d*/4** or 2 in. from ***d*/2**. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

COMMENTARY

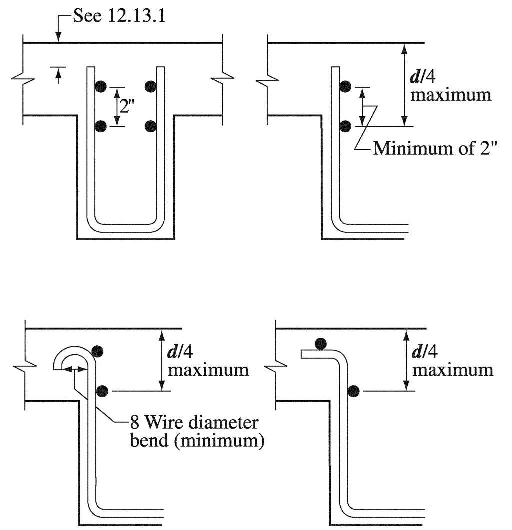


Fig. R12.13.2.3—Anchorage in compression zone of welded plain wire fabric U-stirrups.

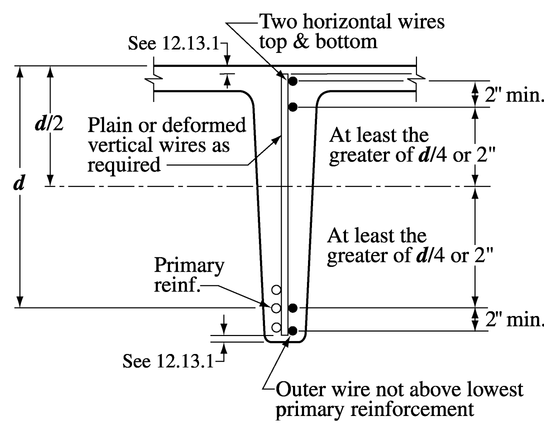


Fig. R12.13.2.4—Anchorage of single leg welded wire fabric shear reinforcement.

R12.13.2.4 — Use of welded wire fabric for shear reinforcement has become commonplace in the precast, prestressed concrete industry. The rationale for acceptance of straight sheets of wire fabric as shear reinforcement is presented in a report by a joint PCI/WRI Ad Hoc Committee on Welded Wire Fabric for Shear Reinforcement.[12.24]

The provisions for anchorage of single leg welded wire fabric in the tension face emphasize the location of the longitudinal wire at the same depth as the primary flexural reinforcement to avoid a splitting problem at the tension steel level. Figure R12.13.2.4 illustrates the anchorage requirements for single leg welded wire fabric. For anchorage of single leg welded wire fabric, the code has permitted hooks and embedment length in the compression and tension faces of members (see 12.13.2.1 and 12.13.2.3), and embedment only in the compression face (see 12.13.2.2). Section 12.13.2.4 provides for anchorage of straight single leg welded wire fabric using longitudinal wire anchorage with adequate embedment length in compression and tension faces of members.

CODE

12.13.2.5 — In joist construction as defined in 8.11, for No. 4 bar and D20 wire and smaller, a standard hook.

12.13.3 — Between anchored ends, each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar.

12.13.4 — Longitudinal bars bent to act as shear reinforcement, if extended into a region of tension, shall be continuous with longitudinal reinforcement and, if extended into a region of compression, shall be anchored beyond mid-depth $d/2$ as specified for development length in 12.2 for that part of f_y required to satisfy Eq. (11-17).

12.13.5 — Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when length of laps are $1.3\ell_d$. In members at least 18 in. deep, such splices with $A_b f_y$ not more than 9000 lb per leg shall be considered adequate if stirrup legs extend the full available depth of member.

12.14 — Splices of reinforcement—General

12.14.1 — Splices of reinforcement shall be made only as required or permitted on design drawings, or in specifications, or as authorized by the engineer.

12.14.2 — Lap splices

12.14.2.1 — Lap splices shall not be used for bars larger than No. 11 except as provided in 12.16.2 and 15.8.2.3.

12.14.2.2 — Lap splices of bars in a bundle shall be based on the lap splice length required for individual bars within the bundle, increased in accordance with 12.4. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced.

12.14.2.3 — Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than one-fifth the required lap splice length, nor 6 in.

COMMENTARY

R12.13.2.5 — In joists, a small bar or wire can be anchored by a standard hook not engaging longitudinal reinforcement, allowing a continuously bent bar to form a series of single-leg stirrups in the joist.

R12.13.5 — These requirements for lapping of double U-stirrups to form closed stirrups control over the provisions of 12.15.

R12.14 — Splices of reinforcement—General

Splices should, if possible, be located away from points of maximum tensile stress. The lap splice requirements of 12.15 encourage this practice.

R12.14.2 — Lap splices

R12.14.2.1 — Because of lack of adequate experimental data on lap splices of No. 14 and No. 18 bars in compression and in tension, lap splicing of these bar sizes is prohibited except as permitted in 12.16.2 and 15.8.2.3 for compression lap splices of No. 14 and No. 18 bars with smaller bars.

R12.14.2.2 — The increased length of lap required for bars in bundles is based on the reduction in the exposed perimeter of the bars. Only individual bars are lap spliced along the bundle.

R12.14.2.3 — If individual bars in noncontact lap splices are too widely spaced, an unreinforced section is created. Forcing a potential crack to follow a zigzag line (5-to-1 slope) is considered a minimum precaution. The 6 in. maximum spacing is added because most research available on the lap splicing of deformed bars was conducted with reinforcement within this spacing.

CODE

12.14.3 — Mechanical and welded splices

12.14.3.1 — Mechanical and welded splices shall be permitted.

12.14.3.2 — A full mechanical splice shall develop in tension or compression, as required, at least 125 percent of specified yield strength f_y of the bar.

12.14.3.3 — Except as provided in this code, all welding shall conform to "Structural Welding Code—Reinforcing Steel" (ANSI/AWS D1.4).

12.14.3.4 — A full penetration welded splice shall develop at least 125 percent of the specified yield strength f_y of the bar.

12.14.3.5 — Mechanical or welded splices not meeting requirements of 12.14.3.2 or 12.14.3.4 shall be permitted only for No. 5 bars and smaller and in accordance with 12.15.4.

12.15 — Splices of deformed bars and deformed wire in tension

12.15.1 — Minimum length of lap for tension lap splices shall be as required for Class A or B splice, but not less than 12 in., where:

Class A splice .... $1.0\ell_d$

Class B splice .... $1.3\ell_d$

where ℓ_d is the tensile development length for the specified yield strength f_y in accordance with 12.2 without the modification factor of 12.2.5.

COMMENTARY

R12.14.3 — Mechanical and welded splices

R12.14.3.2 — The maximum reinforcement stress used in design under the code is the specified yield strength. To ensure sufficient strength in splices so that yielding can be achieved in a member and thus brittle failure avoided, the 25 percent increase above the specified yield strength was selected as both an adequate minimum for safety and a practicable maximum for economy.

R12.14.3.3 — See R3.5.2 for discussion on welding.

R12.14.3.4 — A full welded splice is primarily intended for large bars (No. 6 and larger) in main members. The tensile strength requirement of 125 percent of specified yield strength is intended to provide sound welding that is also adequate for compression. See the discussion on strength in R12.14.3.2. The 318-95 code eliminated a requirement that the bars be butted because indirect butt welds are permitted by ANSI/AWS D1.4, although ANSI/AWS D1.4 does indicate that wherever practical, direct butt splices are preferable for No. 7 bars and larger.

R12.14.3.5 — The use of mechanical or welded splices of less strength than 125 percent of specified yield strength is permitted if the minimum design criteria of 12.15.4 are met. Therefore, lap welds of reinforcing bars, either with or without backup material, welds to plate connections, and end-bearing splices are allowed under certain conditions. The 318-95 code limited these lower strength welds and connections to No. 5 bars and smaller due to the potentially brittle nature of failure at these welds.

R12.15 — Splices of deformed bars and deformed wire in tension

R12.15.1 — Lap splices in tension are classified as Type A or B, with length of lap a multiple of the tensile development length ℓ_d. The development length ℓ_d used to obtain lap length should be based on f_y because the splice classifications already reflect any excess reinforcement at the splice location; therefore, the factor from 12.2.5 for excess A_s should not be used. When multiple bars located in the same plane are spliced at the same section, the clear spacing is the minimum clear distance between the adjacent splices. For splices in columns with offset bars, Fig. R12.15.1(a) illustrates the clear spacing to be used. For staggered splices, the clear spacing is the minimum distance between adjacent splices (distance x in Fig. R12.15.1(b)).

The 318-89 code contained several changes in development length in tension that eliminated many of the concerns regarding tension splices due to closely spaced bars with minimal cover. Thus, the Class C splice was eliminated

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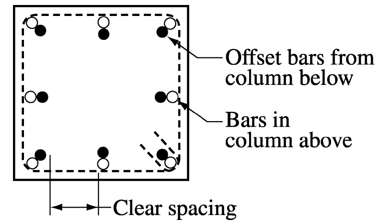
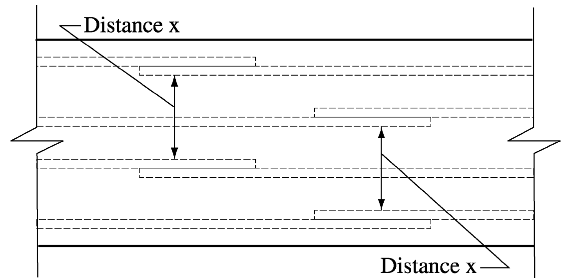


(a) Offset column bars



(b) Staggered splices

Fig. R12.15.1—Clear spacing of spliced bars.

TABLE R12.15.2—TENSION LAP SPLICES

$\frac{A_s \text{ provided}^*}{A_s \text{ required}}$	Maximum percent of A_s spliced within required lap length	
	50	100
Equal to or greater than 2	Class A	Class B
Less than 2	Class B	Class B

* Ratio of area of reinforcement provided to area of reinforcement required by analysis at splice locations.

although development lengths, on which splice lengths are based, have in some cases increased. Committee 318 considered suggestions from many sources, including ACI Committee 408, but has retained a two-level splice length primarily to encourage designers to splice bars at points of minimum stress and to stagger splices to improve behavior of critical details.

12.15.2 — Lap splices of deformed bars and deformed wire in tension shall be Class B splices except that Class A splices are allowed when:

(a) The area of reinforcement provided is at least twice that required by analysis, over the entire length of the splice; and

(b) One-half or less of the entire reinforcement is spliced within the required lap length.

Also, in nonprestressed circular tanks designed for hoop tension, splices in hoop reinforcement shall be Class B and the location of splices shall be staggered. Adjacent hoop reinforcement splices shall be staggered horizontally (center of lap below to center of lap above) by not less than one lap length nor 3 ft, and shall not coincide in vertical arrays more frequently than every third bar.

R12.15.2 — The tension lap splice requirements of 12.15.1 encourage the location of splices away from regions of high tensile stress, to locations where the area of steel provided is at least twice that required by analysis. Table R12.15.2 presents the splice requirements in tabular form as presented in earlier ACI 318 code editions.

While the walls of nonprestressed circular tanks are placed in direct tension by the liquid contents, they should not be considered as "tension tie members" for which full welded or mechanical splices are required by 12.15.5. The large number of splices makes any one splice less critical. Staggering of splices in the hoop reinforcement makes progressive failure of adjacent splices unlikely. Thus, Class B lap splices can be used, and full welded or mechanical splices are not needed.

CODE

12.15.3 — Mechanical or welded splices used where area of reinforcement provided is less than twice that required by analysis shall meet requirements of 12.14.3.2 or 12.14.3.4.

12.15.4 — Mechanical or welded splices not meeting the requirements of 12.14.3.2 or 12.14.3.4 shall be permitted for No. 5 bars and smaller if the requirements of 12.15.4.1 through 12.15.4.3 are met:

12.15.4.1 — Splices shall be staggered at least 24 in.

12.15.4.2 — In computing the tensile forces that can be developed at each section, the spliced reinforcement stress shall be taken as the specified splice strength, but not greater than f_y. The stress in the unspliced reinforcement shall be taken as f_y times the ratio of the shortest length embedded beyond the section to ℓ_d, but not greater than f_y.

12.15.4.3 — The total tensile force that can be developed at each section must be at least twice that required by analysis, and at least 20,000 psi times the total area of reinforcement provided.

12.15.5 — Splices in tension tie members shall be made with a full mechanical or full welded splice in accordance with 12.14.3.2 or 12.14.3.4 and splices in adjacent bars shall be staggered at least 30 in.

12.16 — Splices of deformed bars in compression

COMMENTARY

R12.15.3 — A mechanical or welded splice should develop at least 125 percent of the specified yield strength when located in regions of high tensile stress in the reinforcement. Such splices need not be staggered, although such staggering is encouraged where the area of reinforcement provided is less than twice that required by the analysis.

R12.15.4 — See R12.14.3.5. Section 12.15.4 concerns the situation where mechanical or welded splices of strength less than 125 percent of the specified yield strength of the reinforcement may be used. It provides a relaxation in the splice requirements where the splices are staggered and excess reinforcement area is available. The criterion of twice the computed tensile force is used to cover sections containing partial tensile splices with various percentages of total continuous steel. The usual partial tensile splice is a flare groove weld between bars or bar and structural steel piece.

To detail such welding, the length of weld should be specified. Such welds are rated at the product of total weld length times effective size of groove weld (established by bar size) times allowable stress permitted by **"Structural Welding Code—Reinforcing Steel"** (ANSI/AWS D1.4).

A full mechanical or welded splice conforming to 12.14.3.2 or 12.14.3.4 can be used without the stagger requirement in lieu of the lower strength mechanical or welded splice.

R12.15.5 — A tension tie member has the following characteristics: member having an axial tensile force sufficient to create tension over the cross section; a level of stress in the reinforcement such that every bar must be fully effective; and limited concrete cover on all sides. Examples of members that may be classified as tension ties are arch ties, hangers carrying load to an overhead supporting structure, and main tension elements in a truss.

In determining if a member should be classified as a tension tie, consideration should be given to the importance, function, proportions, and stress conditions of the member related to the above characteristics. For example, a usual large circular tank, with many bars and with splices well staggered and widely spaced, should not be classified as a tension tie member, and Class B splices may be used.

R12.16 — Splices of deformed bars in compression

Bond research has been primarily related to bars in tension. Bond behavior of compression bars is not complicated by the problem of transverse tension cracking and thus, compression splices do not require provisions as strict as those specified for tension splices. The minimum lengths for column splices contained originally in the 1956 ACI Building Code have been carried forward in later ACI 318 codes, and extended to compression bars in beams and to higher strength steels. No changes have been made in the provisions for compression splices since the 1971 code.

CODE

12.16.1 — Compression lap splice length shall be **$0.0005f_yd_b$**, for f_y of 60,000 psi or less, or **$(0.0009f_y - 24)d_b$** for f_y greater than 60,000 psi, but not less than 12 in. For f_c' less than 3000 psi, length of lap shall be increased by one-third.

12.16.2 — When bars of different size are lap spliced in compression, splice length shall be the larger of either development length of larger bar, or splice length of smaller bar. Lap splices of No. 14 and No. 18 bars to No. 11 and smaller bars shall be permitted.

12.16.3 — Mechanical or welded splices used in compression shall meet requirements of 12.14.3.2 or 12.14.3.4.

12.16.4 — End-bearing splices

12.16.4.1 — In bars required for compression only, transmission of compressive stress by bearing of square cut ends held in concentric contact by a suitable device shall be permitted.

12.16.4.2 — Bar ends shall terminate in flat surfaces within 1.5 deg of a right angle to the axis of the bars and shall be fitted within 3 deg of full bearing after assembly.

12.16.4.3 — End-bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

12.17 — Special splice requirements for columns

COMMENTARY

R12.16.1 — Essentially, lap requirements for compression splices have remained the same since the 1963 ACI Building Code.

The 1963 ACI Building Code values were modified in the 1971 code to recognize various degrees of confinement and to permit design with reinforcement up to 80,000 psi yield strength. Tests[12.1,12.25] have shown that splice strengths in compression depend considerably on end bearing, and do not increase proportionally in strength when the splice length is doubled. Accordingly, for yield strengths above 60,000 psi, compression lap lengths are significantly increased, except where spiral enclosures are used (as in spiral columns) the increase is about 10 percent at 75,000 psi.

R12.16.2 — The lap splice length is to be computed based on the larger of the compression splice length of the smaller bar or the compression development length of the larger bar. Lap splices are generally prohibited for No. 14 or No. 18 bars; however, for compression only, lap splices are permitted for No. 14 or No. 18 bars to No. 11 or smaller bars.

R12.16.4 — End-bearing splices

R12.16.4.1 — Experience with end-bearing splices has been almost exclusively with vertical bars in columns. If bars are significantly inclined from the vertical, special attention is required to ensure that adequate end-bearing contact can be achieved and maintained.

R12.16.4.2 — These tolerances were added in the 1971 code, representing practice based on tests of full-size members containing No. 18 bars.

R12.16.4.3 — This limitation was added in the 1971 code to ensure a minimum shear resistance in sections containing end-bearing splices.

R12.17 — Special splice requirements for columns

In columns subject to flexure and axial loads, tension stresses may occur on one face of the column for moderate and large eccentricities as shown in Fig. R12.17. When such tensions occur, 12.17 requires tension splices to be used or an adequate tensile resistance to be provided. Furthermore, a minimum tension capacity is required in each face of all columns even where analysis indicates compression only.

The 318-89 code clarifies this section on the basis that a compressive lap splice has a tension capacity of at least one-quarter f_y, which simplifies the calculation requirements in previous codes.

CODE

COMMENTARY

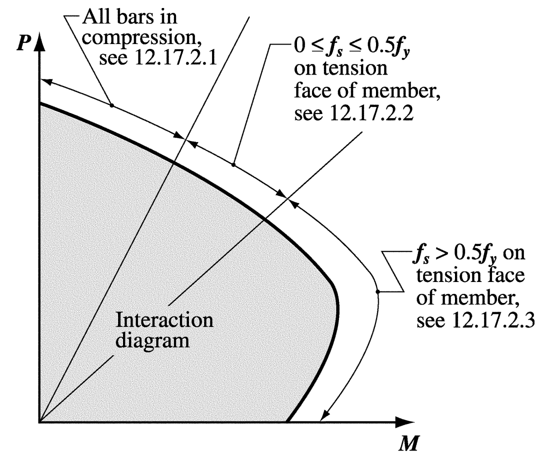


Fig. R12.17—Special splice requirements for columns.

Note that the column splice should satisfy requirements for all load combinations for the column. Frequently, the basic gravity load combination will govern the design of the column itself, but a load combination including wind or seismic loads may induce greater tension in some column bars, and the column splice should be designed for this tension.

12.17.1 — Lap splices, mechanical splices, butt-welded splices, and end-bearing splices shall be used with the limitations of 12.17.2 through 12.17.4. A splice shall satisfy requirements for all load combinations for the column.

12.17.2 — Lap splices in columns

12.17.2.1 — Where the bar stress due to factored loads is compressive, lap splices shall conform to 12.16.1, 12.16.2, and, where applicable, to 12.17.2.4 or 12.17.2.5.

12.17.2.2 — Where the bar stress due to factored loads is tensile and does not exceed $0.5f_y$ in tension, lap splices shall be Class B tension lap splices if more than one-half of the bars are spliced at any section, or Class A tension lap splices if half or fewer of the bars are spliced at any section and alternate lap splices are staggered by ℓ_d.

12.17.2.3 — Where the bar stress due to factored loads is greater than $0.5f_y$ in tension, lap splices shall be Class B tension lap splices.

12.17.2.4 — In tied reinforced compression members, where ties throughout the lap splice length have an effective area not less than $0.0015hs$, lap splice length shall be permitted to be multiplied by 0.83, but lap length shall not be less than 12 in. Tie legs perpendicular to dimension h shall be used in determining effective area.

R12.17.2 — Lap splices in columns

R12.17.2.1 — The 1989 code was simplified for column bars always in compression on the basis that a compressive lap splice is adequate for sufficient tension to preclude special requirements.

R12.17.2.4 — Reduced lap lengths are allowed when the splice is enclosed throughout its length by minimum ties.

The tie legs perpendicular to each direction are computed separately, and the requirement must be satisfied in each direction. This is illustrated in Fig. R12.17.2, where four legs are effective in one direction, and two legs in the other direction. This calculation is critical in one direction that normally can be determined by inspection.

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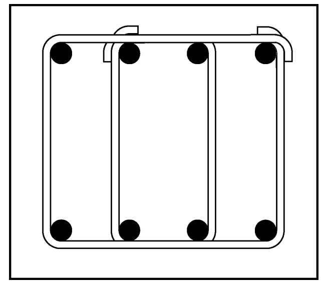

Fig. R12.17.2—The legs that cross the axis of bending are used to compute effective area. In the case shown, four legs are effective.

12.17.2.5 — In spirally reinforced compression members, lap splice length of bars within a spiral shall be permitted to be multiplied by 0.75, but lap length shall not be less than 12 in.

R12.17.2.5 — Compression lap lengths may be reduced when the lap splice is enclosed throughout its length by spirals because of increased splitting resistance. Spirals should meet requirements of 7.10.4 and 10.9.3.

12.17.3 — Mechanical or welded splices in columns

Mechanical or welded splices in columns shall meet the requirements of 12.14.3.2 or 12.14.3.4.

R12.17.3 — Mechanical or welded splices in columns

Mechanical or welded splices are allowed for splices in columns, but should be designed as a full mechanical splice or a full welded splice developing 125 percent f_y as required by 12.14.3.2 or 12.14.3.4. Splice capacity is traditionally tested in tension, and full strength is required to reflect the high compression loads possible in column reinforcement due to creep effects. If a mechanical splice developing less than a full mechanical splice is used, then the splice is required to conform to all requirements of end-bearing splices of 12.16.4 and 12.17.4.

12.17.4 — End-bearing splices in columns

End-bearing splices complying with 12.16.4 shall be permitted to be used for column bars stressed in compression provided the splices are staggered or additional bars are provided at splice locations. The continuing bars in each face of the column shall have a tensile strength, based on the specified yield strength f_y, not less than $0.25f_y$ times the area of the vertical reinforcement in that face.

R12.17.4 — End-bearing splices in columns

End-bearing splices used to splice column bars always in compression should have a tension capacity of 25 percent of the yield strength of the steel area on each face of the column, either by staggering the end-bearing splices or by adding additional steel through the splice location. The end-bearing splice should conform to 12.16.4.

12.18 — Splices of welded deformed wire fabric in tension

12.18.1 — Minimum length of lap for lap splices of welded deformed wire fabric measured between the ends of each fabric sheet shall be not less than $1.3\ell_d$ nor 8 in., and the overlap measured between outermost cross wires of each fabric sheet shall be not less than 2 in. ℓ_d shall be the development length for the specified yield strength f_y in accordance with 12.7.

12.18.2 — Lap splices of welded deformed wire fabric, with no cross wires within the lap splice length, shall be determined as for deformed wire.

R12.18 — Splices of welded deformed wire fabric in tension

Splice provisions for deformed fabric are based on available tests.[12.26] The requirements were simplified (1976 code supplement) from provisions of the 1971 code by assuming that only one cross wire of each fabric sheet is overlapped and by computing the splice length as $1.3\ell_d$. The development length ℓ_d is that computed in accordance with the provisions of 12.7 without regard to the 8 in. minimum. The 8 in. applies to the overall splice length. See Fig. R12.18. If no cross wires are within the lap length, the provisions for deformed wire apply.

CODE

12.18.3 — When any plain wires are present in the deformed wire fabric in the direction of the lap splice or when deformed wire fabric is lap spliced to plain wire fabric, the fabric shall be lap spliced in accordance with 12.19.

COMMENTARY

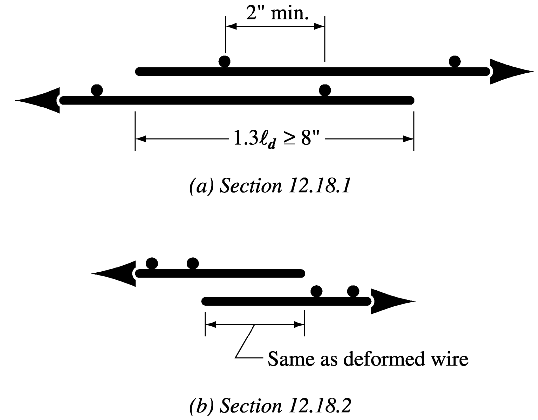

Fig. R12.18—Lap splices of deformed fabric.

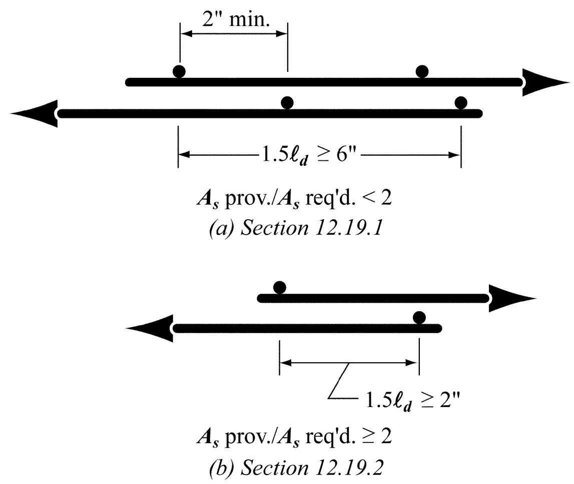

Fig. R12.19—Lap splices of plain fabric.

12.19 — Splices of welded plain wire fabric in tension

Minimum length of lap for lap splices of welded plain wire fabric shall be in accordance with 12.19.1 and 12.19.2.

12.19.1 — When area of reinforcement provided is less than twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each fabric sheet shall be not less than one spacing of cross wires plus 2 in., nor less than $1.5\ell_d$, nor 6 in. The development length ℓ_d for the specified yield strength f_y shall be in accordance with 12.8.

12.19.2 — When area of reinforcement provided is at least twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each fabric sheet shall not be less than $1.5\ell_d$, nor 2 in. ℓ_d shall be the development length for the specified yield strength f_y in accordance with 12.8.

R12.19 — Splices of welded plain wire fabric in tension

The strength of lap splices of welded plain wire fabric is dependent primarily on the anchorage obtained from the cross wires rather than on the length of wire in the splice. For this reason, the lap is specified in terms of overlap of cross wires rather than in wire diameters or inches. The 2 in. additional lap required is to assure overlapping of the cross wires and to provide space for satisfactory consolidation of the concrete between cross wires. Research[12.27] has shown an increased splice length is required when fabric of large, closely spaced wires, is lapped and as a consequence additional splice length requirements are provided for these fabrics, in addition to an absolute minimum of 6 in. The development length ℓ_d is that computed in accordance with the provisions of 12.8 without regard to the 6 in. minimum. Splice requirements are illustrated in Fig. R12.19.

CODE

COMMENTARY

Notes

PART 5 — STRUCTURAL SYSTEMS OR ELEMENTS

CHAPTER 13 — TWO-WAY SLAB SYSTEMS

CODE

13.0 — Notation

b_1 = width of the critical section defined in 11.12.1.2 measured in the direction of the span for which moments are determined, in.

b_2 = width of the critical section defined in 11.12.1.2 measured in the direction perpendicular to b_1, in.

c_1 = size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, in.

c_2 = size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which moments are being determined, in.

C = cross-sectional constant to define torsional properties

$$= \sum\left(1 - 0.63\frac{x}{y}\right)\frac{x^3y}{3}$$

The constant C for T- or L-sections shall be permitted to be evaluated by dividing the section into separate rectangular parts and summing the values of C for each part

E_{cb} = modulus of elasticity of beam concrete, psi

E_{cs} = modulus of elasticity of slab concrete, psi

h = overall thickness of member, in.

I_b = moment of inertia about centroidal axis of gross section of beam as defined in 13.2.4, in.^{4}

I_s = moment of inertia about centroidal axis of gross section of slab, in.^{4}

= $h^3/12$ times width of slab defined in notations α and β_t

K_t = torsional stiffness of torsional member; moment per unit rotation. See R13.7.5.

ℓ_n = length of clear span in direction that moments are being determined, measured face-to-face of supports, in.

ℓ_1 = length of span in direction that moments are being determined, measured center-to-center of supports, in.

ℓ_2 = length of span transverse to ℓ_1, measured center-to-center of supports, in. See also 13.6.2.3 and 13.6.2.4

COMMENTARY

The design methods given in Chapter 13 are based on analysis of the results of an extensive series of tests[13.1-13.7] and the well established performance record of various slab systems. Much of Chapter 13 is concerned with the selection and distribution of flexural reinforcement. The designer is cautioned that the problem related to safety of a slab system is the transmission of load from the slab to the columns by flexure, torsion, and shear. Design criteria for shear and torsion in slabs are given in Chapter 11.

Design aids for use in the engineering analysis and design of two-way slab systems are given in the *ACI Design Handbook*.[13.8] Design aids are provided to simplify application of the direct design and equivalent frame methods of Chapter 13.

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip. However, the values of constants used in equations are dependent on the units of the parameters. If the user chooses to use alternate units, the constants must be adjusted accordingly.

The requirements of ACI 318, Chapter 13, are generally applicable to the design of environmental engineering concrete structures. Deviations from ACI 318 primarily relate to modifying requirements that are explicitly or implicitly due to pattern loading effects from live loads derived from fluid pressures.

CODE

M_o = total factored static moment, in.-lb

M_u = factored moment at section, in.-lb

V_c = nominal shear strength provided by concrete, lb. See 11.12.2.1

V_u = factored shear force at section, lb

w_d = factored dead load per unit area

w_ℓ = factored live load per unit area

w_u = factored load per unit area

x = shorter overall dimension of rectangular part of cross section, in.

y = longer overall dimension of rectangular part of cross section, in.

α = ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of the beam

= $\dfrac{E_{cb}I_b}{E_{cs}I_s}$

α_1 = α in direction of ℓ_1

α_2 = α in direction of ℓ_2

β_t = ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of beam, center-to-center of supports

= $\dfrac{E_{cb}C}{2E_{cs}I_s}$

γ_f = fraction of unbalanced moment transferred by flexure at slab-column connections. See 13.5.3.2

γ_v = fraction of unbalanced moment transferred by eccentricity of shear at slab-column connections

= $1 - \gamma_f$

ρ = ratio of nonprestressed tension reinforcement

ρ_b = reinforcement ratio producing balanced strain conditions

ϕ = strength reduction factor

13.1 — Scope

13.1.1 — Provisions of Chapter 13 shall apply for design of slab systems and straight tank walls reinforced for flexure in more than one direction, with or without beams between supports.

13.1.2 — For a slab system supported by columns or walls, the dimensions c_1 and c_2 and the clear span ℓ_n shall be based on an effective support area defined by the intersection of the bottom surface of the slab, or of the drop panel if there is one, with the largest right circular cone, right pyramid, or tapered wedge whose surfaces are located within the column and the capital or bracket and are oriented no greater than 45 deg to the axis of the column.

COMMENTARY

R13.1 — Scope

The fundamental design principles contained in Chapter 13 are applicable to all planar structural systems subjected to transverse loads. Some of the specific design rules, as well as historical precedents, limit the types of structures to which Chapter 13 applies. General characteristics of slab systems that may be designed according to Chapter 13 are described in this section. These systems include “flat slabs,” “flat plates,” “two-way slabs,” and “waffle slabs.” Slabs with paneled ceilings are two-way wide-band beam systems.

True “one-way slabs,” slabs reinforced to resist flexural stresses in only one direction, are excluded. Also excluded are soil supported slabs, such as “slabs-on-grade,” that do not transmit vertical loads from other parts of the structure to the soil.

CODE

13.1.3 — Solid slabs and slabs with recesses or pockets made by permanent or removable fillers between ribs or joists in two directions are included within the scope of Chapter 13.

13.1.4 — Minimum thickness of slabs designed in accordance with Chapter 13 shall be as required by 9.5.3.

COMMENTARY

For slabs with beams, the explicit design procedures of Chapter 13 apply only when the beams are located at the edges of the panel and when the beams are supported by columns or other essentially nondeflecting supports at the corners of the panel. Two-way slabs with beams in one direction with both slab and beams supported by girders in the other direction may be designed under the general requirements of Chapter 13. Such designs should be based upon analysis compatible with the deflected position of the supporting beams and girders.

For slabs supported on walls, the explicit design procedures in this chapter treat the wall as a beam of infinite stiffness; therefore, each wall should support the entire length of an edge of the panel. (See 13.2.3). Wall-like columns less than a full panel length can be treated as columns.

13.2 — Definitions

13.2.1 — Column strip is a design strip with a width on each side of a column centerline equal to $\mathbf{0.25\ell_2}$ or $\mathbf{0.25\ell_1}$, whichever is less. Column strip includes beams, if any.

13.2.2 — Middle strip is a design strip bounded by two column strips.

13.2.3 — A panel is bounded by column, beam, or wall centerlines on all sides.

13.2.4 — For monolithic or fully composite construction, a beam includes that portion of slab on each side of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness.

R13.2 — Definitions

R13.2.3 — A panel includes all flexural elements between column centerlines. Thus, the column strip includes the beam, if any.

R13.2.4 — For monolithic or fully composite construction, the beams include portions of the slab as flanges. Two examples of the rule in this section are provided in Fig. R13.2.4.

13.3 — Slab reinforcement

13.3.1 — Area of reinforcement in each direction for two-way slab systems shall be determined from moments at critical sections, but shall not be less than required by 7.12.

R13.3 — Slab reinforcement

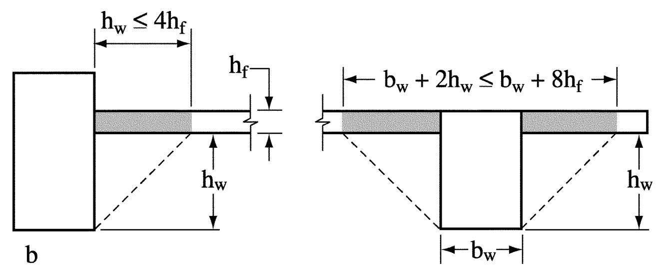

Fig. R13.2.4—Examples of portion of slab to be included with beam under 13.2.4.

CODE

13.3.2 — Spacing of reinforcement shall not exceed two times the slab thickness, nor 12 in., except for portions of slab area of cellular or ribbed construction. In the slab over cellular spaces, reinforcement shall be provided as required by 7.12.

13.3.3 — Positive moment reinforcement perpendicular to a discontinuous edge shall extend to the edge of slab and have embedment, straight or hooked, at least 6 in. in spandrel beams, columns, or walls.

13.3.4 — Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored, in spandrel beams, columns, or walls, to be developed at face of support according to provisions of Chapter 12.

13.3.5 — Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, anchorage of reinforcement shall be permitted within the slab.

13.3.6 — In slabs with beams between supports with a value of α greater than 1.0, special top and bottom slab reinforcement shall be provided at exterior corners in accordance with 13.3.6.1 through 13.3.6.4.

13.3.6.1 — The special reinforcement in both top and bottom of slab shall be sufficient to resist a moment per foot of width equal to the maximum positive moment in the slab.

13.3.6.2 — The moment shall be assumed to be about an axis perpendicular to the diagonal from the corner in the top of the slab and about an axis parallel to the diagonal from the corner in the bottom of the slab.

13.3.6.3 — The special reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth the longer span.

13.3.6.4 — The special reinforcement shall be placed in a band parallel to the diagonal in the top of the slab and a band perpendicular to the diagonal in the bottom of the slab. Alternatively, the special reinforcement shall be placed in two layers parallel to the sides of the slab in both the top and bottom of the slab.

13.3.7 — Where a drop panel is used to reduce amount of negative moment reinforcement over the column of a flat slab, size of drop panel shall be in accordance with 13.3.7.1, 13.3.7.2, and 13.3.7.3.

13.3.7.1 — Drop panel shall extend in each direction from centerline of support a distance not less than

COMMENTARY

R13.3.2 — The requirement that the center-to-center spacing of the reinforcement be not more than two times the slab thickness applies only to the reinforcement in solid slabs, and not to reinforcement joists or waffle slabs. This limitation is to ensure slab action, cracking, and provide for the possibility of loads concentrated on small areas of the slab. See also R10.6.

A 12 in. maximum spacing of reinforcement is required in liquid-retaining structures to ensure adequate crack control.

R13.3.3 to R13.3.5 — Bending moments in slabs at spandrel beams can be subject to great variation. If spandrel beams are built solidly into walls, the slab approaches complete fixity. Without an integral wall, the slab could approach simply supported, depending on the torsional rigidity of the spandrel beam or slab edge. These requirements provide for unknown conditions that might normally occur in a structure.

CODE

one-sixth the span length measured from center-to-center of supports in that direction.

13.3.7.2 — Projection of drop panel below the slab shall be at least one-quarter the slab thickness beyond the drop.

13.3.7.3 — In computing required slab reinforcement, the thickness of the drop panel below the slab shall not be assumed greater than one-quarter the distance from edge of drop panel to edge of column or column capital.

13.3.8 — Details of reinforcement in slabs without beams

13.3.8.1 — In addition to the other requirements of 13.3, reinforcement in slabs without beams shall have minimum extensions as prescribed in Fig. 13.3.8.

13.3.8.2 — Where adjacent spans are unequal, extensions of negative moment reinforcement beyond the face of support as prescribed in Fig. 13.3.8 shall be based on requirements of the longer span.

13.3.8.3 — Bent bars shall be permitted only when depth-span ratio permits use of bends 45 deg or less.

13.3.8.4 — In frames where two-way slabs act as primary members resisting lateral loads, lengths of reinforcement shall be determined by analysis but shall not be less than those prescribed in Fig. 13.3.8.

13.3.8.5 — All bottom bars or wires within the column strip, in each direction, shall be continuous or spliced with Class A tension splices or with mechanical or welded splices satisfying 12.14.3. Splices shall be located as shown in Fig. 13.3.8. At least two of the column strip bottom bars or wires in each direction shall pass within the column core and shall be anchored at exterior supports.

13.3.8.6 — In slabs with shearheads and in lift-slab construction where it is not practical to pass the bottom bars required by 13.3.8.5 through the column, at least two bonded bottom bars or wires in each direction shall pass through the shearhead or lifting collar as close to the column as practicable and be continuous or spliced with a Class A splice. At exterior columns, the reinforcement shall be anchored at the shearhead or lifting collar.

COMMENTARY

R13.3.8 — Details of reinforcement in slabs without beams

In the ACI 318-89 code, bent bars were removed from Fig. 13.3.8. This was done because bent bars are seldom used and are difficult to place properly. Bent bars are permitted, however, if they comply with 13.3.8.3. Refer to 13.4.8 of the 318-83 code.

R13.3.8.4 — For moments resulting from combined lateral and gravity loadings, the minimum lengths and extensions of bars in Fig. 13.3.8 may not be sufficient.

R13.3.8.5 — The continuous column strip bottom reinforcement provides the slab some residual ability to span to the adjacent supports should a single support be damaged. The two continuous column strip bottom bars or wires through the column may be termed "integrity steel," and are provided to give the slab some residual capacity following a single punching shear failure at a single support.[13.9] In the ACI 318-02 code, mechanical and welded splices were explicitly recognized as alternative methods of splicing reinforcement.

R13.3.8.6 — In the ACI 318-92 code, this provision was added to require the same "integrity steel" as for other two-way slabs without beams in case of a punching shear failure at a support.

In some instances, there is sufficient clearance so that the bonded bottom bars can pass under shearheads and through the column. Where clearance under the shearhead is inadequate, the bottom bars must pass through holes in the shearhead arms or within the perimeter of the lifting collar. Shearheads should be kept as low as possible in the slab to increase their effectiveness.

CODE **COMMENTARY**

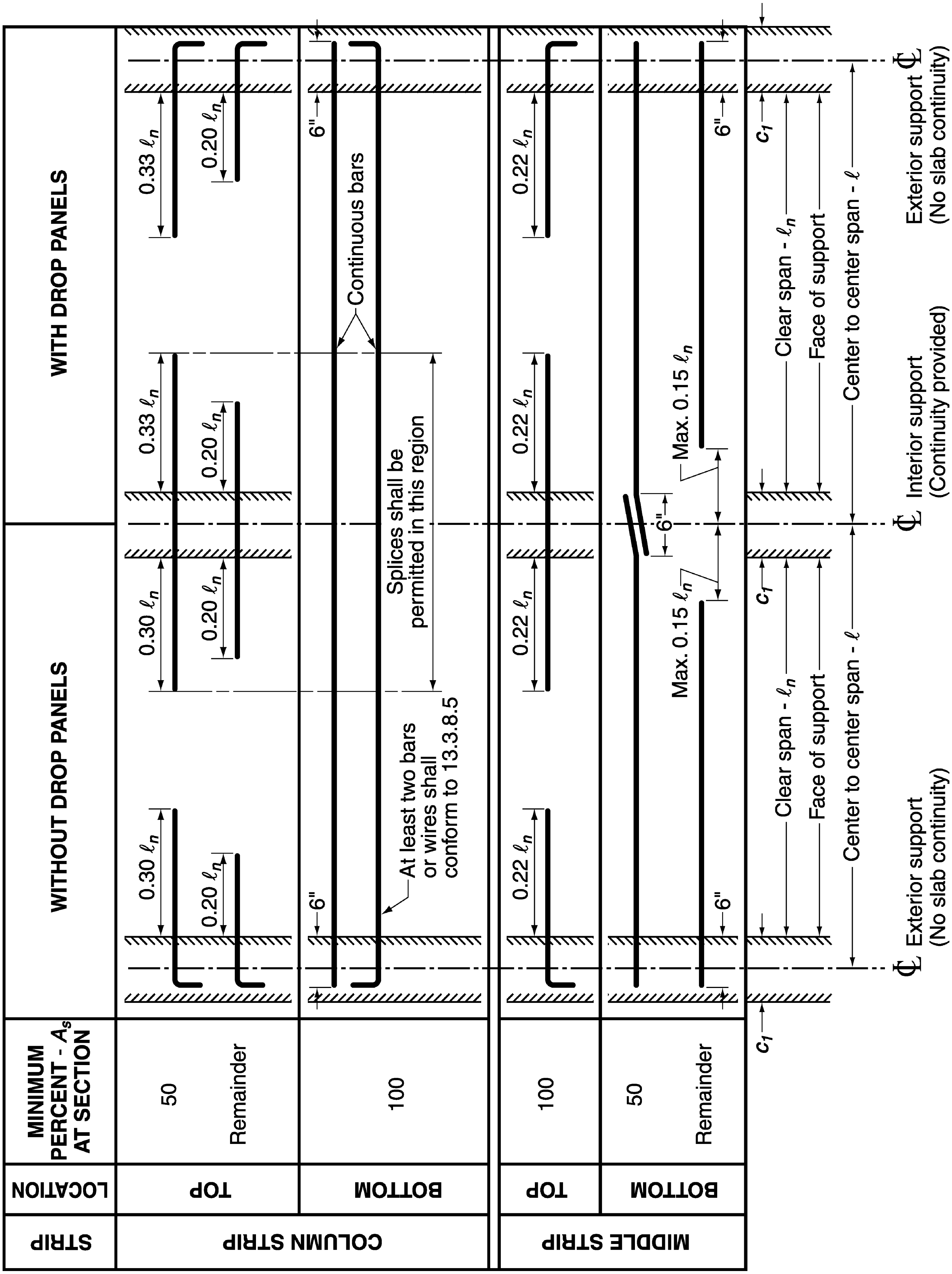

Fig. 13.3.8—Minimum extensions for reinforcement in slabs without beams (see 12.11.1 for reinforcement extension into supports).

CODE

13.4 — Openings in slab systems

13.4.1 — Openings of any size shall be permitted in slab systems if shown by analysis that the design strength is at least equal to the required strength considering 9.2 and 9.3, and that all serviceability conditions, including the specified limits on deflections, are met.

13.4.2 — As an alternate to special analysis as required by 13.4.1, openings shall be permitted in slab systems without beams only in accordance with 13.4.2.1 through 13.4.2.4.

13.4.2.1 — Openings of any size shall be permitted in the area common to intersecting middle strips, provided total amount of reinforcement required for the panel without the opening is maintained.

13.4.2.2 — In the area common to intersecting column strips, not more than one-eighth the width of column strip in either span shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening.

13.4.2.3 — In the area common to one column strip and one middle strip, not more than one-quarter of the reinforcement in either strip shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening.

13.4.2.4 — Shear requirements of 11.12.5 shall be satisfied.

13.5 — Design procedures

13.5.1 — A slab system shall be designed by any procedure satisfying conditions of equilibrium and geometric compatibility, if shown that the design strength at every section is at least equal to the required strength set forth in 9.2 and 9.3, and that all serviceability conditions, including specified limits on deflections, are met.

COMMENTARY

R13.4 — Openings in slab systems

See R11.12.5.

R13.5 — Design procedures

R13.5.1 — This section permits a designer to base a design directly on fundamental principles of structural mechanics, provided it can be demonstrated explicitly that all safety and serviceability criteria are satisfied. The design of the slab may be achieved through the combined use of classic solutions based on a linearly elastic continuum, numerical solutions based on discrete elements, or yield-line analyses, including, in all cases, evaluation of the stress conditions around the supports in relation to shear and torsion as well as flexure. The designer should consider that the design of a slab system involves more than its analysis, and justify any deviations in physical dimensions of the slab from common practice on the basis of knowledge of the expected loads and the reliability of the calculated stresses and deformations of the structure.

In the case of tank walls subjected to lateral loads that vary with depth, there are publications with tabulated shear and moment coefficients based on linear elastic analysis. For example, tables exist for rectangular wall panels supported at three or four sides, with a variety of boundary conditions. Reference 13.10 discusses the analysis and design of

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rectangular tanks, including multi-cell tanks. Reference 13.11 provides additional tables for a greater variety of loading cases and boundary conditions.

13.5.1.1 — Design of a slab system for gravity loads and uniform fluid pressures, including slab and beams (if any) between supports and supporting columns or walls forming orthogonal frames, by either the direct design method of 13.6 or the equivalent frame method of 13.7 shall be permitted.

R13.5.1.1 — For gravity load analysis of two-way slab systems, two analysis methods are specified in 13.6 and 13.7. The specific provisions of both design methods are limited in application to orthogonal frames subject to gravity loads only. Both methods apply to two-way slabs with beams as well as to flat slabs and flat plates. In both methods, the distribution of moments to the critical sections of the slab reflects the effects of reduced stiffness of elements due to cracking and support geometry.

The provisions of the direct design method and the equivalent frame method are applicable to uniform live load pressures produced by fluids. In many environmental engineering concrete structures, there can be loading cases with fluid pressures that act upward on the underside of the slab. In these cases, the methods could also be applied to the net vertical loading.

13.5.1.2 — For lateral loads, analysis of frames shall take into account effects of cracking and reinforcement on stiffness of frame members.

R13.5.1.2 — During the life of a structure, construction loads, ordinary occupancy loads, anticipated overloads, and volume changes will cause cracking of slabs. Cracking reduces stiffness of the slab members and increases lateral flexibility when lateral loads act on the structure. Cracking of slabs should be considered in stiffness assumptions so that drift caused by wind or earthquake is not grossly underestimated.

The designer may model the structure for lateral load analysis using any approach that is shown to satisfy equilibrium and geometric compatibility and to be in reasonable agreement with test data.[13.12,13.3] The selected approach should recognize effects of cracking as well as parameters such as ℓ_2/ℓ_1, c_1/ℓ_1, and c_2/c_1. Some of the available approaches are summarized in Reference 13.14, which includes discussion on the effects of cracking. Acceptable approaches include plate-bending finite-element models, the effective beam width model, and the equivalent frame model. In all cases, framing member stiffnesses should be reduced to account for cracking.

For non-prestressed slabs, it is normally appropriate to reduce slab bending stiffness to between one-half and one-quarter of the uncracked stiffness. For prestressed construction, stiffness greater than those of cracked, non-prestressed slabs may be appropriate. When the analysis is used to determine design drifts or moment magnification, lower-bound slab stiffnesses should be assumed. When the analysis is used to study interactions of the slab with other framing elements, such as structural walls, it may be appropriate to consider a range of slab stiffnesses so that the relative importance of the slab on those interactions can be assessed.

13.5.1.3 — Combining the results of the gravity load analysis with the results of the lateral load analysis shall be permitted.

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13.5.2 — The slab and beams (if any) between supports shall be proportioned for factored moments prevailing at every section.

13.5.3 — When gravity load, wind, earthquake, or other lateral forces cause transfer of moment between slab and column, a fraction of the unbalanced moment shall be transferred by flexure in accordance with 13.5.3.2 and 13.5.3.3.

13.5.3.1 — The fraction of unbalanced moment not transferred by flexure shall be transferred by eccentricity of shear in accordance with 11.12.6.

13.5.3.2 — A fraction of the unbalanced moment given by $\gamma_f M_u$ shall be considered to be transferred by flexure within an effective slab width between lines that are one and one-half slab or drop panel thicknesses **(1.5*h*)** outside opposite faces of the column or capital, where M_u is the moment to be transferred and

$$\gamma_f = \frac{1}{1 + (2/3)\sqrt{b_1/b_2}} \qquad (13\text{-}1)$$

13.5.3.3 — For unbalanced moments about an axis parallel to the edge at exterior supports, the value of γ_f by Eq. (13-1) shall be permitted to be increased up to 1.0 provided that V_u at an edge support does not exceed $0.75\phi V_c$ or at a corner support does not exceed $0.5\phi V_c$. For unbalanced moments at interior supports, and for unbalanced moments about an axis transverse to the edge at exterior supports, the value of γ_f in Eq. (13-1) shall be permitted to be increased by up to 25 percent provided that V_u at the support does not exceed $0.4\phi V_c$. The reinforcement ratio ρ, within the effective slab width defined in 13.5.3.2, shall not exceed $0.375\rho_b$. No adjustments to γ_f shall be permitted for prestressed slab systems.

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R13.5.3 — This section is concerned primarily with slab systems without beams. Tests and experience have shown that, unless special measures are taken to resist the torsional and shear stresses, all reinforcement resisting that part of the moment to be transferred to the column by flexure should be placed between lines that are one and one-half the slab or drop panel thickness, **1.5*h***, on each side of the column. The calculated shear stresses in the slab around the column are required to conform to the requirements of 11.12.2. See R11.12.1.2 and R11.12.2.1 for more details on application of this section.

R13.5.3.3 — The ACI 318-89 code procedures remain unchanged, except that under certain conditions the designer is permitted to adjust the level of moment transferred by shear without revising member sizes. Tests indicate that some flexibility in distribution of unbalanced moments transferred by shear and flexure at both exterior and interior supports is possible. Interior, exterior, and corner supports refer to slab-column connections for which the critical perimeter for rectangular columns has four, three, or two sides, respectively. Changes in the ACI 318-95 code recognized, to some extent, design practices before the ACI 318-71 code.[13.15]

At exterior supports, for unbalanced moments about an axis parallel to the edge, the portion of moment transferred by eccentricity of shear $\gamma_v M_u$ may be reduced provided that the factored shear at the support (excluding the shear produced by moment transfer) does not exceed 75 percent of the shear capacity ϕV_c as defined in 11.12.2.1 for edge columns or 50 percent for corner columns. Tests indicate that there is no significant interaction between shear and unbalanced moment at the exterior support in such cases.[13.16,13.17] Note that as $\gamma_v M_u$ is decreased, $\gamma_f M_u$ is increased.

Tests of interior supports indicate that some flexibility in distributing unbalanced moments transferred by shear and flexure is also possible, but with more severe limitations than for exterior supports. For interior supports, the unbalanced moment transferred by flexure is permitted to be increased up to 25 percent provided that the factored shear (excluding the shear caused by the moment transfer) at the interior supports does not exceed 40 percent of the shear capacity ϕV_c as defined in 11.12.2.1.

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Tests of slab-column connections indicate that a large degree of ductility is required because the interaction between shear and unbalanced moment is critical. When the factored shear is large, the column-slab joint cannot always develop all of the reinforcement provided in the effective width. The modifications for edge, corner, or interior slab-column connections specified in 13.5.3.3 are permitted only when the reinforcement ratio (within the effective width) required to develop the unbalanced moment $\gamma_f M_u$ does not exceed $\mathbf{0.375}\rho_b$. The use of Eq. (13-1) without the modification permitted in 13.5.3.3 will generally indicate overstress conditions on the joint. The provisions of 13.5.3.3 are intended to improve ductile behavior of the column-slab joint. When a reversal of moments occurs at opposite faces of an interior support, both top and bottom reinforcement should be concentrated within the effective width. A ratio of top to bottom reinforcement of about 2 has been observed to be appropriate.

13.5.3.4 — Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist moment on the effective slab width defined in 13.5.3.2.

13.5.4 — Design for transfer of load from slabs to supporting columns or walls through shear and torsion shall be in accordance with Chapter 11.

13.6 — Direct design method

R13.6 — Direct design method

In multi-cell construction, the floors and walls of tank-type structures may sometimes qualify for the direct design method.

The direct design method consists of a set of rules for distributing moments to slab and beam sections to satisfy safety requirements and most serviceability requirements simultaneously. Three fundamental steps are involved as follows:

(1) Determination of the total factored static moment (see 13.6.2);

(2) Distribution of the total factored static moment to negative and positive sections (see 13.6.3);

(3) Distribution of the negative and positive factored moments to the column and middle strips and to the beams, if any (see 13.6.4 through 13.6.6). The distribution of moments to column and middle strips is also used in the equivalent frame method (see 13.7).

13.6.1 — Limitations

Design of slab systems within the limitations of 13.6.1.1 through 13.6.1.8 by the direct design method shall be permitted.

R13.6.1 — Limitations

The direct design method was developed from considerations of theoretical procedures for the determination of moments in slabs with and without beams, requirements for simple design and construction procedures, and precedents supplied by performance of slab systems. Consequently, the

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13.6.1.1 — There shall be a minimum of three continuous spans in each direction.

13.6.1.2 — Panels shall be rectangular, with a ratio of longer to shorter span center-to-center of supports within a panel not greater than 2.

13.6.1.3 — Successive span lengths center-to-center of supports in each direction shall not differ by more than one-third the longer span.

13.6.1.4 — Offset of columns by a maximum of 10 percent of the span (in direction of offset) from either axis between centerlines of successive columns shall be permitted.

13.6.1.5 — All loads shall be due to gravity and fluid pressures only and shall be uniformly distributed over an entire panel. For purposes of determining the dead-to-live-load ratio of Section 13.6.1.5 and when using Eq. (13-4) to determine the column and wall moments, the full or partial portion of the liquid load that is uniform over all spans shall be considered as part of the dead load. Any non-uniform portion of the liquid load due to the slope of the floor or adjacent cells not being filled shall be considered as part of the live load. All liquid loads, whether considered live or dead, shall be multiplied by the appropriate load factor of 1.7, per Chapter 9. If fluid pressures do not act simultaneously on all panels, live load, including that resulting from fluid pressures, shall not exceed three times dead load.

13.6.1.6 — For a panel with beams between supports on all sides, the relative stiffness of beams in two perpendicular directions

$$\frac{\alpha_1 \ell_2^2}{\alpha_2 \ell_1^2} \tag{13-2}$$

shall not be less than 0.2 nor greater than 5.0.

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slab systems to be designed using the direct design method should conform to the limitations in this section.

R13.6.1.1 — The primary reason for the limitation in this section is the magnitude of the negative moments at the interior support in a structure with only two continuous spans. The rules given for the direct design method assume that the slab system at the first interior negative moment section is neither fixed against rotation nor discontinuous.

R13.6.1.2 — If the ratio of the two spans (long span/short span) of a panel exceeds 2, the slab resists the moment in the shorter span essentially as a one-way slab.

R13.6.1.3 — The limitation in this section is related to the possibility of developing negative moments beyond the point where negative moment reinforcement is terminated, as prescribed in Fig. 13.3.8.

R13.6.1.4 — Columns can be offset within specified limits from a regular rectangular array. A cumulative total offset of 20 percent of the span is established as the upper limit.

R13.6.1.5 — The direct design method is based on tests[13.18] for uniform gravity loads and resulting column reactions determined by statics. Lateral loads such as wind or seismic require a frame analysis. Inverted foundation mats designed as two-way slabs (see 15.10) involve application of known column loads. Therefore, even where the soil reaction is assumed to be uniform, a frame analysis is required.

Two-way slab systems are sometimes used for tank bottoms where they are subjected to uniform fluid pressures many times larger than the dead load. As long as the fluid pressures are uniform and act on all panels, they need not be included in the limiting live-to-dead-load ratio, as they cannot produce pattern loading effects. When the fluid pressures vary significantly, such as when slabs have pronounced slope or contain cells where one may be full while the adjacent one is empty, the equivalent frame, or other method of analysis, should be used.

Sediments, which can accumulate in some tanks, should be treated as live loads because there can be pronounced pattern loading effects when tanks are drained and cleaned.

R13.6.1.6 — The elastic distribution of moments will deviate significantly from those assumed in the direct design method unless the requirements for stiffness are satisfied.

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13.6.1.7 — Moment redistribution as permitted by 8.4 shall not be applied for slab systems designed by the direct design method. See 13.6.7.

13.6.1.8 — Variations from the limitations of 13.6.1 shall be permitted if demonstrated by analysis that requirements of 13.5.1 are satisfied.

13.6.2 — Total factored static moment for a span

13.6.2.1 — Total factored static moment for a span shall be determined in a strip bounded laterally by centerline of panel on each side of centerline of supports.

13.6.2.2 — Absolute sum of positive and average negative factored moments in each direction shall not be less than

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} \qquad (13\text{-}3)$$

13.6.2.3 — Where the transverse span of panels on either side of the centerline of supports varies, ℓ_2 in Eq. (13-3) shall be taken as the average of adjacent transverse spans.

13.6.2.4 — When the span adjacent and parallel to an edge is being considered, the distance from edge to panel centerline shall be substituted for ℓ_2 in Eq. (13-3).

13.6.2.5 — Clear span ℓ_n shall extend from face to face of columns, capitals, brackets, or walls. Value of ℓ_n used in Eq. (13-3) shall not be less than $0.65\ell_1$. Circular or regular polygon shaped supports shall be treated as square supports with the same area.

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R13.6.1.7 — Moment redistribution as permitted by 8.4 is not intended where approximate values for bending moments are used. For the direct design method, 10 percent modification is allowed by 13.6.7.

R13.6.1.8 — The designer is permitted to use the direct design method even if the structure does not fit the limitations in this section, provided it can be shown by analysis that the particular limitation does not apply to that structure. For a slab system carrying a non-movable load (such as a water reservoir in which the load on all panels is expected to be the same), the designer need not satisfy the live load limitation of 13.6.1.5.

R13.6.2 — Total factored static moment for a span

R13.6.2.2 — Equation (13-3) follows directly from Nichol's derivation[13.19] with the simplifying assumption that the reactions are concentrated along the faces of the support perpendicular to the span considered. In general, the designer will find it expedient to calculate static moments for two adjacent half panels that include a column strip with a half middle strip along each side.

R13.6.2.5 — If a supporting member does not have a rectangular cross section or if the sides of the rectangle are not parallel to the spans, it is to be treated as a square support having the same area, as illustrated in Fig. R13.6.2.5.

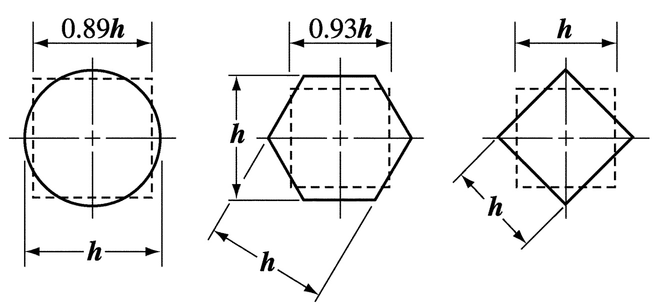

Fig. R13.6.2.5—Examples of equivalent square section for supporting members.

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13.6.3 — Negative and positive factored moments

13.6.3.1 — Negative factored moments shall be located at face of rectangular supports. Circular or regular polygon shaped supports shall be treated as square supports with the same area.

13.6.3.2 — In an interior span, total static moment M_o shall be distributed as follows:

Negative factored moment.................................0.65

Positive factored moment 0.35

13.6.3.3 — In an end span, total factored static moment M_o shall be distributed as follows:

	(1)	(2)	(3)	(4)	(5)
	Exterior edge unrestrained	Slab with beams between all supports	Slab without beams between interior supports: Without edge beam	Slab without beams between interior supports: With edge beam	Exterior edge fully restrained
Interior negative factored moment	0.75	0.70	0.70	0.70	0.65
Positive factored moment	0.63	0.57	0.52	0.50	0.35
Exterior negative factored moment	0	0.16	0.26	0.30	0.65

13.6.3.4 — Negative moment sections shall be designed to resist the larger of the two interior negative factored moments determined for spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with stiffnesses of adjoining elements.

13.6.3.5 — Edge beams or edges of slab shall be proportioned to resist in torsion their share of exterior negative factored moments.

COMMENTARY

R13.6.3 — Negative and positive factored moments

R13.6.3.3 — The moment coefficients for an end span are based on the equivalent column stiffness expressions from References 13.20, 13.21, and 13.22. The coefficients for an unrestrained edge would be used, for example, if the slab were simply supported on a masonry or concrete wall. Those for a fully restrained edge would apply if the slab were constructed integrally with a concrete wall having a flexural stiffness so large compared with that of the slab that little rotation occurs at the slab-to-wall connection.

For other than unrestrained or fully restrained edges, coefficients in the table were selected to be near the upper bound of the range for positive moments and interior negative moments. As a result, exterior negative moments were usually closer to a lower bound. The exterior negative moment capacity for most slab systems is governed by minimum reinforcement to control cracking. The final coefficients in the table have been adjusted so that the absolute sum of the positive and average moments equal M_o.

For two-way slab systems with beams between supports on all sides (two-way slabs), moment coefficients of Column (2) apply. For slab systems without beams between interior supports (flat plates and flat slabs), the moment coefficients of Column (3) or (4) apply, without or with an edge (spandrel) beam, respectively.

In the 1977 ACI Building Code, distribution factors defined as a function of the stiffness ratio of the equivalent exterior support were used for proportioning the total static moment M_o in an end span. The approach may be used in place of values in 13.6.3.3.

R13.6.3.4 — The differences in slab moment on either side of a column or other type of support should be accounted for in the design of the support. If an analysis is made to distribute unbalanced moments, flexural stiffness may be obtained on the basis of the gross concrete section of the members involved.

R13.6.3.5 — Moments perpendicular to, and at the edge of, the slab structure should be transmitted to the supporting columns or walls. Torsional stresses caused by the moment assigned to the slab should be investigated.

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13.6.3.6 — The gravity load moment to be transferred between slab and edge column in accordance with 13.5.3.1 shall be $\mathbf{0.3}\boldsymbol{M_o}$.

13.6.4 — Factored moments in column strips

13.6.4.1 — Column strips shall be proportioned to resist the following portions in percent of interior negative factored moments:

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1) = 0$	75	75	75
$(\alpha_1\ell_2/\ell_1) \geq 1.0$	90	75	45

Linear interpolations shall be made between values shown.

13.6.4.2 — Column strips shall be proportioned to resist the following portions in percent of exterior negative factored moments:

ℓ_2/ℓ_1		0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1) = 0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	75	75	75
$(\alpha_1\ell_2/\ell_1) \geq 1.0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	90	75	45

Linear interpolations shall be made between values shown.

13.6.4.3 — Where supports consist of columns or walls extending for a distance equal to or greater than three-quarters the span length ℓ_2 used to compute M_o, negative moments shall be considered to be uniformly distributed across ℓ_2.

13.6.4.4 — Column strips shall be proportioned to resist the following portions in percent of positive factored moments:

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1) = 0$	60	60	60
$(\alpha_1\ell_2/\ell_1) \geq 1.0$	90	75	45

Linear interpolations shall be made between values shown.

13.6.4.5 — For slabs with beams between supports, the slab portion of column strips shall be proportioned to resist that portion of column strip moments not resisted by beams.

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R13.6.4, R13.6.5, and R13.6.6 — Factored moments in column strips, beams, and middle strips

The rules given for assigning moments to the column strips, beams, and middle strips are based on studies of moments in linearly elastic slabs with different beam stiffness[13.23] tempered by the moment coefficients that have been used successfully.

For the purpose of establishing moments in the half column strip adjacent to an edge supported by a wall, ℓ_n in Eq. (13-3) may be assumed equal to ℓ_n of the parallel adjacent column to column span, and the wall may be considered as a beam having a moment of inertia I_b equal to infinity.

R13.6.4.2 — The effect of the torsional stiffness parameter β_t is to assign all of the exterior negative factored moment to the column strip, and none to the middle strip, unless the beam torsional stiffness is high relative to the flexural stiffness of the supported slab. In the definition of β_t, the shear modulus has been taken as $E_{cb}/2$.

Where walls are used as supports along column lines, they can be regarded as very stiff beams with an $\alpha_1\ell_2/\ell_1$ value greater than one. Where the exterior support consists of a wall perpendicular to the direction in which moments are being determined, β_t may be taken as zero if the wall is of masonry without torsional resistance, and β_t may be taken as 2.5 for a concrete wall with great torsional resistance which is monolithic with the slab.

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13.6.5 — Factored moments in beams

13.6.5.1 — Beams between supports shall be proportioned to resist 85 percent of column strip moments if $(\alpha_1\ell_2/\ell_1)$ is equal to or greater than 1.0.

13.6.5.2 — For values of $\alpha_1\ell_2/\ell_1$ between 1.0 and zero, proportion of column strip moments resisted by beams shall be obtained by linear interpolation between 85 and zero percent.

13.6.5.3 — In addition to moments calculated for uniform loads according to 13.6.2.2, 13.6.5.1, and 13.6.5.2, beams shall be proportioned to resist all moments caused by concentrated or linear loads applied directly to beams, including weight of projecting beam stem above or below the slab.

13.6.6 — Factored moments in middle strips

13.6.6.1 — That portion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to corresponding half middle strips.

13.6.6.2 — Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.

13.6.6.3 — A middle strip adjacent to and parallel with a wall-supported edge shall be proportioned to resist twice the moment assigned to the half middle strip corresponding to the first row of interior supports.

13.6.7 — Modification of factored moments

Modification of negative and positive factored moments by 10 percent shall be permitted provided the total static moment for a panel in the direction considered is not less than that required by Eq. (13-3).

13.6.8 — Factored shear in slab systems with beams

13.6.8.1 — Beams with $\alpha_1\ell_2/\ell_1$ equal to or greater than 1.0 shall be proportioned to resist shear caused by factored loads on tributary areas that are bounded by 45 deg lines drawn from the corners of the panels and the centerlines of the adjacent panels parallel to the long sides.

13.6.8.2 — In proportioning of beams with $\alpha_1\ell_2/\ell_1$ less than 1.0 to resist shear, linear interpolation, assuming beams carry no load at $\alpha_1 = 0$, shall be permitted.

13.6.8.3 — In addition to shears calculated according to 13.6.8.1 and 13.6.8.2, beams shall be proportioned to resist shears caused by factored loads applied directly on beams.

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R13.6.5 — Factored moments in beams

Loads assigned directly to beams are in addition to the uniform dead load of slab, uniform superimposed dead loads such as the ceiling, floor finish, or assumed equivalent partition loads, and uniform live loads. All of these loads are normally included with w_u in Eq. (13-3). Linear loads applied directly to beams include partition walls over or along beam centerlines and additional dead load of the projecting beam stem. Concentrated loads include posts above or hangers below the beams. For the purpose of assigning directly applied loads, only loads located within the width of the beam stem should be considered as directly applied to the beams. (The effective width of a beam as defined in 13.2.4 is solely for strength and relative stiffness calculations.) Line loads and concentrated loads located on the slab away from the beam stem require special consideration to determine their apportionment to slab and beams.

R13.6.8 — Factored shear in slab systems with beams

The tributary area for computing shear on an interior beam is shown shaded in Fig. R13.6.8. If the stiffness for the beam $\alpha_1\ell_2/\ell_1$ is less than 1.0, the shear on the beam may be obtained by linear interpolation. In such cases, the beams framing into the column will not account for all the shear force applied on the column. The remaining shear force will produce shear stresses in the slab around the column that should be checked in the same manner as for flat slabs, as required by 13.6.8.4. Sections 13.6.8.1 through 13.6.8.3 do not apply to the calculation of torsional moments on the beams. These moments should be based on the calculated flexural moments acting on the sides of the beam.

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13.6.8.4 — Computation of slab shear strength on the assumption that load is distributed to supporting beams in accordance with 13.6.8.1 or 13.6.8.2 shall be permitted. Resistance to total shear occurring on a panel shall be provided.

13.6.8.5 — Shear strength shall satisfy requirements of Chapter 11.

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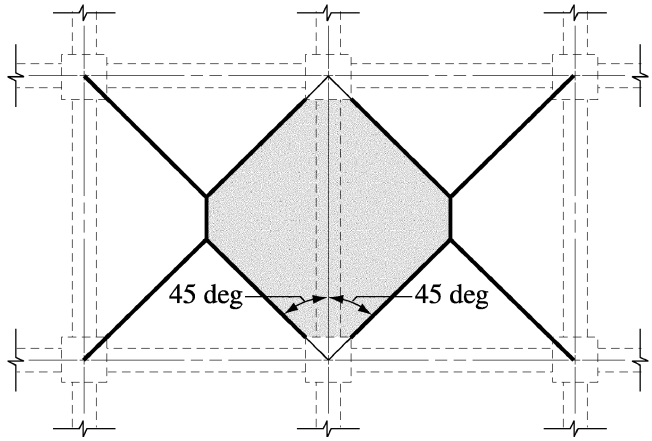

Fig. R13.6.8—Tributary area for shear on interior beam.

13.6.9 — Factored moments in columns and walls

13.6.9.1 — Columns and walls built integrally with a slab system shall resist moments caused by factored loads on the slab system.

13.6.9.2 — At an interior support, supporting elements above and below the slab shall resist the moment specified by Eq. (13-4) in direct proportion to their stiffnesses unless a general analysis is made.

$$M = 0.07[(w_d + 0.5w_\ell)\ell_2 \ell_n^2 - w_d'\ell_2'(\ell_n')^2] \quad (13\text{-}4)$$

where w_d', ℓ_2', and ℓ_n' refer to shorter span.

R13.6.9 — Factored moments in columns and walls

Equation (13-4) refers to two adjoining spans, with one span longer than the other, with full dead load plus one-half live load applied on the longer span and only dead load applied on the shorter span.

The term w_ℓ in Eq. (13-4) need not include uniform fluid pressures that act simultaneously on both spans. Where there is a variation in fluid pressure due to a moderate slope, or due to adjacent cells being full or empty, the full value of the difference in average pressure between adjacent spans should be used in place of $0.5w_\ell$ in Eq. (13-4).

Design and detailing of the reinforcement transferring the moment from the slab to the edge column is critical to both the performance and the safety of flat slabs or flat plates without edge beams or cantilever slabs. It is important that complete design details be shown on design drawings, such as concentration of reinforcement over the column by closer spacing or additional reinforcement.

13.7 — Equivalent frame method

13.7.1 — Design of slab systems by the equivalent frame method shall be based on assumptions given in 13.7.2 through 13.7.6, and all sections of slabs and supporting members shall be proportioned for moments and shears thus obtained.

13.7.1.1 — Where metal column capitals are used, it shall be permitted to take account of their contributions to stiffness and resistance to moment and to shear.

R13.7 — Equivalent frame method

The equivalent frame method involves the representation of the three-dimensional slab system by a series of two-dimensional frames that are then analyzed for loads acting in the plane of the frames. The negative and positive moments so determined at the critical design sections of the frame are distributed to the slab sections in accordance with 13.6.4 (column strips), 13.6.5 (beams), and 13.6.6 (middle strips). The equivalent frame method is based on studies reported in References 13.20, 13.21, and 13.22. Many of the details of the equivalent frame method given in the Commentary in the ACI 318-89 code were removed in the ACI 318-95 code.

R13.7.1.1 — Metal column capitals (that is, shear heads) are seldom used in liquid-containing structures.

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13.7.1.2 — It shall be permitted to neglect the change in length of columns and slabs due to direct stress, and deflections due to shear.

13.7.2 — Equivalent frame

13.7.2.1 — The structure shall be considered to be made up of equivalent frames on column lines taken longitudinally and transversely through the building.

13.7.2.2 — Each frame shall consist of a row of columns or supports and slab-beam strips, bounded laterally by the centerline of panel on each side of the centerline of columns or supports.

13.7.2.3 — Columns or supports shall be assumed to be attached to slab-beam strips by torsional members (see 13.7.5) transverse to the direction of the span for which moments are being determined and extending to bounding lateral panel centerlines on each side of a column.

13.7.2.4 — Frames adjacent and parallel to an edge shall be bounded by that edge and the centerline of adjacent panel.

13.7.2.5 — Analysis of each equivalent frame in its entirety shall be permitted. Alternatively, for gravity loading, a separate analysis of each floor or roof with far ends of columns considered fixed shall be permitted.

13.7.2.6 — Where slab-beams are analyzed separately, determination of moment at a given support assuming that the slab-beam is fixed at any support two panels distant therefrom shall be permitted, provided the slab continues beyond that point.

COMMENTARY

R13.7.2 — Equivalent frame

Application of the equivalent frame to a regular structure is illustrated in Fig. R13.7.2. The three-dimensional building is divided into a series of two-dimensional frame bents (equivalent frames) centered on column or support centerlines with each frame extending the full height of the building. The width of each equivalent frame is bounded by the centerlines of the adjacent panels. The complete analysis of a slab system for a building consists of analyzing a series of equivalent (interior and exterior) frames spanning longitudinally and transversely through the building.

The equivalent frame comprises three parts: (1) the horizontal slab strip, including any beams spanning in the direction of the frame; (2) the columns or other vertical supporting members, extending above and below the slab; and (3) the elements of the structure that provide moment transfer between the horizontal and vertical members.

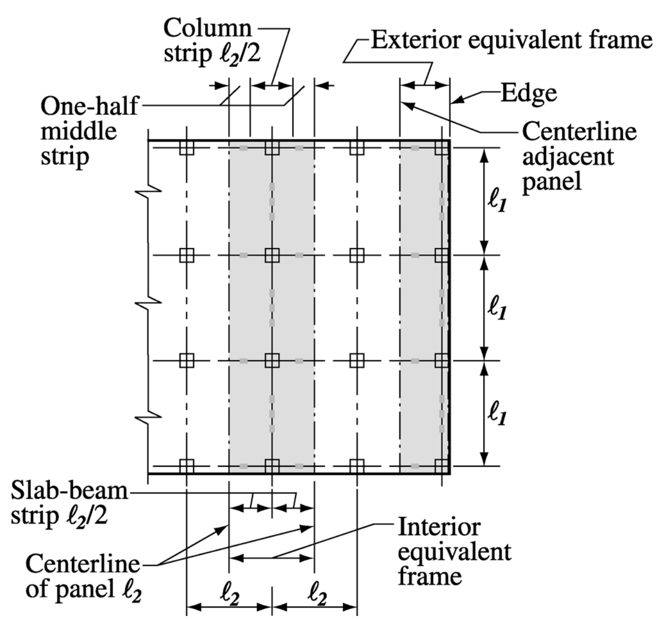

Fig. R13.7.2—Definitions of equivalent frame.

CODE

13.7.3 — Slab-beams

13.7.3.1 — Determination of the moment of inertia of slab-beams at any cross section outside of joints or column capitals using the gross area of concrete shall be permitted.

13.7.3.2 — Variation in moment of inertia along axis of slab-beams shall be taken into account.

13.7.3.3 — Moment of inertia of slab-beams from center of column to face of column, bracket, or capital shall be assumed equal to the moment of inertia of the slab-beam at face of column, bracket, or capital divided by the quantity $(1 - c_2/\ell_2)^2$ where c_2 and ℓ_2 are measured transverse to the direction of the span for which moments are being determined.

13.7.4 — Columns

13.7.4.1 — Determination of the moment of inertia of columns at any cross section outside of joints or column capitals using the gross area of concrete shall be permitted.

13.7.4.2 — Variation in moment of inertia along axis of columns shall be taken into account.

13.7.4.3 — Moment of inertia of columns from top to bottom of the slab-beam at a joint shall be assumed infinite.

COMMENTARY

R13.7.3 — Slab-beams

R13.7.3.3 — A support is defined as a column, capital, bracket, or wall. Note that a beam is not considered to be a support member for the equivalent frame.

R13.7.4 — Columns

Column stiffness is based on the length of the column from mid-depth of slab above to mid-depth of slab below. Column moment of inertia is computed on the basis of its cross section, taking into account the increase in stiffness provided by the capital, if any.

When slab-beams are analyzed separately for gravity loads, the concept of an equivalent column, combining the stiffness of the slab-beam and torsional member into a composite element, is used. The column flexibility is modified to account for the torsional flexibility of the slab-to-column connection that reduces its efficiency for transmission of moments. The equivalent column consists of the actual columns above and below the slab-beam plus "attached" torsional members on each side of the columns extending to the centerline of the adjacent panels as shown in Fig. R13.7.4.

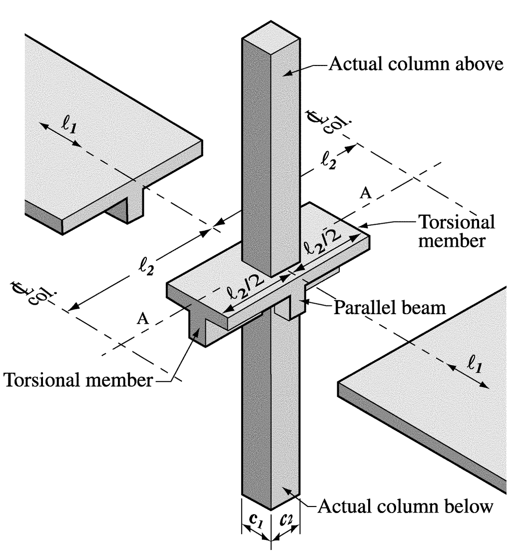

Fig. R13.7.4—Equivalent column (column plus torsional members).

CODE

13.7.5 — Torsional members

13.7.5.1 — Torsional members (see 13.7.2.3) shall be assumed to have a constant cross section throughout their length consisting of the largest of (a), (b), and (c):

(a) A portion of slab having a width equal to that of the column, bracket, or capital in the direction of the span for which moments are being determined;

(b) For monolithic or fully composite construction, the portion of slab specified in (a) plus that part of the transverse beam above and below the slab;

(c) The transverse beam as defined in 13.2.4.

13.7.5.2 — Where beams frame into columns in the direction of the span for which moments are being determined, the torsional stiffness shall be multiplied by the ratio of the moment of inertia of the slab with such a beam to the moment of inertia of the slab without such a beam.

13.7.6 — Arrangement of live load

13.7.6.1 — When loading pattern is known, the equivalent frame shall be analyzed for that load.

13.7.6.2 — When live load is variable but does not exceed three-quarters of the dead load, or the nature of live load is such that all panels will be loaded simultaneously, it shall be permitted to assume that maximum factored moments occur at all sections with full factored live load on entire slab system.

13.7.6.3 — For loading conditions other than those defined in 13.7.6.2, it shall be permitted to assume that maximum positive factored moment near midspan of a panel occurs with three-quarters of the full factored live load on the panel and on alternate panels; and it shall be permitted to assume that

COMMENTARY

R13.7.5 — Torsional members

Computation of the stiffness of the torsional member requires several simplifying assumptions. If no transverse-beam frames into the column, a portion of the slab equal to the width of the column or capital is assumed to be the torsional member. If a beam frames into the column, T-beam or L-beam action is assumed, with the flanges extending on each side of the beam a distance equal to the projection of the beam above or below the slab, but not greater than four times the thickness of the slab. Furthermore, it is assumed that no torsional rotation occurs in the beam over the width of the support.

The member sections to be used for calculating the torsional stiffness are defined in 13.7.5.1. In the ACI 318-89 code, Eq. (13-6) specified the stiffness coefficient K_t of the torsional members. The approximate expression for K_t has been moved to the commentary, and the expression for the torsional constant (Eq. (13-7) in the ACI 318-89 code) is now defined in 13.0.

Studies of three-dimensional analyses of various slab configurations suggest that a reasonable value of the torsional stiffness can be obtained by assuming a moment distribution along the torsional member that varies linearly from a maximum at the center of the column to zero at the middle of the panel. The assumed distribution of unit twisting moment along the column centerline is shown in Fig. R13.7.5.

An approximate expression for the stiffness of the torsional member, based on the results of three-dimensional analyses of various slab configurations (References 13.18, 13.19, and 13.20) is given below as

$$K_t = \sum \frac{9E_{cs}C}{\ell_2\left(1 - \frac{c_2}{\ell_2}\right)^3}$$

where an expression for C is given in 13.0.

R13.7.6 — Arrangement of live load

The use of only three-quarters of the full-factored live load for maximum moment loading patterns is based on the fact that maximum negative and maximum positive live load moments cannot occur simultaneously and that redistribution of maximum moments is thus possible before failure occurs. This procedure, in effect, permits some local overstress under the full-factored live load if it is distributed in the prescribed manner, but still ensures that the ultimate capacity of the slab

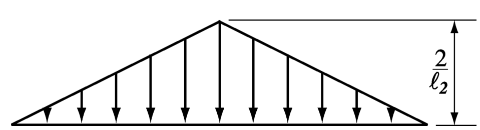

Fig. R13.7.5—Distribution of unit twisting moment along column centerline AA shown in Fig. R13.7.4.

CODE

maximum negative factored moment in the slab at a support occurs with three-quarters of the full live load on adjacent panels only.

13.7.6.4 — Factored moments shall be taken not less than those occurring with full factored live load on all panels.

13.7.7 — Factored moments

13.7.7.1 — At interior supports, the critical section for negative factored moment (in both column and middle strips) shall be taken at face of rectilinear supports, but not farther away than $\mathbf{0.175\ell_1}$ from center of a column.

13.7.7.2 — At exterior supports provided with brackets or capitals, the critical section for negative factored moment in the span perpendicular to an edge shall be taken at a distance from face of supporting element not greater than one-half the projection of bracket or capital beyond face of supporting element.

13.7.7.3 — Circular or regular polygon shaped supports shall be treated as square supports with the same area for location of critical section for negative design moment.

13.7.7.4 — When slab systems within limitations of 13.6.1 are analyzed by the equivalent frame method, it shall be permitted to reduce the resulting computed moments in such proportion that the absolute sum of the positive and average negative moments used in design need not exceed the value obtained from Eq. (13-3).

13.7.7.5 — Distribution of moments at critical sections across the slab-beam strip of each frame to column strips, beams, and middle strips as provided in 13.6.4, 13.6.5, and 13.6.6 shall be permitted if the requirement of 13.6.1.6 is satisfied.

COMMENTARY

system after redistribution of moment is not less than that required to carry the full factored dead and live loads on all panels.

R13.7.7 — Factored moments

R13.7.7.1 to R13.7.7.3 — These code sections adjust the negative factored moments to the face of the supports. The adjustment is modified at an exterior support to limit reductions in the exterior negative moment. Figure R13.6.2.5 illustrates several equivalent rectangular supports for use in establishing faces of supports for design with nonrectangular supports.

R13.7.7.4 — Previous 318 codes have contained this section. It is based on the principle that if two different methods are prescribed to obtain a particular answer, the code should not require a value greater than the least acceptable value. Due to the long satisfactory experience with designs having total factored static moments not exceeding those given by Eq. (13-3), it is considered that these values are satisfactory for design when applicable limitations are met.

CHAPTER 14 — WALLS

CODE

14.0 — Notation

A_g = gross area of section, in.^{2}
A_s = area of longitudinal tension reinforcement in wall segment, in.^{2}
A_{se} = area of effective longitudinal tension reinforcement in wall segment, in.^{2}, as calculated by Eq. (14-8)
c = distance from extreme compression fiber to neutral axis, in.
d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in.
E_c = modulus of elasticity of concrete, psi
f_c' = specified compressive strength of concrete, psi
f_y = specified yield strength of nonprestressed reinforcement, psi
h = overall thickness of member, in.
I_{cr} = moment of inertia of cracked section transformed to concrete, in.^{4}
I_e = effective moment of inertia for computation of deflection, in.^{4}
k = effective length factor
ℓ_c = vertical distance between supports, in.
ℓ_w = horizontal length of wall, in.
M = maximum unfactored moment due to service loads, including $P\Delta$ effects, in.-lb
M_a = maximum moment in member at stage deflection is computed, in.-lb
M_{cr} = moment causing flexural cracking due to applied lateral and vertical loads, in.-lb
M_n = nominal moment strength at section, in.-lb
M_{sa} = maximum unfactored applied moment due to service loads, not including $P\Delta$ effects, in.-lb
M_u = factored moment at section, including $P\Delta$ effects, in.-lb
M_{ua} = moment at the midheight section of the wall due to factored lateral and eccentric vertical loads, in.-lb
n = modular ratio of elasticity, but not less than 6
= E_s/E_c
P_{nw} = nominal axial load strength of wall designed by the empirical method, lb (see 14.5)
P_s = unfactored axial load at the design (midheight) section including effects of self-weight, lb
P_u = factored axial load, lb
Δ_s = maximum deflection at or near midheight due to service loads, in.
Δ_u = deflection at midheight of wall due to factored loads, in.
ϕ = strength reduction factor. See 9.3

COMMENTARY

R14.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip. However, the values of constants used in equations are dependent on the units of the parameters. If the user chooses to use alternate units, the constants must be adjusted accordingly.

CODE

ρ = ratio of tension reinforcement
= $A_s/(\ell_w d)$

ρ_b = reinforcement ratio producing balanced strain conditions

14.1 — Scope

14.1.1 — Provisions of Chapter 14 shall apply for design of planar walls subject to axial load, with or without flexure; and circular walls with axial load and hoop tension, with or without flexure.

14.1.2 — Cantilever retaining walls are designed according to flexural design provisions of Chapter 10 with minimum horizontal reinforcement according to 14.3.3.

14.2 — General

14.2.1 — Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.

14.2.2 — Walls subject to axial loads shall be designed in accordance with 14.2, 14.3, and either 14.4, 14.5, or 14.8.

14.2.3 — Design for shear shall be in accordance with 11.10.

14.2.4 — Unless otherwise demonstrated by an analysis, the horizontal length of wall considered as effective for each concentrated load shall not exceed center-to-center distance between loads, nor the bearing width plus four times the wall thickness.

14.2.5 — Compression members built integrally with walls shall conform to 10.8.2.

14.2.6 — Walls shall be anchored to intersecting elements such as floors and roofs, or to columns, pilasters, buttresses, of intersecting walls, and to footings.

14.2.7 — Transfer of force to footing at base of wall shall be in accordance with 15.8.

14.3 — Minimum reinforcement

COMMENTARY

R14.1 — Scope

Chapter 14 applies generally to walls as vertical load-carrying members. Cantilever retaining walls are designed according to the flexural design provisions of Chapter 10. Walls designed to resist shear forces, such as shearwalls, should be designed in accordance with Chapter 14 and 11.10 as applicable.

In the ACI 318-77 code, walls could be designed according to Chapter 14 or 10.15. In the ACI 318-83 code these two were combined in Chapter 14.

R14.2 — General

Walls should be designed to resist all loads to which they are subjected, including eccentric axial loads and lateral forces. Design is to be carried out in accordance with 14.4 unless the wall meets the requirements of 14.5.1.

In either case, walls may be designed using either the strength design method of the code or the alternate design method of Appendix I in accordance with I.6.3.

R14.3 — Minimum reinforcement

The requirements of 14.3 are similar to those in previous ACI Building Codes. These apply to walls designed according to 14.4, 14.5, or 14.8. For walls resisting horizontal shear forces in the plane of the wall, reinforcement designed according to 11.10.9.2 and 11.10.9.4 may exceed the minimum reinforcement specified in 14.3.

CODE

14.3.1 — In concretes made with ASTM C 150 or C 595 cements, the minimum vertical and horizontal reinforcement for temperature and shrinkage shall be in accordance with 14.3.2 and 14.3.3.

14.3.2 — Minimum ratio of vertical reinforcement area to gross concrete area shall be 0.0030.

14.3.3 — Minimum ratio of horizontal reinforcement area to gross concrete area shall be based on the length between movement joints, and shall conform to 7.12.2.1.

14.3.4 — Walls more than 10 in. thick shall have reinforcement for each direction placed in two layers parallel to faces of wall in accordance with the following:

(a) One layer, consisting of not less than one-half nor more than two-thirds of total reinforcement required for each direction, shall be placed with not less than the concrete cover limits given in 7.7 nor more than one-third the thickness of wall from the exterior surface;

(b) The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than the concrete cover limits given in 7.7 nor more than one-third of the thickness of wall from the interior surface.

14.3.5 — Vertical and horizontal reinforcement shall not be placed farther apart than 12 in.

14.3.6 — Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

14.3.7 — In addition to the minimum reinforcement previously specified, an amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening. Such bars shall be extended to develop the bar beyond the corners of the angular opening, or beyond the intersection with other trim bars of circular openings, but not less than 24 in.

14.4 — Walls designed as compression members

Except as provided in 14.5, walls subjected to axial load, or combined flexure and axial load, or walls that receive their vertical stability from curvature shall be designed as compression members in accordance with provisions of 10.2, 10.3, 10.10, 10.11, 10.12, 10.13, 10.14, 10.17, 14.2, and 14.3.

COMMENTARY

R14.3.7 — Where there are openings in walls with lateral loads, the reinforcement described in 14.3.7 is considered minimum reinforcement. The wall should be designed to replace the strength lost at the opening and transfer the load around the opening.

CODE

14.5 — Empirical design method

14.5.1 — Walls of solid rectangular cross section shall be permitted to be designed by the empirical provisions of 14.5 if the resultant of all factored loads is located within the middle third of the overall thickness of the wall and all limits of 14.2, 14.3, and 14.5 are satisfied.

14.5.2 — Design axial load strength ϕP_{nw} of a wall satisfying limitations of 14.5.1 shall be computed by Eq. (14-1) unless designed in accordance with 14.4

$$\phi P_{nw} = 0.55\phi f'_c A_g\left[1 - \left(\frac{k\ell_c}{32h}\right)^2\right] \quad (14\text{-}1)$$

where ϕ = **0.70** and effective length factor k shall be:

For walls braced top and bottom against lateral translation and

(a) restrained against rotation at one or both ends (top and/or bottom)..0.8

(b) unrestrained against rotation at both ends 1.0

For walls not braced against lateral translation 2.0

14.5.3 — Minimum thickness of walls designed by empirical design method

14.5.3.1 — Thickness of bearing walls that do not receive their vertical stability from curvature shall not be less than 1/25 the supported height or length, whichever is less, nor less than 8 in.

COMMENTARY

R14.5 — Empirical design method

The empirical design method applies only to solid rectangular cross sections. All other shapes must be designed according to 14.4.

Eccentric loads and lateral forces are used to determine the total eccentricity of the factored axial load P_u. When the resultant load for all applicable load combinations falls within the middle third of the wall thickness (eccentricity not greater than $h/6$) at all sections along the length of the undeformed wall, the empirical design method may be used. The design is then carried out considering P_u as the concentric load. The factored axial load P_u should be less than or equal to the design axial load strength ϕP_{nw} computed by Eq. (14-1), $P_u \leq \phi P_{nw}$.

With the 1980 ACI 318 supplement, Eq. (14-1) was revised to reflect the general range of end conditions encountered in wall designs. The wall strength equation in the ACI 318-77 code was based on the assumption of a wall with top and bottom fixed against lateral movement, and with moment restraint at one end corresponding to an effective length factor between 0.8 and 0.9. Axial load strength values determined from the original equation were unconservative when compared with test results[14.1] for walls with pinned conditions at both ends, as can occur in some precast and tilt-up applications, or when the top of the wall is not effectively braced against translation, as occurs with many freestanding walls or in large structures where significant roof diaphragm deflections occur due to wind and seismic loads. Equation (14-1) gives the same results as the ACI 318-77 code for walls braced against translation and with reasonable base restraint against rotation.[14.2] Values of effective vertical length factors k are given for commonly occurring wall end conditions. The end condition “restrained against rotation” required for a k-factor of 0.8 implies attachment to a member having flexural stiffness EI/ℓ at least as large as that of the wall.

The slenderness portion of Eq. (14-1) results in relatively comparable strengths by either 14.3 or 14.4 for members loaded at the middle third of the thickness with different braced and restrained end conditions. See Fig. R14.5.

R14.5.3 — Minimum thickness of walls designed by empirical design method

The minimum thickness requirements need not be applied to walls designed according to 14.4.

CODE

14.6 — Minimum wall thickness

14.6.1 — Thickness of nonbearing walls that do not receive their vertical stability from curvature shall not be less than 6 in., nor less than 1/30 the least distance between members that supply lateral support.

14.6.2 — The minimum thickness of conventionally reinforced cast-in-place concrete walls that are in contact with liquids and are at least 10 ft high shall be 12 in.

14.7 — Walls as grade beams

14.7.1 — Walls designed as grade beams shall have top and bottom reinforcement as required for moment in accordance with provisions of 10.2 through 10.7. Design for shear shall be in accordance with provisions of Chapter 11.

14.7.2 — Portions of grade beam walls exposed above grade shall also meet requirements of 14.3.

14.8 — Alternative design of slender walls

14.8.1 — When flexural tension controls the design of a wall, the requirements of 14.8 are considered to satisfy 10.10.

14.8.2 — Walls designed by the provisions of 14.8 shall satisfy 14.8.2.1 through 14.8.2.6.

14.8.2.1 — The wall panel shall be designed as a simply supported, axially loaded member subjected to an out-of-plane uniform lateral load, with maximum moments and deflections occurring at midspan.

14.8.2.2 — The cross section is constant over the height of the panel.

14.8.2.3 — The reinforcement ratio ρ shall not exceed $0.6\rho_b$.

14.8.2.4 — Reinforcement shall provide a design strength

$$\phi M_n \geq M_{cr} \tag{14-2}$$

where M_{cr} shall be obtained using the modulus of rupture given by Eq. (9-11).

14.8.2.5 — Concentrated gravity loads applied to the wall above the design flexural section shall be assumed to be distributed over a width:

(a) Equal to the bearing width, plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design section; but

(b) Not greater than the spacing of the concentrated loads; and

(c) Not extending beyond the edges of the wall panel.

COMMENTARY

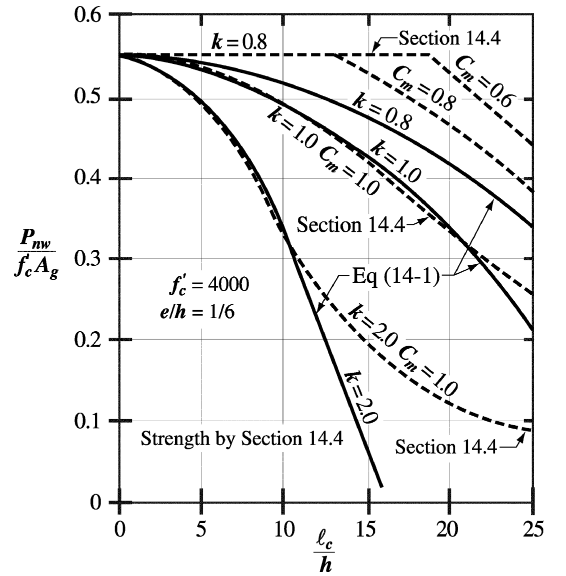

Fig. R14.5—Empirical design of walls, Eq. (14-1), versus 14.4.

R14.8 — Alternative design of slender walls

Section 14.8 is based on the corresponding requirements in the Uniform Building Code (UBC)[14.3] and experimental research.[14.4]

The procedure is presented as an alternative to the requirements of 10.10 for the out-of-plane design of precast wall panels, where the panels are restrained against overturning at the top.

The procedure, as prescribed in the UBC,[14.3] has been converted from working stress to factored load design.

Panels that have windows or other large openings are not considered to have constant cross section over the height of the panel. Such walls are to be designed taking into account the effects of openings.

Many aspects of the design of tilt-up walls and buildings are discussed in References 14.5 and 14.6.

CODE

COMMENTARY

14.8.2.6 — Vertical stress P_u/A_g at the midheight section shall not exceed $0.06f_c'$.

14.8.3 — The design moment strength ϕM_n for combined flexure and axial loads at the midheight cross section shall be

$$\phi M_n \geq M_u \tag{14-3}$$

where

$$M_u = M_{ua} + P_u \Delta_u \tag{14-4}$$

M_{ua} is the moment at the midheight section of the wall due to factored loads, and Δ_u is

$$\Delta_u = \frac{5M_u \ell_c^2}{(0.75)48E_c I_{cr}} \tag{14-5}$$

M_u shall be obtained by iteration of deflections, or by direct calculation using Eq. (14-6)

$$M_u = \frac{M_{ua}}{1 - \dfrac{5P_u \ell_c^2}{(0.75)48E_c I_{cr}}} \tag{14-6}$$

where

$$I_{cr} = nA_{se}(d-c)^2 + \frac{\ell_w c^3}{3} \tag{14-7}$$

and

$$A_{se} = \frac{P_u + A_s f_y}{f_y} \tag{14-8}$$

14.8.4 — The maximum deflection Δ_s due to service loads, including $P\Delta$ effects, shall not exceed $\ell_c/150$. The midheight deflection Δ_s shall be determined by

$$\Delta_s = \frac{(5M)\ell_c^2}{48E_c I_e} \tag{14-9}$$

$$M = \frac{M_{sa}}{1 - \dfrac{5P_s \ell_c^2}{48E_c I_e}} \tag{14-10}$$

I_e shall be calculated using the procedure of 9.5.2.3, substituting M for M_a. I_{cr} shall be evaluated using Eq. (14-7).

CHAPTER 15 — FOOTINGS

CODE

15.0 — Notation

A_g = gross area of section, in.^{2}
d_p = diameter of pile at footing base
β = ratio of long side to short side of footing

15.1 — Scope

15.1.1 — Provisions of Chapter 15 shall apply for design of isolated footings and, where applicable, to combined footings and mats.

15.1.2 — Additional requirements for design of combined footings and mats are given in 15.10.

15.2 — Loads and reactions

15.2.1 — Footings shall be proportioned to resist the factored loads and induced reactions, in accordance with the appropriate design requirements of this code and as provided in Chapter 15.

15.2.2 — Base area of footing or number and arrangement of piles shall be determined from unfactored forces and moments transmitted by footing to soil or piles and permissible soil pressure or permissible pile capacity determined through principles of soil mechanics.

15.2.3 — For footings on piles, computations for moments and shears shall be permitted to be based on the assumption that the reaction from any pile is concentrated at pile center.

COMMENTARY

R15.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip. However, the values of constants used in equations are dependent on the units of the parameters. If the user chooses to use alternate units, the constants must be adjusted accordingly.

R15.1 — Scope

While the provisions of Chapter 15 apply to isolated footings supporting a single column or wall, most of the provisions are generally applicable to combined footings and mats supporting several columns or walls or a combination thereof.[15.1,15.2]

R15.2 — Loads and reactions

Footings are required to be proportioned to sustain the applied factored loads and induced reactions that include axial loads, moments, and shears that have to be resisted at the base of the footing or pile cap.

After the permissible soil pressure or the permissible pile capacity has been determined by principles of soil mechanics and in accord with the general building code, the size of the base area of a footing on soil or the number and arrangement of the piles should be established on the basis of unfactored (service) loads D, L, W, and E in whatever combination that governs the design.

Only the computed end moments that exist at the base of a column (or pedestal) need be transferred to the footing; the minimum moment requirement for slenderness considerations given in 10.12.3.2 need not be considered for transfer of forces and moments to footings.

In cases in which eccentric loads or moments are to be considered, the extreme soil pressure or pile reaction obtained from this loading should be within the permissible values. Similarly, the resultant reactions due to service loads combined with moments, shears, or both, caused by wind or earthquake loads should not exceed the increased values that may be permitted by the general building code.

To proportion a footing or pile cap for strength, the contact soil pressure or pile reaction due to the applied factored loading (see 8.1.1) should be determined. For a single concentrically loaded spread footing, the soil reaction q_s due to the factored loading is $q_s = U/A_f$, where U is the factored

CODE

COMMENTARY

concentric load to be resisted by the footing, and A_f is the base area of the footing as determined by the principles stated in 15.2.2 using the unfactored loads and the permissible soil pressure.

q_s is a calculated reaction to the factored loading used to produce the same required strength conditions regarding flexure, shear, and development of reinforcement in the footing or pile cap, as in any other member.

In the case of eccentric loading, load factors may cause eccentricities and reactions that are different from those obtained by unfactored loads.

When the alternate design method of Appendix I is used for design of footings, the soil bearing pressures or pile reactions are those caused by the service loads (without load factors). The permissible soil pressures or permissible pile reactions are equated directly with the applied service-load pressures or reactions to determine base area of footing or number and arrangement of piles. When lateral loads due to wind or earthquake are included in the governing load combination for footings, advantage may be taken of the 25 percent reduction in required strength in accordance with Section I.2.2.

15.3 — Footings supporting circular or regular polygon shaped columns or pedestals

For location of critical sections for moment, shear, and development of reinforcement in footings, it shall be permitted to treat circular or regular polygon shaped concrete columns or pedestals as square members with the same area.

15.4 — Moment in footings

15.4.1 — External moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over entire area of footing on one side of that vertical plane.

15.4.2 — Maximum factored moment for an isolated footing shall be computed as prescribed in 15.4.1 at critical sections located as follows:

(a) At face of column, pedestal, or wall, for footings supporting a concrete column, pedestal, or wall;

(b) Halfway between middle and edge of wall, for footings supporting a masonry wall;

(c) Halfway between face of column and edge of steel base plate, for footings supporting a column with steel base plate.

R15.4 — Moment in footings

CODE

15.4.3 — In one-way footings, and two-way square footings, reinforcement shall be distributed uniformly across entire width of footing.

15.4.4 — In two-way rectangular footings, reinforcement shall be distributed in accordance with 15.4.4.1 and 15.4.4.2.

15.4.4.1 — Reinforcement in long direction shall be distributed uniformly across entire width of footing.

15.4.4.2 — For reinforcement in short direction, a portion of the total reinforcement given by Eq. (15-1) shall be distributed uniformly over a band width (centered on centerline of column or pedestal) equal to the length of short side of footing. Remainder of reinforcement required in short direction shall be distributed uniformly outside center band width of footing

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta + 1)} \qquad (15\text{-}1)$$

15.5 — Shear in footings

15.5.1 — Shear strength of footings shall be in accordance with 11.12.

15.5.2 — Location of critical section for shear in accordance with Chapter 11 shall be measured from face of column, pedestal, or wall, for footings supporting a column, pedestal, or wall. For footings supporting a column or pedestal with steel base plates, the critical section shall be measured from location defined in 15.4.2(c).

COMMENTARY

R15.4.4 — In previous ACI Building Codes, the reinforcement in the short direction of rectangular footings should be distributed so that an area of steel given by Eq. (15-1) is provided in a band width equal to the length of the short side of the footing. The band width is centered about the column centerline.

The remaining reinforcement required in the short direction is to be distributed equally over the two segments outside the band width, one-half to each segment.

R15.5 — Shear in footings

R15.5.1 and R15.5.2 — The shear strength of footings are determined for the more severe condition of 11.12.1.1 or 11.12.1.2. The critical section for shear is measured from the face of supported member (column, pedestal, or wall), except for supported members on steel base plates.

Computation of shear requires that the soil reaction q_s be obtained from the factored loads and the design be in accordance with the appropriate equations of Chapter 11.

Where necessary, shear around individual piles may be investigated in accordance with 11.12.1.2. If shear perimeters overlap, the modified critical perimeter b_o should be taken as that portion of the smallest envelope of individual shear perimeter that will actually resist the critical shear for the group under consideration. One such situation is illustrated in Fig. R15.5.

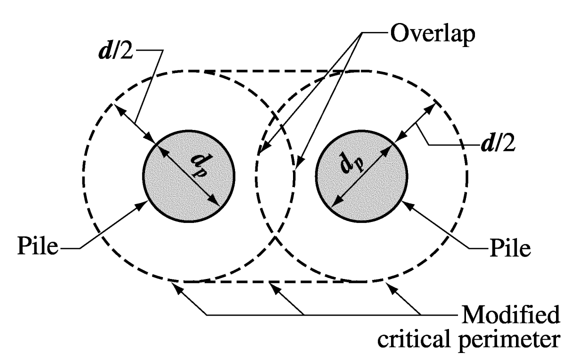

Fig. R15.5—Modified critical perimeter for shear with overlapping critical perimeters.

CODE

15.5.3 — Where the distance between the axis of any pile to the axis of the column is more than two times the distance between the top of the pile cap and the top of the pile, the pile cap shall satisfy 11.12 and 15.5.4. Other pile caps shall satisfy 11.12 or 15.5.4.

15.5.4 — Computation of shear on any section through a footing supported on piles shall be in accordance with 15.5.4.1, 15.5.4.2, and 15.5.4.3.

15.5.4.1 — Entire reaction from any pile whose center is located $d_p/2$ or more outside the section shall be considered as producing shear on that section.

15.5.4.2 — Reaction from any pile whose center is located $d_p/2$ or more inside the section shall be considered as producing no shear on that section.

15.5.4.3 — For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the section shall be based on straight-line interpolation between full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section.

15.6 — Development of reinforcement in footings

15.6.1 — Development of reinforcement in footings shall be in accordance with Chapter 12.

15.6.2 — Calculated tension or compression in reinforcement at each section shall be developed on each side of that section by embedment length, hook (tension only) or mechanical device, or a combination thereof.

15.6.3 — Critical sections for development of reinforcement shall be assumed at the same locations as defined in 15.4.2 for maximum factored moment, and at all other vertical planes where changes of section or reinforcement occur. See also 12.10.6.

15.7 — Minimum footing depth

Depth of footing above bottom reinforcement shall not be less than 6 in. for footings on soil, nor less than 12 in. for footings on piles.

15.8 — Transfer of force at base of column, wall, or reinforced pedestal

COMMENTARY

R15.5.3 — When piles are located inside the critical sections d or $d/2$ from face of column, for one-way or two-way shear, respectively, an upper limit on the shear strength at a section adjacent to the face of the column should be considered. The CRSI Handbook[15.3] offers guidance for this situation.

R15.8 — Transfer of force at base of column, wall, or reinforced pedestal

Section 15.8 provides the specific requirements for force transfer from a column, wall, or pedestal (supported member) to a pedestal or footing (supporting member). Force transfer should be by bearing on concrete (compressive force only) and by reinforcement (tensile or compressive force). Reinforcement may consist of extended longitudinal bars, dowels, anchor bolts, or suitable mechanical connectors.

CODE

COMMENTARY

The requirements of 15.8.1 apply to both cast-in-place construction and precast construction. Additional requirements for cast-in-place construction are given in 15.8.2. Section 15.8.3 gives additional requirements for precast construction.

15.8.1 — Forces and moments at base of column, wall, or pedestal shall be transferred to supporting pedestal or footing by bearing on concrete and by reinforcement, dowels, and mechanical connectors.

15.8.1.1 — Bearing on concrete at contact surface between supported and supporting member shall not exceed concrete bearing strength for either surface as given by 10.17.

R15.8.1.1 — Compressive force may be transmitted to a supporting pedestal or footing by bearing on concrete. For strength design, allowable bearing stress on the loaded area is equal to $0.85\phi f_c'$, if the loaded area is equal to the area on which it is supported.

In the common case of a column bearing on a footing larger than the column, bearing strength should be checked at the base of the column and the top of the footing. Strength in the lower part of the column should be checked because the column reinforcement cannot be considered effective near the column base because the force in the reinforcement is not developed for some distance above the base, unless dowels are provided, or the column reinforcement is extended into the footing. The unit bearing stress on the column will normally be $0.85\phi f_c'$. The permissible bearing strength on the footing may be increased in accordance with 10.17, and will usually be two times $0.85\phi f_c'$. The compressive force that exceeds that developed by the permissible bearing strength at the base of the column or at the top of the footing should be carried by dowels or extended longitudinal bars.

For the alternate design method of Appendix I, permissible bearing stresses are limited to 50 percent of the values in 10.17.

15.8.1.2 — Reinforcement, dowels, or mechanical connectors between supported and supporting members shall be adequate to transfer:

(a) All compressive force that exceeds concrete bearing strength of either member;

(b) Any computed tensile force across interface.

In addition, reinforcement, dowels, or mechanical connectors shall satisfy 15.8.2 or 15.8.3.

R15.8.1.2 — All tensile forces, whether created by uplift, moment, or other means, should be transferred to supporting pedestal or footing entirely by reinforcement or suitable mechanical connectors. Generally, mechanical connectors would be used only in precast construction.

15.8.1.3 — If calculated moments are transferred to supporting pedestal or footing, then reinforcement, dowels, or mechanical connectors shall be adequate to satisfy 12.17.

R15.8.1.3 — If computed moments are transferred from the column to the footing, the concrete in the compression zone of the column will generally be stressed to $0.85f_c'$ under factored load conditions and, as a result, all the reinforcement will generally have to be doweled into the footing.

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15.8.1.4 — Lateral forces shall be transferred to supporting pedestal or footing in accordance with shear-friction provisions of 11.7, or by other appropriate means.

15.8.2 — In cast-in-place construction, reinforcement required to satisfy 15.8.1 shall be provided either by extending longitudinal bars into supporting pedestal or footing, or by dowels.

15.8.2.1 — For cast-in-place columns and pedestals, area of reinforcement across interface shall be not less than 0.005 times gross area of supported member.

15.8.2.2 — For cast-in-place walls, area of reinforcement across interface shall be not less than minimum vertical reinforcement given in 14.3.2.

15.8.2.3 — At footings, it shall be permitted to lap splice No. 14 and No. 18 longitudinal bars, in compression only, with dowels to provide reinforcement required to satisfy 15.8.1. Dowels shall not be larger than No. 11 bar and shall extend into supported member a distance not less than the development length of No. 14 or No. 18 bars or the splice length of the dowels, whichever is greater, and into the footing a distance not less than the development length of the dowels.

15.8.2.4 — If a pinned or rocker connection is provided in cast-in-place construction, connection shall conform to 15.8.1 and 15.8.3.

15.8.3 — In precast construction, anchor bolts or suitable mechanical connectors shall be permitted for satisfying 15.8.1. Anchor bolts shall be designed in accordance with Appendix D.

15.8.3.1 — Connection between precast columns or pedestals and supporting members shall meet the requirements of 16.5.1.3(a).

15.8.3.2 — Connection between precast walls and supporting members shall meet the requirements of 16.5.1.3(b) and (c).

COMMENTARY

R15.8.1.4 — The shear-friction method given in 11.7 may be used to check for transfer of lateral forces to supporting pedestal or footing. Shear keys may be used, provided that the reinforcement crossing the joint satisfies 15.8.2.1, 15.8.3.1, and the shear-friction requirements of 11.7. In precast construction, resistance to lateral forces may be provided by shear-friction, shear keys, or mechanical devices.

R15.8.2.1 and R15.8.2.2 — A minimum amount of reinforcement is required between all supported and supporting members to ensure ductile behavior. The code does not require that all bars in a column be extended through and be anchored into a footing. Reinforcement with an area of 0.005 times the column area or an equal area of properly spliced dowels, however, is required to extend into the footing with proper anchorage. This reinforcement is required to provide a degree of structural integrity during the construction stage and during the life of the structure.

R15.8.2.3 — Lap splices of No. 14 and No. 18 longitudinal bars in compression only to dowels from a footing are specifically permitted in 15.8.2.3. The dowel bars should be No. 11 or smaller in size. The dowel lap splice length should meet the larger of the two criteria: (a) be able to transfer the stress in the No. 14 and No. 18 bars; and (b) fully develop the stress in the dowels as a splice.

This provision is an exception to 12.14.2.1, which prohibits lap splicing of No. 14 and No. 18 bars. This exception results from many years of successful experience with the lap splicing of these large column bars with footing dowels of the smaller size. The reason for the restriction on dowel bar size is recognition of the anchorage length problem of the large bars, and to allow use of the smaller size dowels. A similar exception is allowed for compression splices between different size bars in 12.16.2.

R15.8.3.1 and R15.8.3.2 — For cast-in-place columns, 15.8.2.1 requires a minimum area of reinforcement equal to $\mathbf{0.005}A_g$ across the column-footing interface to provide some degree of structural integrity. For precast columns, this requirement is expressed in terms of an equivalent tensile force that should be transferred. Thus, across the joint, $A_s f_y = \mathbf{200}A_g$ (see

CODE

15.8.3.3 — Anchor bolts and mechanical connectors shall be designed to reach their design strength before anchorage failure or failure of surrounding concrete. Anchor bolts shall be designed in accordance with Appendix D.

15.9 — Sloped or stepped footings

15.9.1 — In sloped or stepped footings, angle of slope or depth and location of steps shall be such that design requirements are satisfied at every section. (See also 12.10.6.)

15.9.2 — Sloped or stepped footings designed as a unit shall be constructed to assure action as a unit.

15.10 — Combined footings and mats

15.10.1 — Footings supporting more than one column, pedestal, or wall (combined footings or mats) shall be proportioned to resist the factored loads and induced reactions, in accordance with appropriate design requirements of this code.

15.10.2 — The direct design method of Chapter 13 shall not be used for design of combined footings and mats.

15.10.3 — Distribution of soil pressure under combined footings and mats shall be consistent with properties of the soil and the structure and with established principles of soil mechanics.

COMMENTARY

16.5.1.3(a)). The minimum tensile strength required for precast wall-to-footing connection (see 16.5.1.3(b)) is somewhat less than that required for columns, because an overload would be distributed laterally and a sudden failure would be less likely. Because the tensile strength values of 16.5.1.3 have been arbitrarily chosen, it is not necessary to include a strength reduction factor ϕ for these calculations.

R15.10 — Combined footings and mats

R15.10.1 — Any reasonable assumption with respect to the distribution of soil pressure or pile reactions can be used as long as it is consistent with the type of structure and the properties of the soil, and conforms with established principles of soil mechanics (see 15.1). Similarly, as prescribed in 15.2.2 for isolated footings, the base area or pile arrangement of combined footings and mats should be determined using the unfactored forces, moments, or both, transmitted by the footing to the soil, considering permissible soil pressures and pile reactions.

Design methods using factored loads and strength reduction factors ϕ can be applied to combined footings or mats, regardless of the soil pressure distribution.

Detailed recommendations for design of combined footings and mats are reported by ACI Committee 336.[15.1] See also Reference 15.2.

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COMMENTARY

Notes

CHAPTER 16 — PRECAST CONCRETE

CODE

16.0 — Notation

A_g = gross area of column, in.^{2}
ℓ = clear span, in.

16.1 — Scope

16.1.1 — All provisions of this code, not specifically excluded and not in conflict with the provisions of Chapter 16, shall apply to structures incorporating precast concrete structural members.

16.2 — General

16.2.1 — Design of precast members and connections shall include loading and restraint conditions from initial fabrication to end use in the structure, including form removal, storage, transportation, and erection.

16.2.2 — When precast members are incorporated into a structural system, the forces and deformations occurring in and adjacent to connections shall be included in the design.

16.2.3 — Tolerances for both precast members and interfacing members shall be specified. Design of precast members and connections shall include the effects of these tolerances.

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R16.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip.

R16.1 — Scope

R16.1.1 — See 2.1 for definition of precast concrete.

Design and construction requirements for precast concrete structural members differ in some respects from those for cast-in-place concrete structural members and these differences are addressed in this chapter. Where provisions for cast-in-place concrete apply equally to precast concrete, they have not been repeated. Similarly, items related to composite concrete in Chapter 17 and to prestressed concrete in Chapter 18 that apply to precast concrete are not restated.

More detailed recommendations concerning precast concrete are given in References 16.1 through 16.7. Tilt-up concrete construction is a form of precast concrete. It is recommended that Reference 16.8 be reviewed for tilt-up structures.

The provisions of this chapter are based on precast concrete members produced under plant-controlled conditions. Environmental structural elements precast at the job site will also qualify under this section if the control of form dimensions, placing of reinforcement, quality control of concrete, and curing procedures are equal to those specified in **"PCI Manual for Plant Quality Control."**

R16.2 — General

R16.2.1 — Stresses developed in precast members during the period from casting to final connection may be greater than the service load stresses. Handling procedures may cause undesirable deformations. Hence, care must be given to the methods of storing, transporting, and erecting precast members so that performance at service loads and strength under factored loads meet the code's requirements.

R16.2.2 — The structural behavior of precast members may differ substantially from that of similar members that are cast-in-place. Design of connections to minimize or transmit forces due to shrinkage, creep, temperature change, elastic deformation, differential settlement, wind, and earthquake require special consideration in precast construction.

R16.2.3 — Design of precast members and connections is particularly sensitive to tolerances on the dimensions of individual members and on their position in the structure. To prevent misunderstanding, the tolerances used in design should be specified in the contract documents. The designer

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COMMENTARY

may specify the tolerance standard assumed in design. It is especially important to specify any deviations from accepted standards.

The tolerances required by 7.5 are considered to be a minimum acceptable standard for reinforcement in precast concrete. The designer should refer to publications of the Precast/Prestressed Concrete Institute (PCI) (References 16.9, 16.10, and 16.11) for guidance on industry established standard product and erection tolerances. Added guidance is given in Reference 16.12.

16.2.4 — In addition to the requirements for drawings and specifications in 1.2, (a) and (b) shall be included in either the contract documents or shop drawings:

(a) Details of reinforcement, inserts, and lifting devices required to resist temporary loads from handling, storage, transportation, and erection;

(b) Required concrete strength at stated ages or stages of construction.

R16.2.4 — The additional requirements may be included in either contract documents or shop drawings depending on the assignment of responsibility for design.

16.3 — Distribution of forces among members

16.3.1 — Distribution of forces that are perpendicular to the plane of members shall be established by analysis or by test.

16.3.2 — Where the system behavior requires in-plane forces to be transferred between the members of a precast floor or wall system, 16.3.2.1 and 16.3.2.2 shall apply:

16.3.2.1 — In-plane force paths shall be continuous through both connections and members.

16.3.2.2 — Where tension forces occur, a continuous path of steel or steel reinforcement shall be provided.

R16.3 — Distribution of forces among members

R16.3.1 — Concentrated point and line loads can be distributed among members provided they have sufficient torsional stiffness and that shear can be transferred across joints. Torsionally stiff members such as hollow-core or solid slabs have more favorable load distribution properties than do torsionally flexible members such as double tees with thin flanges. The actual distribution of the load depends on many factors discussed in detail in References 16.13 through 16.19. Large openings can cause significant changes in distribution of forces.

R16.3.2 — In-plane forces result primarily from diaphragm action in floors and roofs, causing tension or compression in the chords and shear in the body of the diaphragm. A continuous path of steel, steel reinforcement, or both, using lap splices, mechanical or welded splices, or mechanical connectors, should be provided to carry the tension, whereas the shear and compression may be carried by the net concrete section. A continuous path of steel through a connection may include bolts, weld plates, headed studs or other steel devices. Tension forces in the connections are to be transferred to the primary reinforcement in the members.

In-plane forces in precast wall systems result primarily from diaphragm reactions and external lateral loads.

Connection details should provide for the forces and deformations due to shrinkage, creep, and thermal effects. Connection details may be selected to accommodate volume changes and rotations caused by temperature gradients and long-term deflections. When these effects are restrained, connections and members should be designed to provide adequate strength and ductility.

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16.4 — Member design

16.4.1 — In one-way precast floor and roof slabs and in one-way precast, prestressed wall panels, all not wider than 12 ft, and where members are not mechanically connected to cause restraint in the transverse direction, the shrinkage and temperature reinforcement requirements of 7.12 in the direction normal to the flexural reinforcement shall be permitted to be waived. This waiver shall not apply to members that require reinforcement to resist transverse flexural stresses.

16.4.2 — For precast, nonprestressed walls, the reinforcement shall be designed in accordance with the provisions of Chapter 10 or 14 except that the area of horizontal and vertical reinforcement shall each be not less than 0.001 times the gross cross-sectional area of the wall panel. Spacing of reinforcement shall not exceed 12 in.

16.5 — Structural integrity

16.5.1 — Except where the provisions of 16.5.2 govern, the minimum provisions of 16.5.1.1 through 16.5.1.4 for structural integrity shall apply to all precast concrete structures.

16.5.1.1 — Longitudinal and transverse ties required by 7.13.3 shall connect members to a lateral load resisting system.

COMMENTARY

R16.4 — Member design

R16.4.1 — For prestressed concrete members, not wider than 12 ft, such as hollow-core slabs, solid slabs, or slabs with closely spaced ribs, there is usually no need to provide transverse reinforcement to withstand shrinkage and temperature stresses in the short direction. This is generally true also for nonprestressed floor and roof slabs. The 12 ft width is less than that in which shrinkage and temperature stresses can build up to a magnitude requiring transverse reinforcement. In addition, much of the shrinkage occurs before the members are tied into the structure. Once in the final structure, the members are usually not as rigidly connected transversely as monolithic concrete, thus, the transverse restraint stresses due to both shrinkage and temperature change are significantly reduced.

The waiver does not apply, for example, to members such as single and double tees with thin, wide flanges.

For prestressed concrete members or slabs tied together with a grout or concrete topping, shrinkage and temperature nonprestressed reinforcement in the topping should be provided in accordance with the requirements of cast-in-place concrete.

R16.4.2 — This minimum area of wall reinforcement, instead of the minimum values in 14.3, has generally been used for many years and is recommended by PCI[16.4] and the Canadian building code.[16.20] The provisions for reduced minimum reinforcement and greater spacing recognize that precast wall panels have very little restraint at their edges during early stages of curing and, therefore, develop less shrinkage stress than comparable cast-in-place walls.

R16.5 — Structural integrity

R16.5.1 — The general provisions of 7.13.3 apply to all precast concrete structures. Sections 16.5.1 and 16.5.2 give minimum requirements to satisfy 7.13.3. It is not intended that these minimum requirements override other applicable provisions of the code for design of precast concrete structures.

The overall integrity of a structure can be substantially enhanced by minor changes in the amount, location, and detailing of member reinforcement and in the detailing of connection hardware.

R16.5.1.1 — Individual members may be connected into this lateral load resisting system by alternative methods. For example, a load-bearing spandrel could be connected to a diaphragm (part of the lateral load resisting system). Structural integrity could be achieved by connecting the spandrel into all or a portion of the deck members forming the diaphragm. Alternatively, the spandrel could be connected only to its supporting columns, which in turn must be connected to the diaphragm.

CODE

16.5.1.2 — Where precast elements form floor or roof diaphragms, the connections between diaphragm and those members being laterally supported shall have a nominal tensile strength capable of resisting not less than 300 lb per linear ft.

16.5.1.3 — Vertical tension tie requirements of 7.13.3 shall apply to all vertical structural members, except cladding, and shall be achieved by providing connections at horizontal joints in accordance with (a) through (c):

(a) Precast columns shall have a nominal strength in tension not less than $200A_g$ in lb. For columns with a larger cross section than required by consideration of loading, a reduced effective area A_g, based on cross section required but not less than one-half the total area, shall be permitted;

(b) Precast wall panels shall have a minimum of two ties per panel, with a nominal tensile strength not less than 10,000 lb per tie;

(c) When design forces result in no tension at the base, the ties required by 16.5.1.3(b) shall be permitted to be anchored into an appropriately reinforced concrete floor slab on grade.

16.5.1.4 — Connection details that rely solely on friction caused by gravity loads shall not be used.

16.5.2 — For precast concrete bearing wall structures three or more stories in height, the minimum provisions of 16.5.2.1 through 16.5.2.5 shall apply:

16.5.2.1 — Longitudinal and transverse ties shall be provided in floor and roof systems to provide a nominal strength of 1500 lb per foot of width or length. Ties shall be provided over interior wall supports and between members and exterior walls. Ties shall be positioned in or within 2 ft of the plane of the floor or roof system.

COMMENTARY

R16.5.1.2 — Diaphragms are typically provided as part of the lateral load resisting system. The ties prescribed in 16.5.1.2 are the minimum required to attach members to the floor or roof diaphragms. The tie force is equivalent to the service load value of 200 lb/ft given in the Uniform Building Code.

R16.5.1.3 — Base connections and connections at horizontal joints in precast columns and wall panels, including shear walls, are designed to transfer all design forces and moments. The minimum tie requirements of 16.5.1.3 are not additive to these design requirements. Industry practice is to place the wall ties symmetrically about the vertical centerline of the wall panel and within the outer quarters of the panel width, wherever possible.

R16.5.1.4 — In the event of damage to a beam, it is important that displacement of its supporting members be minimized, so that other members will not lose their load-carrying capacity. This is a situation that shows why connection details that rely solely on friction caused by gravity loads are not used. An exception could be heavy modular unit structures (one or more cells in cell-type structures) where resistance to overturning or sliding in any direction has a large factor of safety. Acceptance of such systems should be based on the provisions of 1.4.

R16.5.2 — The structural integrity minimum tie provisions for bearing wall structures, often called large panel structures, are intended to provide catenary hanger supports in case of loss of a bearing wall support, as shown by test.[16.21] Forces induced by loading, temperature change, creep, and wind or seismic action may require a larger amount of tie force. It is intended that the general precast concrete provisions of 16.5.1 apply to bearing wall structures less than three stories in height.

Minimum ties in structures three or more stories in height, in accordance with 16.5.2.1, 16.5.2.2, 16.5.2.3, 16.5.2.4, and 16.5.2.5, are required for structural integrity (Fig. R16.5.2). These provisions are based on PCI's recommendations for design of precast concrete bearing wall buildings.[16.22] Tie capacity is based on yield strength.

R16.5.2.1 — Longitudinal ties may project from slabs and be lap spliced, welded, or mechanically connected, or they may be embedded in grout joints, with sufficient length and cover to develop the required force. Bond length for unstressed prestressing steel should be sufficient to develop

CODE

16.5.2.2 — Longitudinal ties parallel to floor or roof slab spans shall be spaced not more than 10 ft on centers. Provisions shall be made to transfer forces around openings.

16.5.2.3 — Transverse ties perpendicular to floor or roof slab spans shall be spaced not greater than the bearing wall spacing.

16.5.2.4 — Ties around the perimeter of each floor and roof, within 4 ft of the edge, shall provide a nominal strength in tension not less than 16,000 lb.

16.5.2.5 — Vertical tension ties shall be provided in all walls and shall be continuous over the height of the building. They shall provide a nominal tensile strength not less than 3000 lb per horizontal foot of wall. Not less than two ties shall be provided for each precast panel.

16.6 — Connection and bearing design

16.6.1 — Forces shall be permitted to be transferred between members by grouted joints, shear keys, mechanical connectors, reinforcing steel connections, reinforced topping, or a combination of these means.

16.6.1.1 — The adequacy of connections to transfer forces between members shall be determined by analysis or by test. Where shear is the primary imposed loading, it shall be permitted to use the provisions of 11.7 as applicable.

16.6.1.2 — When designing a connection using materials with different structural properties, their relative stiffnesses, strengths, and ductilities shall be considered.

16.6.2 — Bearing for precast floor and roof members on simple supports shall satisfy the following:

COMMENTARY

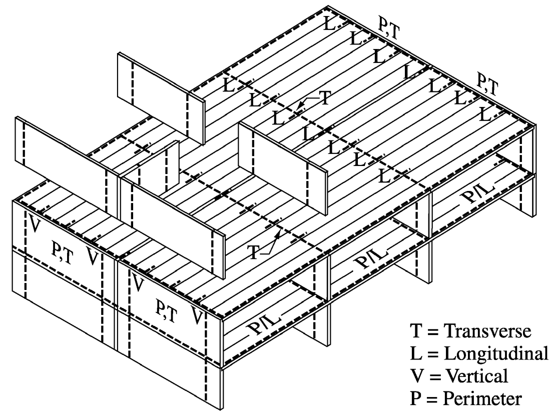

Fig. R16.5.2—Typical arrangement of tensile ties in large panel structures.

the yield strength.[16.23] It is not uncommon to have ties positioned in the walls reasonably close to the plane of the floor or roof system.

R16.5.2.3 — Transverse ties may be uniformly spaced either encased in the panels or in a topping, or they may be concentrated at the transverse bearing walls.

R16.5.2.4 —The perimeter tie requirements need not be additive with the longitudinal and transverse tie requirements.

R16.6 — Connection and bearing design

R16.6.1 — The code permits a variety of methods for connecting members. These are intended for transfer of forces both in-plane and perpendicular to the plane of the members.

R16.6.1.2 — Various components in a connection (for example, bolts, welds, plates, inserts, etc.) have different properties that can affect the overall behavior of the connection.

CODE

16.6.2.1—The allowable bearing stress at the contact surface between supported and supporting members and between any intermediate bearing elements shall not exceed the bearing strength for either surface and the bearing element. Concrete bearing strength shall be as given in 10.17.

16.6.2.2 — Unless shown by test or analysis that performance will not be impaired, (a) and (b) shall be met:

(a) Each member and its supporting system shall have design dimensions selected so that, after consideration of tolerances, the distance from the edge of the support to the end of the precast member in the direction of the span is at least 1/180 of the clear span ℓ, but not less than:

For solid or hollow-core slabs............................ 2 in.
For beams or stemmed members .... 3 in.

(b) Bearing pads at unarmored edges shall be set back a minimum of 1/2 in. from the face of the support, or at least the chamfer dimension at chamfered edges.

16.6.2.3 — The requirements of 12.11.1 shall not apply to the positive bending moment reinforcement for statically determinate precast members, but at least one-third of such reinforcement shall extend to the center of the bearing length, taking into account permitted tolerances in 7.5.2.2 and 16.2.3.

16.7 — Items embedded after concrete placement

16.7.1 — When approved by the licensed design professional, embedded items (such as dowels or inserts) that either protrude from the concrete or remain exposed for inspection shall be permitted to be embedded while the concrete is in a plastic state provided 16.7.1.1, 16.7.1.2, and 16.7.1.3 are met.

16.7.1.1 — Embedded items are not required to be hooked or tied to reinforcement within the concrete.

16.7.1.2 — Embedded items are maintained in the correct position while the concrete remains plastic.

16.7.1.3 — The concrete is properly consolidated around the embedded item.

COMMENTARY

R16.6.2.1 — When tensile forces occur in the plane of the bearing, it may be desirable to reduce the allowable bearing stress, provide confinement reinforcement, or both. Guidelines are provided in Reference 16.4.

R16.6.2.2 — This section differentiates between bearing length and length of the end of a precast member over the support (Fig. R16.6.2). Bearing pads distribute concentrated loads and reactions over the bearing area, and allow limited horizontal and rotational movements for stress relief. To prevent spalling under heavily loaded bearing areas, bearing pads should not extend to the edge of the support unless the edge is armored. Edges can be armored with anchored steel plates or angles. Section 11.9.7 gives requirements for bearing on brackets or corbels.

R16.6.2.3 — It is unnecessary to develop positive bending moment reinforcement beyond the ends of the precast element if the system is statically determinate. Tolerances need to be considered to avoid bearing on plain concrete where reinforcement has been discontinued.

R16.7 — Items embedded after concrete placement

R16.7.1 — Section 16.7.1 is an exception to the provisions of 7.5.1. Many precast products are manufactured in such a way that it is difficult, if not impossible, to position reinforcement that protrudes from the concrete before the concrete is placed. Experience has shown that such items as ties for horizontal shear and inserts can be placed while the concrete is plastic, if proper precautions are taken. This exception is not applicable to reinforcement that is completely embedded, or to embedded items that must be hooked or tied to embedded reinforcement.

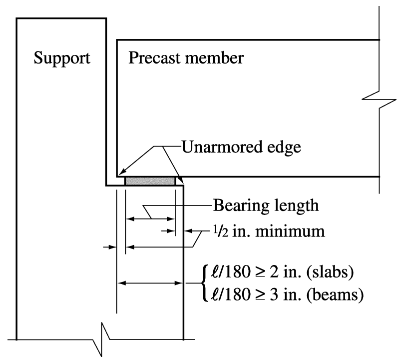

Fig. R16.6.2—Bearing length versus length of member on support.

CODE

16.8 — Marking and identification

16.8.1 — Each precast member shall be marked to indicate its location and orientation in the structure and date of manufacture.

16.8.2 — Identification marks shall correspond to placing drawings.

16.9 — Handling

16.9.1 — Member design shall consider forces and distortions during curing, stripping, storage, transportation, and erection so that precast members are not overstressed or otherwise damaged.

16.9.2 — Precast members and structures shall be adequately supported and braced during erection to ensure proper alignment and structural integrity until permanent connections are completed.

16.10 — Strength evaluation of precast construction

16.10.1 — A precast element to be made composite with cast-in-place concrete shall be permitted to be tested in flexure as a precast element alone in accordance with 16.10.1.1 and 16.10.1.2.

16.10.1.1 — Test loads shall be applied only when calculations indicate the isolated precast element will not be critical in compression or buckling.

16.10.1.2 — The test load shall be that load which, when applied to the precast member alone, induces the same total force in the tension reinforcement as would be induced by loading the composite member with the test load required by 20.3.2.

16.10.2 — The provisions of 20.5 shall be the basis for acceptance or rejection of the precast element.

COMMENTARY

R16.9 — Handling

R16.9.1 — The code requires acceptable performance at service loads and adequate strength under factored loads. Handling loads, however, should not produce permanent stresses, strains, cracking, or deflections inconsistent with the provisions of the code. A precast member should not be rejected for minor cracking or spalling where strength and durability are not affected. Guidance on assessing cracks in precast members is given in two Precast/Prestressed Concrete Institute reports on fabrication and shipment cracks.[16.24,16.25]

R16.9.2 — All temporary erection connections, bracing, and shoring are shown on contract or erection drawings.

R16.10 — Strength evaluation of precast construction

The strength evaluation procedures of Chapter 20 are applicable to precast members.

CODE

COMMENTARY

Notes

CHAPTER 17 — COMPOSITE CONCRETE FLEXURAL MEMBERS

CODE

17.0 — Notation

A_c = area of contact surface being investigated for horizontal shear, in.^{2}
A_v = area of ties within a distance s, in.^{2}
b_v = width of cross section at contact surface being investigated for horizontal shear, in.
d = distance from extreme compression fiber to centroid of tension reinforcement for entire composite section, in.
f_y = specified yield strength of nonprestressed reinforcement, psi
h = overall thickness of composite member, in.
s = spacing of ties measured along the longitudinal axis of the member, in.
V_{nh} = nominal horizontal shear strength, lb
V_u = factored shear force at section, lb
λ = correction factor related to unit weight of concrete
ρ_v = ratio of tie reinforcement area to area of contact surface
= A_v/b_vs
ϕ = strength reduction factor. See 9.3

17.1 — Scope

17.1.1 — Provisions of Chapter 17 shall apply for design of composite concrete flexural members defined as precast concrete, cast-in-place concrete elements, or both, constructed in separate placements but so interconnected that all elements respond to loads as a unit.

17.1.2 — All provisions of this code shall apply to composite concrete flexural members, except as specifically modified in Chapter 17.

17.2 — General

17.2.1 — The use of an entire composite member or portions thereof for resisting shear and moment shall be permitted.

17.2.2 — Individual elements shall be investigated for all critical stages of loading.

17.2.3 — If the specified strength, unit weight, or other properties of the various elements are different, properties of the individual elements or the most critical values shall be used in design.

COMMENTARY

R17.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip. However, the values of constants used in equations are dependent on the units of the parameters. If the user chooses to use alternate units, the constants must be adjusted accordingly.

R17.1 — Scope

R17.1.1 — The scope of Chapter 17 is intended to include all types of composite concrete flexural members. In some cases with fully cast-in-place concrete, it may be necessary to design the interface of consecutive placements of concrete as required for composite members. Composite structural steel-concrete members are not covered in this chapter, because design provisions for such composite members are covered in Reference 17.1.

R17.2 — General

CODE

17.2.4 — In strength computations of composite members, no distinction shall be made between shored and unshored members.

17.2.5 — All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.

17.2.6 — Reinforcement shall be provided as required to minimize cracking and to prevent separation of individual elements of composite members.

17.2.7 — Composite members shall meet requirements for control of deflections in accordance with 9.5.5.

17.3 — Shoring

When used, shoring shall not be removed until supported elements have developed design properties required to support all loads and limit deflections and cracking at time of shoring removal.

17.4 — Vertical shear strength

17.4.1 — When an entire composite member is assumed to resist vertical shear, design shall be in accordance with requirements of Chapter 11 as for a monolithically cast member of the same cross-sectional shape.

17.4.2 — Shear reinforcement shall be fully anchored into interconnected elements in accordance with 12.13.

17.4.3 — Extended and anchored shear reinforcement shall be permitted to be included as ties for horizontal shear.

COMMENTARY

R17.2.4 — Tests have indicated that the strength of a composite member is the same whether or not the first element cast is shored during casting and curing of the second element.

R17.2.6 — The extent of cracking is dependent on such factors as environment, aesthetics, and occupancy. In addition, composite action should not be impaired.

Where members are exposed to normal environmental exposures, the reinforcement stress levels and spacing requirements of 10.6.4.1 should be satisfied. Where members are exposed to severe environmental exposures, the reinforcement stress levels and spacings should comply with requirements of 10.6.4.2. The z quantity for members exposed to other atmospheres should meet the requirements of 10.6.4.

R17.2.7 — The premature loading of precast elements can cause excessive creep and shrinkage deflections. This is especially so at early ages when the moisture content is high and the strength low.

The transfer of shear by direct bond is important if excessive deflection from slippage is to be prevented. A shear key is an added mechanical factor of safety but it cannot operate until slippage occurs.

R17.3 — Shoring

The provisions of 9.5.5 cover the requirements pertaining to deflections of shored and unshored members.

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17.5 — Horizontal shear strength

17.5.1 — In a composite member, full transfer of horizontal shear forces shall be ensured at contact surfaces of interconnected elements.

17.5.2 — Unless calculated in accordance with 17.5.3, design of cross sections subject to horizontal shear shall be based on

$$V_u \leq \phi V_{nh} \tag{17-1}$$

where V_u is factored shear force at section considered and V_{nh} is nominal horizontal shear strength in accordance with 17.5.2.1 and 17.5.2.5.

17.5.2.1 — When contact surfaces are clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not be taken greater than $80 b_v d$, in lb.

17.5.2.2 — When minimum ties are provided in accordance with 17.6, and contact surfaces are clean and free of laitance, but not intentionally roughened, shear strength V_{nh} shall not be taken greater than $80 b_v d$, in lb.

17.5.2.3 — When ties are provided in accordance with 17.6, and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in., shear strength V_{nh} shall be taken equal to $(260 + 0.6\rho_v f_y)\lambda b_v d$, in lb, but not greater than $500 b_v d$, in lb. Values for λ in 11.7.4.3 shall apply.

17.5.2.4 — When factored shear force V_u at section considered exceeds $\phi(500 b_v d)$, design for horizontal shear shall be in accordance with 11.7.4.

COMMENTARY

R17.5 — Horizontal shear strength

R17.5.1 — Full transfer of horizontal shear between segments of composite members should be ensured by horizontal shear strength at contact surfaces or properly anchored ties, or both.

R17.5.2 — The nominal horizontal shear strengths V_{nh} apply when the design is based on the load factors and ϕ-factors of Chapter 9.

When the alternate design method of Appendix I is used for design of composite members, V_u is the shear due to service loads, and 55 percent of the values given in 17.5.2 are applicable. See I.7.3. Also, when gravity loads are combined with lateral loads due to wind or earthquake in the governing load combination for horizontal shear, advantage may be taken of the 25 percent reduction in required strength in accordance with I.2.2.

In reviewing composite concrete flexural members for handling and construction loads, V_u may be replaced by the handling service load shear in Eq. (17-1). The handling load horizontal shear should be compared with a nominal horizontal shear strength value of $0.55V_{nh}$ (as provided in Appendix I for the alternate design method) to ensure that an adequate factor of safety results for handling and construction loads.

Prestressed members used in composite construction may have variations in depth of tension reinforcement along member length due to draped or depressed tendons. Because of this variation, the definition of d used in Chapter 11 for determination of vertical shear strength is also appropriate when determining horizontal shear strength.

R17.5.2.3 — The permitted horizontal shear strengths and the requirement of 1/4 in. amplitude for intentional roughness are based on tests discussed in References 17.2 through 17.4.

CODE

17.5.2.5 — When determining nominal horizontal shear strength over prestressed concrete elements, d shall be as defined or $0.8h$, whichever is greater.

17.5.3 — As an alternative to 17.5.2, horizontal shear shall be permitted to be determined by computing the actual change in compressive or tensile force in any segment, and provisions shall be made to transfer that force as horizontal shear to the supporting element. The factored horizontal shear force shall not exceed horizontal shear strength ϕV_{nh} as given in 17.5.2.1 through 17.5.2.4, where area of contact surface A_c shall be substituted for $b_v d$.

17.5.3.1 — When ties provided to resist horizontal shear are designed to satisfy 17.5.3, the tie area to tie spacing ratio along the member shall approximately reflect the distribution of shear forces in the member.

17.5.4 — When tension exists across any contact surface between interconnected elements, shear transfer by contact shall be permitted only when minimum ties are provided in accordance with 17.6.

17.6 — Ties for horizontal shear

17.6.1 — When ties are provided to transfer horizontal shear, tie area shall not be less than that required by 11.5.5.3, and tie spacing shall not exceed four times the least dimension of supported element, nor exceed 24 in.

17.6.2 — Ties for horizontal shear shall consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric (plain or deformed).

17.6.3 — All ties shall be fully anchored into interconnected elements in accordance with 12.13.

COMMENTARY

R17.5.3.1 — The distribution of horizontal shear stresses along the contact surface in a composite member will reflect the distribution of shear along the member. Horizontal shear failure will initiate where the horizontal shear stress is a maximum and will spread to regions of lower stress. Because the slip at peak horizontal shear resistance is small for a concrete to concrete contact surface, longitudinal redistribution of horizontal shear resistance is very limited. The spacing of the ties along the contact surface should, therefore, be such as to provide horizontal shear resistance distributed approximately as the shear acting on the member is distributed.

R17.5.4 — Proper anchorage of ties extending across interfaces is required to maintain contact of the interfaces.

R17.6 — Ties for horizontal shear

The minimum areas and maximum spacings are based on test data given in References 17.2 through 17.6.

CHAPTER 18 — PRESTRESSED CONCRETE

CODE

18.0 — Notation

A = area of that part of cross section between flexural tension face and center of gravity of gross section, in.^{2}

A_{cf} = larger gross cross-sectional area of the slab-beam strips of the two orthogonal equivalent frames intersecting at a column of a two-way slab, in.^{2}

A_{ps} = area of prestressed reinforcement in tension zone, in.^{2}

A_s = area of nonprestressed tension reinforcement, in.^{2}

A'_s = area of compression reinforcement, in.^{2}

b = width of compression face of member, in.

c_c = clear cover from the nearest surface in tension to the surface of the flexural tension steel, in.

d = distance from extreme compression fiber to centroid of nonprestressed tension reinforcement, in.

d' = distance from extreme compression fiber to centroid of compression reinforcement, in.

d_p = distance from extreme compression fiber to centroid of prestressed reinforcement, in.

D = dead loads, or related internal moments and forces

e = base of Napierian logarithms

f'_c = specified compressive strength of concrete, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi

f'_{ci} = compressive strength of concrete at time of initial prestress, psi

$\sqrt{f'_{ci}}$ = square root of compressive strength of concrete at time of initial prestress, psi

f_{dc} = decompression stress. Stress in the prestressing steel when stress is zero in the concrete at the same level as the centroid of the tendons, ksi

f_{pc} = average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses), psi

f_{ps} = stress in prestressed reinforcement at nominal strength, psi

COMMENTARY

R18.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip. Nevertheless, care should be taken to properly account for the effect of differing units on constants given in equations that may include factors to account for the units of defined variables.

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f_{pu} = specified tensile strength of prestressing steel, psi

f_{py} = specified yield strength of prestressing steel, psi

f_r = modulus of rupture of concrete, psi

f_{se} = effective stress in prestressed reinforcement (after allowance for all prestress losses), psi

f_t = extreme fiber stress in tension in the precompressed tensile zone, computed using gross section properties, psi

f_y = specified yield strength of nonprestressed reinforcement, psi

h = overall thickness of member, in.

K = wobble friction coefficient per foot of tendon

ℓ_x = length of prestressing steel element from jacking end to any point x, ft. See Eq. (18-1) and (18-2)

L = live loads, or related internal moments and forces

n = number of monostrand anchorage devices in a group

N_c = tensile force in concrete due to unfactored dead load plus live load $(D + L)$, lb

P_s = prestressing force at jacking end, lb

P_{su} = factored prestressing force at the anchorage device, lb

P_x = prestressing force at any point x, lb

s = center-to-center spacing of flexural tension steel near the extreme tension face, in. Where there is only one bar or tendon near the extreme tension face, s is the width of extreme tension face, in.

α = total angular change of prestressing tendon profile in radians from tendon jacking end to any point x

β_1 = factor defined in 10.2.7.3

Δf_{ps} = stress in prestressing steel at service loads less decompression stress, ksi

γ_p = factor for type of prestressing steel

= 0.55 for f_{py}/f_{pu} not less than 0.80

= 0.40 for f_{py}/f_{pu} not less than 0.85

COMMENTARY

The factored pressing force P_{su} is the product of the load factor (1.2 from Section 9.2.5) and the maximum prestressing force allowed. Under 18.5.1 this is usually overstressing to $0.94f_{py}$ but not greater than $0.80f_{pu}$, which is permitted for short periods of time

$$P_{su} = (1.2)(0.80)\, f_{pu}A_{ps}$$

$$= 0.96f_{pu}A_{ps}$$

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= 0.28 for f_{py}/f_{pu} not less than 0.90

λ = correction factor related to unit weight of concrete (See 11.7.4.3)

μ = curvature friction coefficient

ρ = ratio of nonprestressed tension reinforcement

= A_s/bd

ρ' = ratio of compression reinforcement

= A'_s/bd

ρ_p = ratio of prestressed reinforcement

= A_{ps}/bd_p

ϕ = strength reduction factor. See 9.3

ω = $\rho f_y/f_c'$

ω' = $\rho' f_y/f_c'$

ω_p = $\rho_p f_{ps}/f_c'$

$\omega_w, \omega_{pw}, \omega'_w$ = reinforcement indices for flanged sections computed as for ω, ω_p, and ω' except that b shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only

18.1 — Scope

18.1.1 — Provisions of Chapter 18 shall apply to members prestressed with wire, strands, or bars conforming to provisions for prestressing steel in 3.5.5. Circular and other prestressed concrete internal tendon tanks are covered in this chapter. Circular prestressed concrete tanks circumferentially wrapped with wire or strand are covered in Appendix G.

18.1.2 — All provisions of this code not specifically excluded, and not in conflict with provisions of Chapter 18, shall apply to prestressed concrete.

18.1.3 — The following provisions of this code shall not apply to prestressed concrete, except as specifically noted: Sections 7.6.5, 8.4, 8.10.2, 8.10.3, 8.10.4, 8.11, 10.3.2, 10.3.3, 10.5, 10.6, 10.9.1, and 10.9.2; Chapter 13; and Sections 14.3, 14.5, and 14.6.

COMMENTARY

R18.1 — Scope

R18.1.1 — The provisions of Chapter 18 are for internal tendon stressing of structural members such as two-way flat plate mat foundations, horizontally or vertically prestressed walls of water, wastewater, and other liquid-retaining tanks (including circular, rectangular, and egg-shaped) and two-way prestressed roofs over water, wastewater, and other liquid-retaining structures. Many of the provisions, however, may be applied to other types of construction such as pressure vessels, pavements, and pipes. Application of the provisions is left to the judgment of the engineer in cases not specifically cited in the code.

Button-headed wire tendons are rarely used. If used, parallel wire tendons in ducts require special considerations, such as spacers between wires, to ensure proper grout encapsulation for corrosion protection.

R18.1.3 — Some sections of the code are excluded from use in the design of prestressed concrete for specific reasons. The following discussion provides explanation for such exclusions:

Section 7.6.5 — Section 7.6.5 of the code is excluded from application to prestressed concrete because the requirements

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for bonded reinforcement and unbonded tendons for cast-in-place members are provided in 18.9 and 18.12, respectively.

Section 8.4 — Section 8.4 of the code is excluded because moment redistribution for prestressed concrete is provided in 18.10.4.

Sections 8.10.2, 8.10.3, and 8.10.4 — The empirical provisions of 8.10.2, 8.10.3, and 8.10.4 for T-beams were developed for nonprestressed reinforced concrete and, if applied to prestressed concrete, would exclude many standard prestressed products in satisfactory use today. Hence, proof by experience permits variations.

By excluding 8.10.2, 8.10.3, and 8.10.4, no special requirements for prestressed concrete T-beams appear in the code. Instead, the determination of an effective width of flange is left to the experience and judgment of the engineer. Where possible, the flange widths in 8.10.2, 8.10.3, and 8.10.4 should be used unless experience has proven that variations are safe and satisfactory. It is not necessarily conservative in elastic analysis and design considerations to use the maximum flange width as permitted in 8.10.2.

Sections 8.10.1 and 8.10.5 provide general requirements for T-beams that are also applicable to prestressed concrete members. The spacing limitations for slab reinforcement are based on flange thickness, which for tapered flanges can be taken as the average thickness.

Section 8.11 — The empirical limits established for conventionally reinforced concrete joist floors are based on successful past performance of joist construction using standard joist forming systems. See R8.11. For prestressed joist construction, experience and judgment should be used. The provisions of 8.11 may be used as a guide.

Sections 10.3.2, 10.3.3, 10.5, 10.9.1, and 10.9.2 — For prestressed concrete, the limitations on reinforcement given in 10.3.2, 10.3.3, 10.5, 10.9.1, and 10.9.2 are replaced by those in 18.8, 18.9, and 18.11.2.

Section 10.6 — When originally prepared, the provisions of 10.6 for distribution of flexural reinforcement were not intended for prestressed concrete members. The behavior of a prestressed member is considerably different from that of a nonprestressed member. Experience and judgment should be used for proper distribution of reinforcement in a prestressed member.

Chapter 13 — The design of prestressed concrete slabs requires recognition of induced secondary moments. Also, volume changes due to the prestressing force can create additional loads on the structure that are not adequately covered in Chapter 13. Because of these unique properties associated with prestressing, many of the design procedures of Chapter 13 are not appropriate for prestressed concrete structures, and are replaced by the provisions of 18.12.

Sections 14.5 and 14.6 — The requirements for wall design in 14.5 and 14.6 are largely empirical, utilizing considerations not intended to apply to prestressed concrete.

18.1.4 — The environmental durability factor provisions of Section 9.2.6 of this code shall not apply to prestressed concrete except for the provisions of 9.2.6.4 and 9.2.6.5 for shear design loads.

R18.1.4 — The environmental durability factor was developed for nonprestressed reinforced concrete to be comparable to the successful durability and long-term performance of working stress design using lower allowable stresses in the reinforcement. The requirements of Sections 18.3.3 and 18.4.2 of this code provide the desired durability and serviceability characteristics for prestressed concrete. In prestressed concrete, the environmental durability factor is applied to the determination of shear loads only because the shear is not carried by the prestressed reinforcement.

18.2 — General

18.2.1 — Prestressed members shall meet the strength requirements specified in this code.

18.2.2 — Design of prestressed members shall be based on strength and on behavior at service conditions at all load stages that will be critical during the life of the structure from the time prestress is first applied.

R18.2 — General

R18.2.1 and R18.2.2 — The design investigation should include all stages that may be significant. The three major stages are: (1) jacking stage, or prestress transfer stage—when the tensile force in the prestressing tendons is transferred to the concrete and stress levels may be high relative to concrete strength; (2) service load stage—after long-term volume changes have occurred; and (3) the factored load stage—when the strength of the member is checked. There may be other load stages that require investigation. For example, if the cracking load is significant, this load stage may require study, or the handling and transporting stage may be critical.

From the standpoint of satisfactory behavior, the two stages of most importance are those for service load and factored load.

Service load stage refers to the loads defined in the general building code (without load factors), such as live load and dead load, while the factored load stage refers to loads multiplied by the appropriate load factors.

Section 18.3.2 provides assumptions that may be used for investigation at service loads and after transfer of the prestressing force.

18.2.3 — Stress concentrations due to prestressing shall be considered in design.

18.2.4 — Provisions shall be made for effects on adjoining construction of elastic and plastic deformations, deflections, changes in length, and rotations due to prestressing. Effects of temperature and shrinkage shall also be included.

18.2.5 — The possibility of buckling in a member between points where there is intermittent contact between the prestressing steel and an oversized duct, and buckling in thin webs and flanges shall be considered.

R18.2.5 — Section 18.2.5 refers to the type of post-tensioning where the prestressing steel makes intermittent contact with an oversized duct. Precautions should be taken to prevent buckling of such members.

If the prestressing steel is in complete contact with the member being prestressed, or is unbonded with the sheathing not excessively larger than the prestressing steel,

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it is not possible to buckle the member under the prestressing force being introduced.

18.2.6 — In computing section properties before bonding of prestressing steel, effect of loss of area due to open ducts shall be considered.

R18.2.6 — In considering the area of the open ducts, the critical sections should include those that have coupler sheaths that may be of a larger size than the duct containing the prestressing steel. Also, in some instances, the trumpet or transition piece from the conduit to the anchorage may be of such a size as to create a critical section. If the effect of the open duct area on design is deemed negligible, section properties may be based on total area.

In post-tensioned members after grouting and in pretensioned members, section properties may be based on effective sections using transformed areas of bonded prestressing steel and nonprestressed reinforcement gross sections, or net sections.

18.2.7 — When circumferential prestressing tendons are used in circular tanks, they shall be bonded tendons.

R18.2.7 — When unbonded tendons are used in other wall applications, consideration should be given to increasing the reinforcement requirement of 18.9. This additional reinforcement is recommended to control cracking at service levels.

18.3 — Design assumptions

18.3.1 — Strength design of prestressed members for flexure and axial loads shall be based on assumptions given in 10.2, except that 10.2.4 shall apply only to reinforcement conforming to 3.5.3. Design of prestressed concrete liquid-containment vessels shall be based on elastic analysis methods for external loads and prestressing.

18.3.2 — For investigation of stresses at transfer of prestress, at service loads, and at cracking loads, elastic theory shall be used with the assumptions of 18.3.2.1 and 18.3.2.2.

18.3.2.1 — Strains vary linearly with depth through entire load range.

R18.3 — Design assumptions

R18.3.1 — Strength design methods are of doubtful relevance to the design of prestressed concrete liquid-containment vessels. Serviceability of the structure under the design loads is of paramount importance. A working stress approach is therefore required. Provided that the allowable stresses specified herein are not exceeded, strength requirements should also be satisfied.

The design should take into account the effects of all loads and prestressing forces during and after tensioning, and the conditions of edge restraint at wall junctions with the floor and roof. Stresses should not exceed the allowable service stresses described in 18.4.2 and 18.5.1. The designer should also consider the effects of all loads and combinations of loads, including the stresses induced by temperature and moisture gradients. Analyses for internal stresses due to temperature and moisture gradients may be based on inelastic methods that consider the reduction of stress due to creep and surface cracking. The design should also meet the strength requirements of Chapters 9 and 10. The environmental durability factor is not required for prestressed design when the working stress provisions of this chapter are used.

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18.3.2.2 — At cracked sections, concrete resists no tension.

18.3.3 — Prestressed flexural members shall be classified as Class U or Class T based on the computed extreme fiber stress f_t at service loads in the precompressed tensile zone, as follows:

(a) Class U: $f_t \leq 7.5\sqrt{f'_c}$;

(b) Class T: $7.5\sqrt{f'_c} < f_t \leq 12\sqrt{f'_c}$.

Prestressed two-way slab system shall be designed as Class U.

18.3.4 — For prestressed flexural members, stresses at service loads shall be permitted to be calculated using the uncracked section.

COMMENTARY

R18.3.3 — This section defines two classes of behavior of prestressed flexural members. Class U members are assumed to behave as uncracked members. The behavior of Class T members is assumed to be in transition between uncracked and cracked. The serviceability requirements for each class are summarized in Table R18.3.3. For comparison, Table R18.3.3 also shows corresponding requirements for nonprestressed members. These classes apply to both bonded and unbonded prestressed flexural members, but prestressed two-way slab systems are required to be designed as Class U. Class C members as defined in ACI 318-02 are not used in environmental concrete structures.

The precompressed tensile zone is that portion of the member cross section in which flexural tension occurs under dead and live loads. Prestressed concrete is usually designed so that the prestress force introduces compression into this zone, thus effectively reducing the magnitude of the tensile stress.

TABLE R18.3.3 — SERVICEABILITY DESIGN REQUIREMENTS

	Prestressed	
	Class U	Class T
Assumed behavior	Uncracked	Transition between uncracked and cracked
Section properties for stress calculation at service loads	Gross Section 18.3.4	Gross Section 18.3.4
Allowable stress at transfer	18.4.1	18.4.1
Allowable compressive stress based on uncracked section properties	18.4.2	18.4.2
Tensile stress at service loads 18.3.3	$\leq 7.5\sqrt{f'_c}$	$\leq 7.5\sqrt{f'_c} < f_t < 12\sqrt{f'_c}$
Deflection calculation basis	9.5.4.1 Gross section	9.5.4.2 Cracked section, bilinear
Crack control	No requirement	No requirement
Computation of Δf_{ps} or f_s for crack control	—	—
Side skin reinforcement	No requirement	No requirement

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COMMENTARY

18.3.5 — Deflections of prestressed flexural members shall be calculated in accordance with 9.5.4.

18.3.6 — The minimum compressive stress at the centroid of an element in the direction of the prestressing under maximum service loads after all losses shall be 125 psi for liquid-containing elements.

R18.3.6 — When the liquid-tightness of prestressed concrete must be assured, a minimum precompression at the centroid of the cross section of 125 psi has been found to be necessary. In thinner wall sections, generally 16 in. or less, an increased level of prestressing of 200 psi is commonly used. Experience with this limit comes from the design of walls for circular and rectangular tanks.

The designer's attention is drawn to the need for details that permit elastic shortening of the concrete due to prestress. When restraint cannot be avoided, additional nonprestressed bonded reinforcement should be provided to control cracking. For example, at the base of post-tensioned walls restrained by footings at the time of prestressing, the addition of 1 percent horizontal nonprestressed bonded reinforcement in the bottom 3 ft of the wall, is effective in controlling vertical cracks that may otherwise occur at the base of the wall.

18.3.7 — For empty open-top circular tanks, the minimum circumferential compressive stress at the centroid of an element after all losses shall be 400 psi.

R18.3.7 — Experience has shown that vertical cracks often develop at the top of open-top circular tanks, due to the discontinuity of the shell and as a result of significant temperature differentials and shrinkage on the exposed wall areas generally above the maximum operating liquid level. The additional residual compression at the top of the wall is effective in eliminating the vertical cracks that are formed as a result of these unusual boundary and environmental conditions.

18.4 — Serviceability requirements—Flexural members

R18.4 — Serviceability requirements—Flexural members

Permissible stresses in concrete are given to control serviceability. They do not ensure adequate structural strength, which should be checked in conformance with other code requirements.

18.4.1 — Stresses in concrete immediately after prestress transfer (before time-dependent prestress losses) shall not exceed the following:

(a) Extreme fiber stress in compression .... $0.60f'_c$

(b) Extreme fiber stress in tension except as permitted in (c) .... $3\sqrt{f'_{ci}}$

(c) Extreme fiber stress in tension at ends of simply supported members .... $6\sqrt{f'_{ci}}$

Where computed tensile stresses exceed these values, bonded additional reinforcement (nonprestressed or prestressed) shall be provided in the tensile zone to resist the total tensile force in concrete computed with the assumption of an uncracked section.

R18.4.1 — The concrete stresses at this stage are caused by the force in the prestressing steel at transfer reduced by the losses due to elastic shortening of the concrete, relaxation of the prestressing steel, seating at transfer, and the stresses due to the weight of the member. Generally, shrinkage and creep are not included at this stage. These stresses apply to both pretensioned and post-tensioned concrete with proper modifications of the losses at transfer.

R18.4.1(b) and (c) — The tension stress limits of $3\sqrt{f'_{ci}}$ and $6\sqrt{f'_{ci}}$ refer to tensile stress at locations other than the precompressed tensile zone. Where the tensile stresses exceed the permissible values, the total force in the tensile stress zone may be calculated and reinforcement proportioned on the basis of this force at a stress of $0.6f_y$, but not more than 30,000 psi. The effects of creep and shrinkage begin to reduce the tensile stress almost immediately; however, some tension remains in these areas after allowance is made for all prestress losses.

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18.4.2 — Net tension under service loads shall not be permitted in unreinforced sections in prestressed concrete environmental engineering structures. Nonprestressed reinforcement at the stress levels permitted in Appendix I shall be provided to carry any net concrete tension in a prestressed concrete member. For prestressed flexural members, stresses in concrete at service loads (based on uncracked section properties, and after allowance for all prestress losses) shall not exceed the following:

(a) Extreme fiber stress in compression due to prestress plus sustained loads .... $0.45f'_c$

(b) Extreme fiber stress in compression due to prestress plus total load .... $0.60f'_c$

(c) Extreme fiber stress in tension in precompressed tensile zone .... $6\sqrt{f'_c}$

(d) Extreme fiber stress in tension in precompressed tensile zone of members (except two-way slab systems), where analysis based on transformed cracked sections and on bilinear moment-deflection relationships shows that immediate and long-term deflections comply with requirements of 9.5.4, and where cover requirements comply with 7.7.3.2.................. $12\sqrt{f'_c}$

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R18.4.2(a) and (b) — The compression stress limit of $\mathbf{0.45}f'_c$ was conservatively established to decrease the probability of failure of prestressed concrete members due to repeated loads. This limit seemed reasonable to preclude excessive creep deformation. At higher values of stress, creep strains tend to increase more rapidly as applied stress increases.

The change in allowable stress in the ACI 318-95 code recognized that fatigue tests of prestressed concrete have shown that concrete failures are not the controlling criterion. Designs with transient live loads that are large compared with sustained dead loads have been penalized by the previous single compression stress limit. Therefore, the new stress limit of $\mathbf{0.60}f'_c$ permits a one-third increase in allowable compression stress for members subject to transient loads.

Sustained live load is any portion of the service live load that will be sustained for a sufficient period to cause significant time-dependent deflections. Thus, when sustained live and dead loads are a large percentage of total service load, the $\mathbf{0.45}f'_c$ limit of 18.4.2(a) may control. On the other hand, when a large portion of the total service load consists of a transient or temporary service live load, the increased stress limit of 18.4.2(b) may apply.

The compression stress limit of $\mathbf{0.45}f'_c$ for prestress plus sustained loads will continue to control the long-term behavior of prestressed members.

R18.4.2(c) — The precompressed tensile zone is that portion of the member cross section in which flexural tension occurs under dead and live loads. Prestressed concrete is usually designed so that the prestress force introduces compression into this zone, thus effectively reducing the magnitude of the tensile stress.

The permissible tensile stress of $\mathbf{6}\sqrt{f'_c}$ is compatible with the concrete covers required by 7.7.3.1. For conditions of corrosive environments, defined as an environment in which chemical attack such as seawater, corrosive industrial atmosphere, sewer gas, or other highly corrosive environments are encountered, greater cover than that required by 7.7.3.1 should be used, in accordance with 7.7.3.2, and tension stresses reduced to eliminate possible cracking at service loads. The engineer needs to use judgment to determine the amount of increased cover and whether reduced tension stresses are required.

R18.4.2(c) and (d) — The permissible concrete tensile stress depends on whether or not enough bonded reinforcement is provided to control cracking. Such bonded reinforcement may consist of prestressed or nonprestressed tendons or of reinforcing bars. It should be noted that the control of cracking depends not only on the amount of reinforcement provided, but also on its distribution over the tensile zone.

Because of the bonded reinforcement requirements of 18.9, it is considered that the behavior of segmental members

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generally will be comparable to that of similarly constructed monolithic concrete members. Therefore, the permissible tensile stress limits of 18.4.2(c) and 18.4.2(d) apply to both segmental and monolithic members. If deflections are important, the built-in cracks of segmental members should be considered in the computations.

R18.4.2(d) — The permissible tensile stress of $12\sqrt{f'_c}$ provides improved service load performance, especially when live loads are of a transient nature. To take advantage of the increased permissible stress, the engineer is required to increase the concrete protection on the reinforcement, as stipulated in 7.7.3.2, and to investigate the deflection characteristics of the member, particularly at the load where the member changes from uncracked behavior to cracked behavior.

The exclusion of two-way slab systems is based on Reference 18.1, which recommends that the permissible tension stress be not greater than $6\sqrt{f'_c}$ for design of prestressed concrete flat plates analyzed by the equivalent frame method or other approximate methods. For flat plate designs based on more exact analyses, or for other two-way slab systems rigorously analyzed and designed for strength and serviceability, the limiting stress may be exceeded in accordance with 18.4.3.

Reference 18.2 provides information on the use of bilinear moment-deflection relationships.

18.4.3 — Permissible stresses in concrete of 18.4.1 and 18.4.2 shall be permitted to be exceeded if shown by test or analysis that performance will not be impaired.

R18.4.3 — This section provides a mechanism whereby development of new products, materials, and techniques in prestressed concrete construction need not be inhibited by code limits on stress. Approvals for the design should be in accordance with 1.4 of the code.

18.5 — Permissible stresses in prestressing steel

R18.5 — Permissible stresses in prestressing steel

The code does not distinguish between temporary and effective prestressing steel stresses. Only one limit on prestressing steel stress is provided because the initial prestressing steel stress (immediately after transfer) can prevail for a considerable time, even after the structure has been put into service. This stress, therefore, should have an adequate safety factor under service conditions, and cannot be considered as a temporary stress. Any subsequent decrease in prestressing steel stress due to losses can only improve conditions and, hence, no limit on such stress decrease is provided in the code.

18.5.1 — Tensile stress in prestressing steel shall not exceed the following:

(a) Due to prestressing steel jacking force ....$0.94f_{py}$

but not greater than the lesser of $\mathbf{0.80}f_{pu}$ and the maximum value recommended by the manufacturer of prestressing steel or anchorages;

R18.5.1 — With the ACI 318-83 code, permissible stresses in prestressing steel are revised to recognize the higher yield strength of low-relaxation wire and strand meeting the requirements of ASTM A 421 and A 416 of 3.5.5. For such prestressing steel, it is more appropriate to specify permissible stresses in terms of specified minimum ASTM yield strength rather than specified minimum ASTM tensile strength. For the low-relaxation wire and strands, with f_{py} equal to $0.90f_{pu}$, the $0.94f_{py}$ and $0.82f_{py}$ limits are equivalent

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(b) Immediately after prestress transfer......... $0.82f_{py}$

but not greater than $\mathbf{0.74}f_{pu}$;

(c) Post-tensioning tendons, at anchorages and couplers, immediately after tendon anchorage.... $0.70f_{pu}$

18.6 — Loss of prestress

18.6.1 — To determine effective prestress f_{se}, allowance for the following sources of loss of prestress shall be considered:

(a) Prestresssing steel seating at transfer;

(b) Elastic shortening of concrete;

(c) Creep of concrete;

(d) Shrinkage of concrete;

(e) Relaxation of prestressing steel stress;

(f) Friction loss due to intended or unintended curvature in post-tensioning tendons.

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to $\mathbf{0.85}f_{pu}$ and $\mathbf{0.74}f_{pu}$, respectively. In the 1986 ACI 318 revision and in the ACI 318-89 code, the maximum jacking stress for low-relaxation prestressing steel was reduced to $\mathbf{0.80}f_{pu}$ to ensure closer compatibility with the maximum prestressing steel stress value of $\mathbf{0.74}f_{pu}$ immediately after prestress transfer. The higher yield strength of the low-relaxation prestressing steel does not change the effectiveness of tendon anchorage devices; thus, the permissible stress at post-tensioning anchorage devices and couplers is not increased above the previously permitted value of $\mathbf{0.70}f_{pu}$. For ordinary prestressing steel (wire, strands, and bars) with f_{py} equal to $\mathbf{0.85}f_{pu}$, the $\mathbf{0.94}f_{py}$ and $\mathbf{0.82}f_{py}$ limits are equivalent to $\mathbf{0.80}f_{pu}$ and $\mathbf{0.70}f_{pu}$, respectively, the same as permitted in the ACI 318-77 code. For bar prestressing steel with f_{py} equal to $\mathbf{0.80}f_{pu}$, the same limits are equivalent to $\mathbf{0.75}f_{pu}$ and $\mathbf{0.66}f_{pu}$, respectively.

Because of the higher allowable initial prestressing steel stresses permitted since the ACI 318-83 code, final stresses can be greater. Designers should be concerned with setting a limit on final stress when the structure is subject to corrosive conditions or repeated loadings.

R18.6 — Loss of prestress

R18.6.1 — Long-term losses calculated for prestressed concrete tanks intended to be filled with liquid some or most of the time can vary significantly depending upon whether the tank is assumed to be full or empty. The designer needs to use judgment to determine how to estimate the long-term losses between these two boundary conditions, taking into account the anticipated normal operating conditions and fluctuations in liquid level with time.

For an explanation of how to compute prestress losses, see References 18.3 through 18.6. Lump-sum values of prestress losses for both pretensioned and post-tensioned members that were indicated in pre-1983 editions of the 318 commentary are considered obsolete. Reasonably accurate estimates of prestress losses can be easily calculated in accordance with the recommendations in Reference 18.6 that include consideration of initial stress level ($\mathbf{0.7}f_{pu}$ or higher), type of steel (stress-relieved or low-relaxation; wire, strand, or bar), exposure conditions, and type of construction (pretensioned, bonded post-tensioned, or unbonded post-tensioned).

Actual losses, greater or smaller than the computed values, have little effect on the design strength of the member, but affect service load behavior (deflections, camber, cracking load) and connections. At service loads, overestimation of prestress losses can be almost as detrimental as underestimation, because the former can result in excessive camber and horizontal movement.

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18.6.2 — Friction loss in post-tensioning tendons

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R18.6.2 — Friction loss in post-tensioning tendons

The coefficients tabulated in Table R18.6.2 give a range that generally can be expected. Due to the many types of prestressing steel ducts and sheathing available, these values can only serve as a guide. Where rigid conduit is used, the wobble coefficient K can be considered as zero. For large-diameter prestressing steel in semi-rigid type conduit, the wobble factor can also be considered zero. Values of the coefficients to be used for the particular types of prestressing steel and particular types of ducts should be obtained from the manufacturers of the tendons. An unrealistically low evaluation of the friction loss can lead to improper camber of the member and inadequate prestress. Overestimation of the friction may result in extra prestressing force if the estimated friction values are not attained in the field. This could lead to excessive camber and excessive shortening of a member. If the friction factors are determined to be less than those assumed in the design, the prestressing steel stressing should be adjusted to give only that prestressing force in the critical portions of the structure required by the design.

18.6.2.1 — Effect of friction loss in post-tensioning tendons shall be computed by

$$P_s = P_x e^{(K\ell_x + \mu\alpha)} \qquad (18\text{-}1)$$

When $(K\ell_x + \mu\alpha)$ is not greater than 0.3, effect of friction loss shall be permitted to be computed by

$$P_s = P_x(1 + K\ell_x + \mu\alpha) \qquad (18\text{-}2)$$

18.6.2.2 — Friction loss shall be based on experimentally determined wobble K and curvature μ friction coefficients, and shall be verified during tendon stressing operations.

TABLE R18.6.2—FRICTION COEFFICIENTS FOR POST-TENSIONED TENDONS FOR USE IN EQ. (18-1) OR (18-2)

			Wobble coefficient K	Curvature coefficient μ
		Wire tendons	0.0010-0.0015	0.15-0.25
		High-strength bars	0.0001-0.0006	0.08-0.30
		7-wire strand	0.0005-0.0020	0.15-0.25
Unbonded tendons	Mastic coated	Wire tendons	0.0010-0.0020	0.05-0.15
		7-wire strand	0.0010-0.0020	0.05-0.15
	Pre-greased	Wire tendons	0.0003-0.0020	0.05-0.15
		7-wire strand	0.0003-0.0020	0.05-0.15

18.6.2.3 — Values of wobble and curvature friction coefficients used in design shall be shown on design drawings.

R18.6.2.3 — When the safety or serviceability of the structure may be involved, the acceptable range of prestressing steel jacking forces or other limiting requirements should either be given or approved by the structural engineer in conformance with the permissible stresses of 18.4 and 18.5.

18.6.3 — Where loss of prestress in a member occurs due to connection of member to adjoining construction, such loss of prestress shall be allowed for in design.

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18.7 — Flexural strength

18.7.1 — Design moment strength of flexural members shall be computed by the strength design methods of the code. For prestressing steel, f_{ps} shall be substituted for f_y in strength computations.

18.7.2 — As an alternative to a more accurate determination of f_{ps} based on strain compatibility, the following approximate values of f_{ps} shall be permitted to be used if f_{se} is not less than $0.5f_{pu}$.

(a) For members with bonded tendons

$$f_{ps} = f_{pu}\left\{1 - \frac{\gamma_p}{\beta_1}\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}(\omega - \omega')\right]\right\} \quad (18\text{-}3)$$

If any compression reinforcement is taken into account when calculating f_{ps} by Eq. (18-3), the term

$$\left[\rho_p \frac{f_{pu}}{f_c'} + \frac{d}{d_p}(\omega - \omega')\right]$$

shall be taken not less than 0.17 and d' shall be no greater than $0.15d_p$;

(b) For members with unbonded tendons and with a span-to-depth ratio of 35 or less

$$f_{ps} = f_{se} + 10{,}000 + \frac{f_c'}{100\rho_p} \quad (18\text{-}4)$$

but f_{ps} in Eq. (18-4) shall not be taken greater than f_{py}, nor $(f_{se} + 60{,}000)$;

(c) For members with unbonded tendons and with a span-to-depth ratio greater than 35

$$f_{ps} = f_{se} + 10{,}000 + \frac{f_c'}{300\rho_p} \quad (18\text{-}5)$$

but f_{ps} in Eq. (18-5) shall not be taken greater than f_{py}, nor $(f_{se} + 30{,}000)$.

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R18.7 — Flexural strength

R18.7.1 — Design moment strength of prestressed flexural members may be computed using strength equations similar to those for conventionally reinforced concrete members. Textbooks and ACI 318R-83[18.7] provide strength equations for rectangular and flanged sections, with tension reinforcement only and with tension and compression reinforcement. When part of the prestressing steel is in the compression zone, a method based on applicable conditions of equilibrium and compatibility of strains at a factored load condition should be used.

For other cross sections, the design moment strength ϕM_n is computed by a general analysis based on stress and strain compatibility, using the stress-strain properties of the prestressing steel and the assumptions given in 10.2.

R18.7.2 — Equation (18-3) may underestimate the strength of beams with high percentages of reinforcement and, for more accurate evaluations of their strength, the strain compatibility and equilibrium method should be used. Use of Eq. (18-3) is appropriate when all of the prestressed reinforcement is in the tension zone. When part of the prestressed reinforcement is in the compression zone, a strain compatibility and equilibrium method should be used.

By inclusion of the ω' term, Eq. (18-3) reflects the increased value of f_{ps} obtained when compression reinforcement is provided in a beam with a large reinforcement index. When the term $[\rho_p f_{pu}/f'_c + (d/d_p)(\omega - \omega')]$ in Eq. (18-3) is small, the neutral axis depth is small; hence, the compressive reinforcement does not develop its yield strength, and Eq. (18-3) becomes unconservative. This is the reason why the term $[\rho_p f_{pu}/f'_c + (d/d_p)(\omega - \omega')]$ in Eq. (18-3) may not be taken less than 0.17 if compression reinforcement is taken into account when computing f_{ps}. (Note that if the compression reinforcement is neglected when using Eq. (18-3), that is, ω' is taken as zero, then the term $[\rho_p f_{pu}/f'_c + (d/d_p)\omega]$ may be less than 0.17 and hence, an increased and correct value of f_{ps} is obtained.)

When d' is large, the strain in compression reinforcement can be considerably less than its yield strain. In such a case, the compression reinforcement does not influence f_{ps} as favorably as implied by Eq. (18-3). It is for this reason that the applicability of Eq. (18-3) is limited to beams in which d' is less than or equal to $0.15d_p$.

The term $[\rho_p f_{pu}/f'_c + (d/d_p)(\omega - \omega')]$ in Eq. (18-3) may also be written $[\rho_p f_{pu}/f'_c + A_s f_y/(bd_p f'_c) - A'_s f_y/(bd_p f'_c)]$. This form may sometimes be more conveniently used, for example, when there is no unprestressed tension reinforcement.

Equation (18-5) reflects results of tests on members with unbonded tendons and span-depth ratios greater than 35 (one-way slabs, flat plates, and flat slabs).[18.8] These tests also indicate that Eq. (18-4), formerly used for all span-

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depth ratios, would overestimate the amount of stress increase in such members. Although these same tests indicate that the moment strength of those shallow members designed using Eq. (18-4) meets the factored load strength requirements, this result reflects the code requirements for minimum bonded reinforcement, as well as the limitation on concrete tensile stress that often controls the amount of prestressing force provided.

18.7.3 — Nonprestressed reinforcement conforming to 3.5.3, if used with prestressing steel, shall be permitted to be considered to contribute to the tensile force and to be included in moment strength computations at a stress equal to the specified yield strength f_y. Other nonprestressed reinforcement shall be permitted to be included in strength computations only if a strain compatibility analysis is performed to determine stresses in such reinforcement.

18.8 — Limits for reinforcement of flexural members

18.8.1 — Ratio of prestressed and nonprestressed reinforcement used for computation of moment strength of a member, except as provided in 18.8.2, shall be such that ω_p, $[\omega_p + (d/d_p)(\omega - \omega')]$, or $[\omega_{pw} + (d/d_p)(\omega_w - \omega'_w)]$ is not greater than $0.36\beta_1$.

18.8.2 — When a reinforcement ratio in excess of that specified in 18.8.1 is provided, design moment strength shall not exceed the moment strength based on the compression portion of the moment couple.

18.8.3 — Total amount of prestressed and nonprestressed reinforcement shall be adequate to develop a factored load at least 1.2 times the cracking load computed on the basis of the modulus of rupture f_r specified in 9.5.2.3, except for flexural members with shear and flexural strength at least twice that required by 9.2.

R18.8 — Limits for reinforcement of flexural members

R18.8.1 — It can be shown that the terms ω_p, $[\omega_p + (d/d_p)(\omega - \omega')]$, and $[\omega_{pw} + (d/d_p)(\omega_w - \omega'_w)]$ are each equal to $0.85a/d_p$, where a is the depth of the equivalent rectangular stress distribution for the section under consideration, as defined in 10.2.7.1. Use of this relationship can simplify the calculations necessary to check compliance with 18.8.1.

R18.8.2 — Design moment strength of overreinforced members may be computed using strength equations similar to those for conventionally reinforced concrete members. Textbooks and ACI 318R-83[18.7] provide strength equations for rectangular and flanged sections.

R18.8.3 — This provision is a precaution against abrupt flexural failure developing immediately after cracking. A flexural member designed according to code provisions requires considerable additional load beyond cracking to reach its flexural strength. Thus, considerable deflection would warn that the member strength is approaching. If the flexural strength should be reached shortly after cracking, the warning deflection would not occur.

18.9 — Minimum bonded reinforcement

18.9.1 — A minimum area of bonded reinforcement shall be provided in all flexural members with unbonded tendons as required by 18.9.2 and 18.9.3.

R18.9 — Minimum bonded reinforcement

R18.9.1 — Some bonded reinforcement is required by the code in members prestressed with unbonded tendons to ensure flexural performance at ultimate member strength, rather than behavior as a tied arch, and to control cracking at service load when tensile stresses exceed the modulus of rupture of the concrete. Providing minimum bonded reinforcement, as specified in 18.9, helps to ensure adequate performance.

Research has shown that unbonded post-tensioned members do not inherently provide large capacity for energy dissipation

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under severe earthquake loadings because the member response is primarily elastic. For this reason, unbonded post-tensioned structural elements reinforced in accordance with the provisions of this section should be assumed to carry only vertical loads and to act as horizontal diaphragms between energy dissipating elements under earthquake loadings of the magnitude defined in 21.2.1.1. The minimum bonded reinforcement areas required by Eq. (18-6) and (18-8) are absolute minimum areas independent of grade of steel or design yield strength.

18.9.2 — Except as provided in 18.9.3, minimum area of bonded reinforcement shall be computed by

$$A_s = 0.004A \tag{18-6}$$

18.9.2.1 — Bonded reinforcement required by Eq. (18-6) shall be uniformly distributed over precompressed tensile zone as close as practicable to extreme tension fiber.

18.9.2.2 — Bonded reinforcement shall be required regardless of service load stress conditions.

R18.9.2 — The minimum amount of bonded reinforcement for members other than two-way flat slab systems is based on research comparing the behavior of bonded and unbonded post-tensioned beams.[18.9] Based on this research, it is advisable to apply the provisions of 18.9.2 also to one-way slab systems.

18.9.3 — For two-way flat plates, defined as solid slabs of uniform thickness, minimum area and distribution of bonded reinforcement shall be as required in 18.9.3.1, 18.9.3.2, and 18.9.3.3.

R18.9.3 — The minimum amount of bonded reinforcement in two-way flat slab systems is based on reports by ACI-ASCE Committee 423.[18.3,18.10] Limited research available for two-way flat slabs with drop panels[18.11] indicates that behavior of these particular systems is similar to the behavior of flat plates. Reference 18.10 was revised by Committee 423 in 1983 to clarify that Section 18.9.3 applies to two-way flat slab systems.

18.9.3.1 — Bonded reinforcement shall not be required in positive moment areas where computed tensile stress in concrete at service load (after allowance for all prestress losses) does not exceed $2\sqrt{f'_c}$.

R18.9.3.1 — For usual loads and span lengths, flat plate tests summarized in the Committee 423 report[18.3] and experience since the ACI 318-63 code was adopted indicate satisfactory performance without bonded reinforcement in the areas described in 18.9.3.1.

18.9.3.2 — In positive moment areas where computed tensile stress in concrete at service load exceeds $2\sqrt{f'_c}$, minimum area of bonded reinforcement shall be computed by

$$A_s = \frac{N_c}{0.5f_y} \tag{18-7}$$

where design yield strength f_y shall not exceed 60,000 psi. Bonded reinforcement shall be uniformly distributed over precompressed tensile zone as close as practicable to extreme tension fiber.

R18.9.3.2 — In positive moment areas, where tensile stresses are between $2\sqrt{f'_c}$ and $6\sqrt{f'_c}$, a minimum bonded reinforcement area proportioned according to Eq. (18-7) is required. The tensile force N_c is computed at service load on the basis of an uncracked, homogeneous section.

18.9.3.3 — In negative moment areas at column supports, the minimum area of bonded reinforcement A_s in the top of the slab in each direction shall be computed by

$$A_s = 0.00075A_{cf} \tag{18-8}$$

R18.9.3.3 — Research on unbonded post-tensioned two-way flat slab systems evaluated by ACI-ASCE Committee 423[18.3,18.10-18.12] shows that bonded reinforcement in negative moment regions of two-way flat plates, proportioned on the basis of 0.075 percent of the cross-sectional area of the slab-beam strip, provides sufficient ductility and reduces crack width and spacing. To account for different adjacent

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Bonded reinforcement required by Eq. (18-8) shall be distributed between lines that are **1.5*h*** outside opposite faces of the column support. At least four bars or wires shall be provided in each direction. Spacing of bonded reinforcement shall not exceed 12 in.

18.9.4 — Minimum length of bonded reinforcement required by 18.9.2 and 18.9.3 shall be as required in 18.9.4.1, 18.9.4.2, and 18.9.4.3.

18.9.4.1 — In positive moment areas, minimum length of bonded reinforcement shall be one-third the clear span length and centered in positive moment area.

18.9.4.2 — In negative moment areas, bonded reinforcement shall extend one-sixth the clear span on each side of support.

18.9.4.3 — Where bonded reinforcement is provided for design moment strength in accordance with 18.7.3, or for tensile stress conditions in accordance with 18.9.3.2, minimum length also shall conform to provisions of Chapter 12.

18.10 — Statically indeterminate structures

18.10.1 — Frames and continuous construction of prestressed concrete shall be designed for satisfactory performance at service load conditions and for adequate strength.

18.10.2 — Performance at service load conditions shall be determined by elastic analysis, considering reactions, moments, shears, and axial forces produced by prestressing, creep, shrinkage, temperature change, axial deformation, restraint of attached structural elements, and foundation settlement.

18.10.3 — Moments to be used to compute required strength shall be the sum of the moments due to reactions induced by prestressing (with a load factor of 1.0) and the moments due to factored loads. Adjustment of the sum of these moments shall be permitted as allowed in 18.10.4.

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tributary spans, Eq. (18-8) is given on the basis of the equivalent frame as defined in 13.7.2 and pictured in Fig. R13.7.2. For rectangular slab panels, Eq. (18-8) is conservatively based upon the larger of the cross-sectional areas of the two intersecting equivalent frame slab-beam strips at the column. This ensures that the minimum percentage of steel recommended by research is provided in both directions. Concentration of this reinforcement in the top of the slab directly over and immediately adjacent to the column is important. Research also shows that where low tensile stresses occur at service load, satisfactory behavior has been achieved at factored load without bonded reinforcement. The code, however, requires minimum bonded reinforcement regardless of service load stress levels to help ensure flexural continuity and ductility, and to control cracking due to overload, temperature, or shrinkage. Research on post-tensioned flat plate-to-column connections is reported in References 18.13 through 18.17.

R18.9.4 — Bonded reinforcement should be adequately anchored to develop factored load forces. The requirements of Chapter 12 will ensure that bonded reinforcement required for flexural strength under factored loads in accordance with 18.7.3, or for tensile stress conditions at service load in accordance with 18.9.3.2, will be adequately anchored to develop tension or compression forces. For bonded reinforcement required by 18.9.2 or 18.9.3.3, but not required for flexural strength in accordance with 18.7.3, the minimum lengths apply. Research[18.1] on continuous spans shows that these minimum lengths provide adequate behavior under service load and factored load conditions.

R18.10 — Statically indeterminate structures

R18.10.3 — For statically indeterminate structures, the moments due to reactions induced by prestressing forces, generally referred to as secondary moments, are significant in both the elastic and inelastic states. When hinges and full redistribution of moments occur to create a statically determinate structure, secondary moments disappear. The elastic

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deformations caused by a nonconcordant tendon, however, change the amount of inelastic rotation required to obtain a given amount of moment redistribution. Conversely, for a beam with a given inelastic rotational capacity, the amount by which the moment at the support may be varied is changed by an amount equal to the secondary moment at the support due to prestressing. Thus, the code requires that secondary moments be included in determining design moments.

To determine the moments used in design, the order of calculation should be: (a) determine moments due to dead and live load; (b) modify by algebraic addition of secondary moments; and (c) redistribute as permitted. A positive secondary moment at the support caused by a tendon transformed downward from a concordant profile will, therefore, reduce the negative moments near the supports and increase the positive moments in the midspan regions. A tendon that is transformed upward will have the reverse effect.

18.10.4 — Redistribution of negative moments in continuous prestressed flexural members

18.10.4.1 — Where bonded reinforcement is provided at supports in accordance with 18.9.2, negative moments calculated by elastic theory for any assumed loading arrangement shall be permitted to be increased or decreased by not more than

$$20\left[1-\frac{\omega_p+\dfrac{d}{d_p}(\omega-\omega')}{0.36\beta_1}\right]\text{ percent}$$

18.10.4.2 — The modified negative moments shall be used for calculating moments at sections within spans for the same loading arrangement.

18.10.4.3 — Redistribution of negative moments shall be made only when the section at which moment is reduced is so designed that ω_p,$[\omega_p + (d/d_p)(\omega - \omega')]$, or $[\omega_{pw} + (d/d_p)(\omega_w - \omega'_w)]$, whichever is applicable, is not greater than $0.24\beta_1$.

R18.10.4 — Redistribution of negative moments in continuous prestressed flexural members

As member strength is approached, inelastic behavior at some sections can result in a redistribution of moments in prestressed concrete beams. Recognition of this behavior can be advantageous in design under certain circumstances. A rigorous design method for moment redistribution is quite complex. Recognition of moment redistribution, however, can be accomplished with the simple method of permitting a reasonable adjustment of the sum of the elastically calculated factored gravity load moments and the unfactored secondary moments due to prestress. The amount of adjustment should be kept within predetermined safe limits.

The amount of redistribution allowed depends on the ability of the critical sections to deform inelastically by a sufficient amount. Serviceability under service loads is taken care of by the limiting stresses of 18.4. The choice of $0.24\beta_1$ as the largest tension reinforcement index, ω_p, $[\omega_p + (d/d_p)(\omega - \omega')]$, or $[\omega_{pw} + (d/d_p)(\omega_w - \omega'_w)]$, for which redistribution of moments is allowed, is in agreement with the requirements for conventionally reinforced concrete of $0.5\rho_b$ stated in 8.4.

It can be shown that the terms ω_p, $[\omega_p + (d/d_p)(\omega - \omega')]$, and $[\omega_{pw} + (d/d_p)(\omega_w - \omega'_w)]$ that appear in 18.10.4.1 and 18.10.4.3 are each equal to $0.85a/d_p$, where a is the depth of the equivalent rectangular stress distribution for the section under consideration, as defined in 10.2.7.1. Use of this relationship can simplify the calculations necessary to determine the amount of moment redistribution permitted by 18.10.4.1 and to check compliance with the limitation on flexural reinforcement contained in 18.10.4.3.

For the moment redistribution principles of 18.10.4 to be applicable to beams with unbonded tendons, it is necessary that such beams contain sufficient bonded reinforcement to ensure they will act as beams after cracking, and not as a series of tied arches. The minimum bonded reinforcement requirements of 18.9 will serve this purpose.

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18.11 — Compression members— Combined flexure and axial loads

18.11.1 — Prestressed concrete members subject to combined flexure and axial load, with or without nonprestressed reinforcement, shall be proportioned by the strength design methods of this code for members without prestressing. Effects of prestress, creep, shrinkage, and temperature change shall be included.

18.11.2 — Limits for reinforcement of prestressed compression members

18.11.2.1 — Members with average prestress f_{pc} less than 225 psi shall have minimum reinforcement in accordance with 7.10, 10.9.1, and 10.9.2 for columns, or 14.3 for walls.

18.11.2.2 — Except for walls, members with average prestress f_{pc} equal to or greater than 225 psi shall have all tendons enclosed by spirals or lateral ties in accordance with the following:

(a) Spirals shall conform to 7.10.4;

(b) Lateral ties shall be at least No. 3 in size or welded wire fabric of equivalent area, and spaced vertically not to exceed 48 tie bar or wire diameters, or least dimension of compression member;

(c) Ties shall be located vertically not more than half a tie spacing above top of footing or slab in any story, and not more than half a tie spacing below lowest horizontal reinforcement in members supported above;

(d) Where beams or brackets frame into all sides of a column, ties shall be terminated not more than 3 in. below lowest reinforcement in such beams or brackets.

18.11.2.3 — For walls with average prestress f_{pc} equal to or greater than 225 psi, minimum reinforcement required by 14.3 shall not apply where structural analysis shows adequate strength and stability.

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R18.11 — Compression members— Combined flexure and axial loads

R18.11.2 — Limits for reinforcement of prestressed compression members

R18.11.2.3 — The minimum amounts of reinforcement, specified in 14.3 for walls, need not apply to prestressed concrete walls, provided the average prestress is 225 psi or greater and a complete structural analysis is made to show adequate strength and stability with lower amounts of reinforcement.

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18.12 — Slab systems

18.12.1 — Factored moments and shears in prestressed slab systems reinforced for flexure in more than one direction shall be determined in accordance with provisions of 13.7 (excluding 13.7.7.4 and 13.7.7.5), or by more detailed design procedures.

18.12.2 — Design moment strength of prestressed slabs required by 9.3 at every section shall be equal to or exceed the required strength considering 9.2, 18.10.3, and 18.10.4. Shear strength of prestressed slabs at columns shall be at least equal to the required strength considering 9.2, 9.3, 11.1, 11.12.2, and 11.12.6.2.

18.12.3 — At service load conditions, all serviceability limitations, including specified limits on deflections, shall be met, with appropriate consideration of the factors listed in 18.10.2.

18.12.4 — For normal live loads and loads uniformly distributed, spacing of tendons or groups of tendons in one direction shall not exceed eight times the slab thickness, nor 5 ft. Spacing of tendons also shall provide a minimum average prestress (after allowance for all prestress losses) of 125 psi on the slab section tributary to the tendon or tendon group. A minimum of two tendons shall be provided in each direction through the critical shear section over columns. Special consideration of tendon spacing shall be provided for slabs with concentrated loads.

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R18.12 — Slab systems

R18.12.1 — Use of the equivalent frame method of analysis (see 13.7) or more precise design procedures is required for determination of both service and factored moments and shears for prestressed slab systems. The equivalent frame method of analysis has been shown by tests of large structural models to satisfactorily predict factored moments and shears in prestressed slab systems. (See References 18.13 through 18.15 and 18.18 through 18.20.) The referenced research also shows that analysis using prismatic sections or other approximations of stiffness may provide erroneous results on the unsafe side. Section 13.7.7.4 is excluded from application to prestressed slab systems because it relates to reinforced slabs designed by the direct design method, and because moment redistribution for prestressed slabs is covered in 18.10.4. Section 13.7.7.5 is excluded from application to prestressed slab systems because the distribution of moments between column strips and middle strips required by 13.7.7.5 is based on tests for reinforced concrete slabs. Simplified methods of analysis using average coefficients do not apply to prestressed concrete slab systems.

R18.12.2 — Tests indicate that the moment and shear strength of prestressed slabs is controlled by total prestressing steel strength and by the amount and location of nonprestressed reinforcement, rather than by tendon distribution. (See References 18.13 through 18.15 and 18.18 through 18.20.)

R18.12.3 — For prestressed flat slabs continuous over two or more spans in each direction, the span-thickness ratio generally should not exceed 42 for floors and 48 for roofs; these limits may be increased to 48 and 52, respectively, if calculations verify that both short- and long-term deflection, camber, and vibration frequency and amplitude are not objectionable.

Short- and long-term deflection and camber should be computed and checked against the requirements of serviceability of the particular usage of the structure.

The maximum length of a slab between construction joints is generally limited to 100 to 150 ft to minimize the effects of slab shortening, and to avoid excessive loss of prestress due to friction.

R18.12.4 — This section provides specific guidance concerning tendon distribution that will permit the use of banded tendon distributions in one direction. This method of tendon distribution has been shown to provide satisfactory performance by structural research.

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18.12.5 — In slabs with unbonded tendons, bonded reinforcement shall be provided in accordance with 18.9.3 and 18.9.4.

18.12.6 — In lift slabs, bonded bottom reinforcement shall be detailed in accordance with 13.3.8.6.

18.13 — Post-tensioned tendon anchorage zones

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R18.13 — Post-tensioned tendon anchorage zones

Section 18.13 was extensively revised in the 318-99 code and was made compatible with the 1996 AASHTO "Standard Specifications for Highway Bridges"[18.21] and the recommendations of NCHRP *Report* 356.[18.22]

Following the adoption by AASHTO 1994 of comprehensive provisions for post-tensioned anchorage zones, ACI Committee 350 revised the code to be generally consistent with the AASHTO requirements. Thus, the highly detailed AASHTO provisions for analysis and reinforcement detailing are deemed to satisfy the more general ACI 318 requirements. In the specific areas of anchorage device evaluation and acceptance testing, ACI 318 incorporates the detailed AASHTO provisions by reference.

18.13.1 — Anchorage zone

The anchorage zone shall be considered as composed of two zones:

(a) The local zone is the rectangular prism (or equivalent rectangular prism for circular or oval anchorages) of concrete immediately surrounding the anchorage device and any confining reinforcement;

(b) The general zone is the anchorage zone as defined in 2.1 and includes the local zone.

R18.13.1 — Anchorage zone

Based on the principle of Saint-Venant, the extent of the anchorage zone may be estimated as approximately equal to the largest dimension of the cross section. Local zone and general zone are shown in Fig. R18.13.1(a). When anchorage devices located away from the end of the member are tensioned, large tensile stresses exist locally behind and ahead of the device. These tensile stresses are induced by incompatibility of deformations ahead of (as shown in Fig. R.18.13.1(b)) and behind the anchorage device. The entire shaded region should be considered, as shown in Fig. R18.13.1(b).

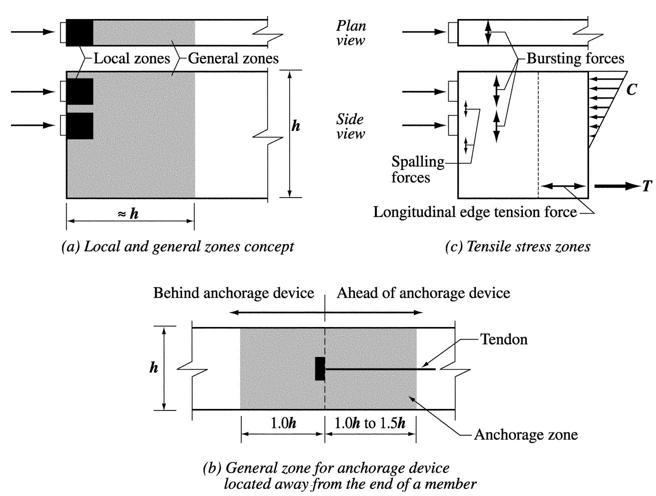

Fig. R18.13.1—Anchorage zones.

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18.13.2 — Local zone

18.13.2.1 — Design of local zones shall be based upon the factored prestressing force P_{su} and the requirements of 9.2.5 and 9.3.2.5.

18.13.2.2 — Local-zone reinforcement shall be provided where required for proper functioning of the anchorage device.

18.13.2.3 — Local-zone requirements of 18.13.2.2 are satisfied by 18.14.1 or 18.15.1 and 18.15.2.

18.13.3 — General zone

18.13.3.1 — Design of general zones shall be based upon the factored prestressing force P_{su} and the requirements of 9.2.5 and 9.3.2.5.

18.13.3.2 — General-zone reinforcement shall be provided where required to resist bursting, spalling, and longitudinal edge tension forces induced by anchorage devices. Effects of abrupt change in section shall be considered.

18.13.3.3 — The general-zone requirements of 18.13.3.2 are satisfied by 18.13.4, 18.13.5, 18.13.6, and whichever one of 18.14.2 or 18.14.3 or 18.15.3 is applicable.

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R18.13.2 — Local zone

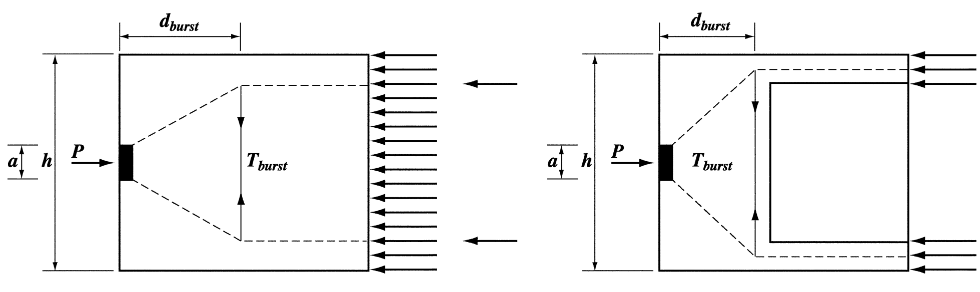
The local zone resists the very high local stresses introduced by the anchorage device and transfers them to the remainder of the anchorage zone. The behavior of the local zone is strongly influenced by the specific characteristics of the anchorage device and its confining reinforcement, and less influenced by the geometry and loading of the overall structure. Local-zone design sometimes cannot be completed until specific anchorage devices are determined at the shop drawing stage. When special anchorage devices are used, the anchorage device supplier should furnish the test information to show the device is satisfactory under AASHTO "Standard Specifications for Highway Bridges," Division II, Article 10.3.2.3 and provide information regarding necessary conditions for use of the device. The main considerations in local-zone design are the effects of the high bearing pressure and the adequacy of any confining reinforcement provided to increase the capacity of the concrete resisting bearing stresses.

R18.13.3 — General zone

Within the general zone the usual assumption of beam theory that plane sections remain plane is not valid.

Design should consider all regions of tensile stresses that can be caused by the tendon anchorage device, including bursting, spalling, and edge tension as shown in Fig. R18.13.1(c). Also, the compressive stresses immediately ahead (as shown in Fig. R18.13.1(b)) of the local zone should be checked. Sometimes reinforcement requirements cannot be determined until specific tendon and anchorage device layouts are determined at the shop-drawing stage. Design and approval responsibilities should be clearly assigned in the contract documents.

Abrupt changes in section can cause substantial deviation in force paths. These deviations can greatly increase tension forces as shown in Fig. R18.13.3.



(a) Rectangular section
T_{burst} ~ 0.25 P

(b) Flanged section with end diaphragm
T_{burst} ~ 0.50 P

Fig. R18.13.3—Effect of cross section change.

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18.13.4 — Nominal material strengths

18.13.4.1 — Nominal tensile strength of bonded reinforcement is limited to f_y for nonprestressed reinforcement and to f_{py} for prestressed reinforcement. Nominal tensile stress of unbonded prestressed reinforcement for resisting tensile forces in the anchorage zone shall be limited to $f_{ps} = f_{se} + 10,000$.

18.13.4.2 — Except for concrete confined within spirals or hoops providing confinement equivalent to that corresponding to Eq. (10-5), nominal compressive strength of concrete in the general zone shall be limited to $0.7\lambda f'_{ci}$.

18.13.4.3 — Compressive strength of concrete at time of post-tensioning shall be specified on the design drawings. Unless oversize anchorage devices sized to compensate for the lower compressive strength are used or the prestressing steel is stressed to no more than 50 percent of the final prestressing force, prestressing steel shall not be stressed until f'_{ci}, as indicated by tests consistent with the curing of the member, is at least 4000 psi for multistrand tendons or at least 2500 psi for single-strand or bar tendons.

18.13.5 — Design methods

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R18.13.4 — Nominal material strengths

Some inelastic deformation of concrete is expected because anchorage zone design is based on a strength approach. The low value for the nominal compressive strength for unconfined concrete reflects this possibility. For well-confined concrete, the effective compressive strength could be increased (see Reference 18.23). The value for nominal tensile strength of bonded prestressing steel is limited to the yield strength of the prestressing steel because Eq. (18-3) may not apply to these nonflexural applications. The value for unbonded prestressing steel is based on the values of 18.7.2(b) and (c), but is somewhat limited for these short-length, nonflexural applications. Test results given in Reference 18.23 indicate that the compressive stress introduced by auxiliary prestressing applied perpendicular to the axis of the main tendons is effective in increasing the anchorage zone capacity. The inclusion of the λ factor for lightweight concrete reflects its lower tensile strength, which is an indirect factor in limiting compressive stresses, as well as the wide scatter and brittleness exhibited in some lightweight concrete anchorage zone tests.

The designer is required to specify concrete strength at the time of stressing in the project drawings and specifications. To limit early shrinkage cracking, monostrand tendons are sometimes stressed at concrete strengths less than 2500 psi. In such cases, either oversized monostrand anchorages are used, or the strands are stressed in stages, often to levels 1/3 to 1/2 the final prestressing force.

R18.13.5 — Design methods

The list of design methods in 18.13.5.1 includes those procedures for which fairly specific guidelines have been given in References 18.21 and 18.22. These procedures have been shown to be conservative predictors of strength when compared with test results.[18.23] The use of strut-and-tie models is especially helpful for general zone design.[18.23] In many anchorage applications, where substantial or massive concrete regions surround the anchorages, simplified equations can be used, except in the cases noted in 18.13.5.2.

For many cases, simplified equations based on References 18.21 and 18.22 can be used. Values for the magnitude of the bursting force T_{burst} and for its centroidal distance from the major bearing surface of the anchorage d_{burst} may be estimated from Eq. (R18-1) and (R18-2), respectively. The terms of Eq. (R18-1) and (R18-2) are shown in Fig. R18.13.5 for a prestressing force with small eccentricity. In the applications of Eq. (R18-1) and (R18-2), the specified stressing sequence should be considered if more than one tendon is present

$$T_{burst} = 0.25\Sigma P_{su}\left[1 - \frac{a}{h}\right] \qquad \text{(R18-1)}$$

$$d_{burst} = 0.5(h - 2e) \qquad \text{(R18-2)}$$

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where

ΣP_{su} = the sum of the total factored prestressing force for the stressing arrangement considered, lb;

a = the depth of anchorage device or single group of closely spaced devices in the direction considered, in.;

e = the eccentricity (always taken as positive) of the anchorage device or group of closely spaced devices with respect to the centroid of the cross section, in.;

h = the depth of the cross section in the direction considered, in.

Anchorage devices should be treated as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage device in the direction considered.

The spalling force for tendons for which the centroid lies within the kern of the section may be estimated as 2 percent of the total factored prestressing force, except for multiple anchorage devices with center-to-center spacing greater than 0.4 times the depth of the section. For large spacings and for cases where the centroid of the tendons is located outside the kern, a detailed analysis is required. In addition, in the post-tensioning of thin sections, or flanged sections, or irregular sections, or when the tendons have appreciable curvature within the general zone, more general procedures such as those of AASHTO Articles 9.21.4 and 9.21.5 will be required. Detailed recommendations for design principles that apply to all design methods are given in Article 9.21.3.4 of Reference 18.21.

18.13.5.1 — The following methods shall be permitted for the design of general zones provided that the specific procedures used result in prediction of strength in substantial agreement with results of comprehensive tests:

(a) Equilibrium based plasticity models (strut-and-tie models);

(b) Linear stress analysis (including finite element analysis or equivalent); or

(c) Simplified equations where applicable.

18.13.5.2 — Simplified equations shall not be used where member cross sections are nonrectangular, where discontinuities in or near the general zone cause deviations in the force flow path, where minimum edge distance is less than 1-1/2 times the anchorage device lateral dimension in that direction, or where multiple anchorage devices are used in other than one closely spaced group.

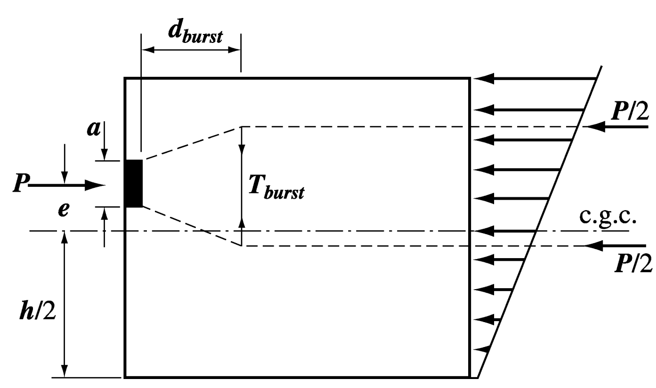

Fig. R18.13.5—Strut-and-tie model example.

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18.13.5.3 — The stressing sequence shall be specified on the design drawings and considered in the design.

18.13.5.4 — Three-dimensional effects shall be considered in design and analyzed using three-dimensional procedures or approximated by considering the summation of effects for two orthogonal planes.

18.13.5.5 — For anchorage devices located away from the end of the member, bonded reinforcement shall be provided to transfer at least $0.35P_{su}$ into the concrete section behind the anchor. Such reinforcement shall be placed symmetrically around the anchorage devices and shall be fully developed both behind and ahead of the anchorage devices.

18.13.5.6 — Where tendons are curved in the general zone, except for monostrand tendons in slabs or where analysis shows reinforcement is not required, bonded reinforcement shall be provided to resist radial and splitting forces.

18.13.5.7 — Except for monostrand tendons in slabs or where analysis shows reinforcement is not required, minimum reinforcement with a nominal tensile strength equal to 2 percent of each prestressing tendon force shall be provided in orthogonal directions parallel to the back face of all anchorage zones to limit spalling.

18.13.5.8 — Tensile strength of concrete shall be neglected in calculations of reinforcement requirements.

18.13.6 — Detailing requirements

Selection of reinforcement sizes, spacings, cover, and other details for anchorage zones shall make allowances for tolerances on the bending, fabrication, and placement of reinforcement, for the size of aggregate, and for adequate placement and consolidation of the concrete.

18.14 — Design of anchorage zones for monostrand or single 5/8 in. diameter bar tendons

18.14.1 — Local zone design

Monostrand or single 5/8 in. or smaller diameter bar anchorage devices and local zone reinforcement shall

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R18.13.5.3 — The sequence of anchorage device stressing can have a significant effect on the general zone stresses. Therefore, it is important to consider not only the final stage of a stressing sequence with all tendons stressed, but also intermediate stages during construction. The most critical bursting forces caused by each of the sequentially post-tensioned tendon combinations, as well as that of the entire group of tendons, should be taken into account.

R18.13.5.4 — The provision for three-dimensional effects was included to alert the designer to effects perpendicular to the main plane of the member, such as bursting forces in the thin direction of webs or slabs. In many cases, these effects can be determined independently for each direction, but some applications require a fully three-dimensional analysis (for example diaphragms for the anchorage of external tendons).

R18.13.5.5 — Where anchorages are located away from the end of a member, local tensile stresses are generated behind these anchorages (see Fig. R18.13.1(b)) due to compatibility requirements for deformations ahead of and behind the anchorages. Bonded tie-back reinforcement is required in the immediate vicinity of the anchorage to limit the extent of cracking behind the anchorage. The requirement $0.35P_{su}$ was developed using 25 percent of the unfactored prestressing force being resisted by reinforcement at $0.6f_y$.

R18.14 — Design of anchorage zones for monostrand or single 5/8 in. diameter bar tendons

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meet the requirements of ACI 423.6 or the special anchorage device requirements of 18.15.2.

18.14.2 — General-zone design for slab tendons

18.14.2.1 — For anchorage devices for 0.5 in. or smaller diameter strands in normalweight concrete slabs, minimum reinforcement meeting the requirements of 18.14.2.2 and 18.14.2.3 shall be provided unless a detailed analysis satisfying 18.13.5 shows such reinforcement is not required.

18.14.2.2 — Two horizontal bars at least No. 4 in size shall be provided parallel to the slab edge. They shall be permitted to be in contact with the front face of the anchorage device and shall be within a distance of $1/2h$ ahead of each device. Those bars shall extend at least 6 in. either side of the outer edges of each device.

18.14.2.3 — If the center-to-center spacing of anchorage devices is 12 in. or less, the anchorage devices shall be considered as a group. For each group of six or more anchorage devices, $n + 1$ hairpin bars or closed stirrups at least No. 3 in size shall be provided, where n is the number of anchorage devices. One hairpin bar or stirrup shall be placed between each anchorage device and one on each side of the group. The hairpin bars or stirrups shall be placed with the legs extending into the slab perpendicular to the edge. The center portion of the hairpin bars or stirrups shall be placed perpendicular to the plane of the slab from $3h/8$ to $h/2$ ahead of the anchorage devices.

18.14.2.4—For anchorage devices not conforming to 18.14.2.1, minimum reinforcement shall be based upon a detailed analysis satisfying 18.13.5.

18.14.3 — General-zone design for groups of monostrand tendons in beams and girders

Design of general zones for groups of monostrand tendons in beams and girders shall meet the requirements of 18.13.3 through 18.13.5.

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R18.14.2 — General-zone design for slab tendons

For monostrand slab tendons, the general-zone minimum reinforcement requirements are based on the recommendations of ACI-ASCE Committee 423,[18.10] which shows typical details. The horizontal bars parallel to the edge required by 18.14.2.2 should be continuous where possible.

The tests on which the recommendations of Reference 18.24 were based were limited to anchorage devices for 1/2 in. diameter, 270 ksi strand, unbonded tendons in normalweight concrete. Thus, for larger strand anchorage devices and for all use in lightweight concrete slabs, ACI-ASCE Committee 423 recommended that the amount and spacing of reinforcement should be conservatively adjusted to provide for the larger anchorage force and smaller splitting tensile strength of lightweight concrete.[18.10]

Both References 18.10 and 18.22 recommend that hairpin bars also be furnished for anchorages located within 12 in. of slab corners to resist edge tension forces. The words "ahead of" in 18.14.2.3 have the meaning shown in Fig. R18.13.1.

In those cases where multistrand anchorage devices are used for slab tendons, 18.15 is applicable.

The bursting reinforcement perpendicular to the plane of the slab required by 18.14.2.3 for groups of relatively closely spaced tendons should also be provided in the case of widely spaced tendons if an anchorage device failure could cause more than local damage.

R18.14.3 — General-zone design for groups of monostrand tendons in beams and girders

Groups of monostrand tendons with individual monostrand anchorage devices are often used in beams and girders. Anchorage devices can be treated as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage device in the direction considered. If a beam or girder has a single anchorage device or a single group of closely spaced anchorage devices, the use of simplified equations such as those given in R18.13.5 is allowed, unless 18.13.5.2 governs. More complex conditions can be designed using strut-and-tie models. Detailed recommendations for use of such models are given in References 18.22 and 18.23 as well as in R18.13.5.

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18.15 — Design of anchorage zones for multistrand tendons

18.15.1 — Local zone design

Basic multistrand anchorage devices and local zone reinforcement shall meet the requirements of AASHTO "Standard Specification for Highway Bridges," Division I, Articles 9.21.7.2.2 through 9.21.7.2.4.

Special anchorage devices shall satisfy the tests required in AASHTO "Standard Specification for Highway Bridges," Division I, Article 9.21.7.3 and described in AASHTO "Standard Specification for Highway Bridges," Division II, Article 10.3.2.3.

18.15.2 — Use of special anchorage devices

Where special anchorage devices are to be used, supplemental skin reinforcement shall be furnished in the corresponding regions of the anchorage zone, in addition to the confining reinforcement specified for the anchorage device. This supplemental reinforcement shall be similar in configuration and at least equivalent in volumetric ratio to any supplementary skin reinforcement used in the qualifying acceptance tests of the anchorage device.

18.15.3 — General-zone design

Design for general zones for multistrand tendons shall meet the requirements of 18.13.3 through 18.13.5.

18.16 — Corrosion protection for unbonded single-strand prestressing tendons

18.16.1 — Unbonded prestressing steel shall be completely encased with sheathing. The prestressing steel shall be completely coated and sheathing around the prestressing steel filled with suitable material to inhibit corrosion.

18.16.2 — Sheathing shall be liquid-tight and continuous over entire length to be unbonded, and shall prevent intrusion of cement paste or loss of coating materials during concrete placement.

18.16.3 — The sheathing shall be connected to all stressing, intermediate and fixed anchorages in a

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R18.15 — Design of anchorage zones for multistrand tendons

R18.15.1 — Local zone design

See R18.13.2.

R18.15.2 — Use of special anchorage devices

Skin reinforcement is reinforcement placed near the outer faces in the anchorage zone to limit local crack width and spacing. Reinforcement in the general zone for other actions (flexure, shear, shrinkage, temperature, and similar) may be used in satisfying the supplementary skin reinforcement requirement. Determination of the supplementary skin reinforcement depends on the anchorage device hardware used and frequently cannot be determined until the shop-drawing stage.

R18.16 — Corrosion protection for unbonded single-strand prestressing tendons

R18.16.1 — For walls of noncircular liquid containment structures using unbonded tendons, consideration should be given to increasing the minimum reinforcement requirement of 14.3 and 18.9. The additional reinforcement is recommended to control cracking at service levels and reduce the potential for a sudden nonductile failure due to loss of prestress. Suitable material for corrosion protection of unbonded prestressing steel should have the properties identified in Section 5.1 of Reference 18.23.

R18.16.2 — Typically, sheathing is a continuous, seamless, high-density polythylene material that is extruded directly onto the coated prestressing steel.

As an additional aid for in-place field inspection of unbonded tendons, it is recommended that the color of the extruded sheathing be significantly different from the color of the corrosion protective grease. This will make visual inspection of the sheathing for damaged areas easier, because the grease extruded from a tear in the sheathing will be plainly evident on the contrasting color of the sheath.

R18.16.3 — A liquid-tight connection may be achieved either by using special connector pieces, which provide a

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liquid-tight fashion, thus providing a complete encapsulation of the prestressing steel from end to end. All voids in sleeves and caps shall be filled with a corrosion-protective material. Unbonded single-stand tendon systems shall meet the hydrostatic pressure testing requirements of ACI 423.6 except with a hydrostatic pressure of 10 psi, instead of the specified 1.25 psi.

18.16.4 — Unbonded single-strand tendons shall be protected against corrosion in accordance with the "aggressive environment" provisions of ACI 423.6 "Specification for Unbonded Single Strand Tendons and Commentary," except as noted above.

18.17 — Post-tensioning ducts

18.17.1 — Ducts for grouted tendons shall be mortar-tight and nonreactive with concrete, prestressing steel, grout, and corrosion inhibitor.

18.17.2 — Ducts for grouted single wire, single strand, or single bar tendons shall have an inside diameter at least 1/4 in. larger than the prestressing steel diameter.

18.17.3 — Ducts for grouted multiple wire, multiple strand, or multiple bar tendons shall have an inside cross-sectional area at least two times area of the prestressing steel.

18.17.4 — Ducts shall be maintained free of liquid if members to be grouted are exposed to temperatures below freezing prior to grouting.

18.18 — Grout for bonded tendons

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liquid-tight connection to the anchor at one end and the sheathing at the other end, or by other means meeting the liquid-tightness test performance criteria and proven to maintain liquid-tightness under field conditions. The 10 psi pressure corresponds to approximately a 20 ft head of water. More restrictive requirements for liquid-tightness may be specified for special applications where a high hydrostatic pressure is anticipated.

R18.16.4 — In the 2001 code, corrosion protection requirements for unbonded single strand tendons were added in accordance with the Post-Tensioning Institute's "Specification for Unbonded Single Strand Tendons." In the 2006 code, the reference changed to ACI 423.6. That specification included additional corrosion-protective measures for single-strand tendons used in aggressive environments. The additional provision on filling of voids in the end anchorage area is to prevent the possible accumulation of water and the associated corrosion in the anchorage area.

R18.17 — Post-tensioning ducts

R18.17.4 — Liquid in ducts may cause distress to the surrounding concrete upon freezing. When strands are present, liquid in ducts should also be avoided. A corrosion inhibitor should be used to provide temporary corrosion protection if prestressing steel is exposed to prolonged periods of moisture in the ducts before grouting.[18.24]

R18.18 — Grout for bonded tendons

Proper grout and grouting procedures are critical to post-tensioned construction.[18.25,18.26] Grout provides the bond between the prestressing steel and the duct, and provides corrosion protection of the prestressing steel.

Past success with grout for bonded tendons has been with portland cement. A blanket endorsement of all cementitious material (defined in 2.1) for use with this grout is deemed inappropriate because of a lack of experience or tests with cementitious materials other than portland cement and a concern that some cementitious materials might introduce chemicals listed as harmful to tendons in R18.18.2. Thus, "portland cement" in 18.18.1 and "water-cement ratio" in 18.18.3.3 are retained in the code.

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18.18.1 — Grout shall consist of portland cement and water; or portland cement, sand, and water; or a 100-percent-solids, two-component epoxy resin system.

18.18.2 — Materials for grout shall conform to 18.18.2.1 through 18.18.2.4

18.18.2.1 — Portland cement shall conform to 3.2.

18.18.2.2 — Water shall conform to 3.4.

18.18.2.3 — Sand, if used, shall conform to "Standard Specification for Aggregate for Masonry Mortar" (ASTM C 144) except that gradation shall be permitted to be modified as necessary to obtain satisfactory workability.

18.18.2.4 — Admixtures conforming to 3.6 and known to have no injurious effects on grout, steel, or concrete shall be permitted. Calcium chloride shall not be used.

18.18.2.5 — Epoxy grout shall be moisture insensitive with a minimum compressive strength of 125 percent of the design concrete compressive strength.

18.18.3 — Selection of grout proportions

18.18.3.1 — Proportions of materials for grout shall be based on either (a) or (b):

(a) Results of tests on fresh and hardened grout prior to beginning grouting operations; or

(b) Prior documented experience with similar materials and equipment and under comparable field conditions.

18.18.3.2 — Cement used in the work shall correspond to that on which selection of grout proportions was based.

18.18.3.3 — Water content shall be minimum necessary for proper pumping of grout; however, water-cement ratio shall not exceed 0.45 by weight.

18.18.3.4 — Water shall not be added to increase grout flowability that has been decreased by delayed use of grout.

18.18.3.5 — Epoxy grout shall have demonstrated by tests or experience to exhibit acceptable pumpability and low exothermic properties considering the geometric configuration of the prestressing steel and duct.

18.18.4 — Mixing and pumping grout

18.18.4.1 — Grout shall be mixed in equipment capable of continuous mechanical mixing and agitation that will produce uniform distribution of materials, passed through screens, and pumped in a manner that will completely fill tendon ducts.

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R18.18.1 — Epoxy grout has been used in limited applications. Caution is recommended in its selection and use. Properties of the material should be reviewed including differences in the coefficient of thermal expansion and heat generation.

R18.18.2 — The limitations on admixtures in 3.6 apply to grout. Substances known to be harmful to prestressing tendons, grout, or concrete are chlorides, fluorides, sulfites, and nitrates. Aluminum powder or other expansive admixtures, when approved, should produce an unconfined expansion of 5 to 10 percent. Neat cement grout is used in almost all building construction. Only with large ducts having large void areas should the advantages of using finely graded sand in the grout be considered. Admixtures are generally used to increase workability, reduce bleeding and shrinkage, or provide expansion. This is especially desirable for grouting of vertical tendons.

R18.18.3 — Selection of grout proportions

Grout proportioned in accordance with these provisions will generally lead to 7-day compressive strength on standard 2 in. cubes in excess of 2500 psi and 28-day strengths of about 4000 psi. The handling and placing properties of grout are usually given more consideration than strength when designing grout mixtures.

R18.18.4 — Mixing and pumping grout

In an ambient temperature of 35 °F, grout with an initial minimum temperature of 60 °F may require as much as 5 days to reach 800 psi. A minimum grout temperature of 60 °F is suggested because it is consistent with the recommended minimum temperature for concrete placed at an ambient

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18.18.4.2 — Temperature of members at time of grouting shall be above 35 °F and shall be maintained above 35 °F until field-cured 2 in. cubes of grout reach a minimum compressive strength of 800 psi.

18.18.4.3 — Grout temperatures shall not be above 90 °F during mixing and pumping.

18.19 — Protection for prestressing steel

Burning or welding operations in vicinity of prestressing steel shall be carefully performed, so that tendons are not subject to excessive temperatures, welding sparks, or ground currents.

18.20 — Application and measurement of prestressing force

18.20.1 — Prestressing force shall be determined by both of (a) and (b):

(a) Measurement of tendon elongation. Required elongation shall be determined from average load-elongation curves for the prestressing steel used;

(b) Observation of jacking force on a calibrated gage or load cell or by use of a calibrated dynamometer.

Cause of any difference in force determination between (a) and (b) that exceeds 5 percent for pretensioned elements or 7 percent for post-tensioned construction shall be ascertained and corrected.

18.20.2 — Where transfer of force from bulkheads of pretensioning bed to concrete is accomplished by flame cutting prestressing steel, cutting points and cutting sequence shall be predetermined to avoid undesired temporary stresses.

18.20.3 — Long lengths of exposed pretensioned strand shall be cut near the member to minimize shock to concrete.

18.20.4 — Total loss of prestress due to unreplaced broken prestressing steel shall not exceed 2 percent of total prestress.

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temperature of 35 °F. Quickset grouts, when approved, may require shorter periods of protection and the recommendations of the suppliers should be followed. Test cubes should be cured under temperature and moisture conditions as close as possible to those of the grout in the member. Grout temperatures in excess of 90 °F will lead to difficulties in pumping.

R18.20 — Application and measurement of prestressing force

R18.20.1 — Elongation measurements for prestressed elements should be in accordance with the procedures outlined in the "Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products," published by the Precast/Prestressed Concrete Institute.[18.27]

Section 18.18.1 of the ACI 318-89 code was revised to permit 7 percent tolerance in tendon force determined by gage pressure and elongation measurements for post-tensioned construction. Elongation measurements for post-tensioned construction are affected by several factors that are less significant, or that do not exist, for pretensioned elements. The friction along post-tensioning tendons may be affected to varying degrees by placing tolerances and small irregularities in profile due to concrete placement. The friction coefficients between the tendons and the duct are also subject to variation. The 5 percent tolerance that has appeared in the code since the ACI 318-63 code was proposed by ACI-ASCE Committee 423 in 1958,[18.3] and primarily reflected experience with production of pretensioned concrete elements. Because the tendons for pretensioned elements are usually stressed in air with minimal friction effects, the 5 percent tolerance for such elements was retained.

R18.20.4 — This provision applies to all prestressed concrete members. For cast-in-place post-tensioned slab systems, a "member" should be that portion considered as an element in the design, such as the joist and effective slab width in one-way joist systems, or the column strip or middle strip in two-way flat plate systems.

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18.21 — Post-tensioning anchorages and couplers

18.21.1 — Anchorages and couplers for bonded and unbonded tendons shall develop at least 95 percent of the specified breaking strength of the prestressing steel, when tested in an unbonded condition, without exceeding anticipated set. For bonded tendons, anchorages and couplers shall be located so that 100 percent of the specified breaking strength of the prestressing steel shall be developed at critical sections after the prestressing steel is bonded in the member.

18.21.2 — Couplers shall be placed in areas approved by the engineer and enclosed in housing long enough to permit necessary movements.

18.21.3 — In unbonded construction subject to repetitive loads, special attention shall be given to the possibility of fatigue in anchorages and couplers.

18.21.4—Anchorages, couplers, and end fittings shall be permanently protected against corrosion.

18.22 — External post-tensioning

18.22.1 — Post-tensioning tendons shall be permitted to be external to any concrete section of a member *provided they are adequately protected against corrosion*.

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R18.21 — Post-tensioning anchorages and couplers

R18.21.1 — In the 1986 interim ACI 318 code provisions, the separate provisions for strength of unbonded and bonded tendon anchorages and couplers presented in 18.19.1 and 18.19.2 of the ACI 318-83 code were combined into a single revised 18.19.1 covering anchorages and couplers for both unbonded and bonded tendons. Since the ACI 318-89 code revision, the required strength of the tendon-anchorage or tendon-coupler assemblies for both unbonded and bonded tendons, when tested in an unbonded state, is based on 95 percent of the specified breaking strength of the tendon material in the test. The tendon material should comply with the minimum provisions of the applicable ASTM specifications as outlined in 3.5.5. The specified strength of anchorages and couplers exceeds the maximum design strength of the tendons by a substantial margin, and, at the same time, recognizes the stress-riser effects associated with most available post-tensioning anchorages and couplers. Anchorage and coupler strength should be attained with a minimum amount of permanent deformation and successive set, recognizing that some deformation and set will occur in testing to failure. Tendon assemblies should conform to the 2 percent elongation requirements in ACI 301[18.28] and industry recommendations.[18.18] Anchorages and couplers for bonded tendons that develop less than 100 percent of the specified breaking strength of the tendon should be used only where the bond transfer length between the anchorage or coupler and critical sections equals or exceeds that required to develop the tendon strength. This bond length may be calculated by the results of tests of bond characteristics of untensioned prestressing strand,[18.29] or by bond tests on other tendon materials, as appropriate.

R18.21.3 — For discussion on fatigue loading, see Reference 18.30.

For detailed recommendations on tests for static and cyclic loading conditions for tendons and anchorage fittings of unbonded tendons, see Section 4.1.3 of Reference 18.10, and Section 15.2.2 of Reference 18.28.

R18.21.4 — For recommendations regarding protection, see Sections 4.2 and 4.3 of Reference 18.10, and Sections 3.4, 3.5, 6, and 8.3 of Reference 18.23. When unbonded tendons are used and additional corrosion protection of the anchorage is desired, the anchorages may be fully encapsulated by plastic coatings.

R18.22 — External post-tensioning

External attachment of tendons is a versatile method of providing additional strength, or improving serviceability, or both, in existing structures. It is well suited to repair or

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18.22.2 — External tendons shall be considered as unbonded tendons when computing flexural strength unless provisions are made to effectively bond the external tendons to the concrete section along its entire length.

18.22.3 — External tendons shall be attached to the concrete member in a manner that maintains the desired eccentricity between the tendons and the concrete centroid throughout the full range of anticipated member deflection.

18.22.4 — The details of the corrosion protection method shall be indicated in the contract documents.

COMMENTARY

upgrade existing structures and permits a wide variety of tendon arrangements.

Additional information on external post-tensioning is given in Reference 18.31.

R18.22.3 — External tendons are often attached to the concrete member at various locations between anchorages (such as midspan, quarter points, or third points) for desired load balancing effects, for tendon alignment, or to address tendon vibration concerns. Consideration should be given to the effects caused by the tendon profile shifting in relationship to the concrete centroid as the member deforms under effects of post-tensioning and applied load.

R18.22.4 — Permanent corrosion protection can be achieved by a variety of methods. The corrosion protection provided should be suitable to the environment in which the tendons are located. Some conditions will require that the prestressing steel be protected by concrete or shotcrete cover or by cement grout in polyethylene or metal tubing; other conditions will permit the protection provided by durable coatings such as paint, grease, epoxy, or hot-dip galvanizing. Corrosion protection methods should meet the fire protection requirements of the general building code, unless the installation of external post-tensioning is to only improve serviceability.

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Notes

CHAPTER 19 — SHELLS AND FOLDED PLATE MEMBERS

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19.0 — Notation

E_c = modulus of elasticity of concrete, psi. See 8.5.1
f_c' = specified compressive strength of concrete, psi
$\sqrt{f_c'}$ = square root of specified compressive strength of concrete, psi
f_y = specified yield strength of nonprestressed reinforcement, psi
h = thickness of shell or folded plate, in.
ℓ_d = development length, in.
ϕ = strength reduction factor. See 9.3

19.1 — Scope and definitions

19.1.1 — Provisions of Chapter 19 shall apply to thin shell and folded plate concrete structures, including ribs and edge members, but do not apply to special structures such as cooling towers and circular prestressed concrete tank construction.

COMMENTARY

R19.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip.

R19.1 — Scope and definitions

The code and commentary provide information on the design, analysis, and construction of concrete thin shells and folded plates. The process began in 1964 with the publication of a practice and commentary by ACI Committee 334, and continued with the inclusion of Chapter 19 in the ACI 318-71 code and in later editions. The ACI 334-64 was revised and published as ACI 334R.1-82 and then as ACI 334.1R-92.[19.1] The revisions reflected additional experience in design, analysis, and construction gained since the earlier publications, and were influenced by the publication of the **"Recommendations for Reinforced Concrete Shells and Folded Plates"** of the International Association for Shell and Spatial Structures (IASS) in 1979.[19.2]

Because Chapter 19 applies to concrete thin shells and folded plates of all shapes, extensive discussion of their design, analysis, and construction in the commentary is not possible. Additional information can be obtained from the references listed for this chapter, which are provided for the assistance of the designer. They are not an official part of the code. The designer is responsible for their interpretation and use. Performance of shells and folded plates requires special attention to detail.[19.3]

R19.1.1 — Chapter 19 is intended to apply to thin shells and folded plate concrete structures in building construction subjected to an environmental exposure.

Concrete crack control is essential to prevent deterioration and corrosion of reinforcement in structures with environmental exposures. Therefore, use only those types of thin shells and folded plates where considerable information and experience have been documented concerning cracking behavior, and where crack control can be provided at service loads through proper design, analysis, and construction.

Discussion of the application of thin shells in special structures such as cooling towers and circular prestressed concrete tanks may be found in the reports of ACI-ASCE Committee 334[19.4] and ACI Committees 372[19.5] and 373.[19.6]

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19.1.2 — All provisions of this code not specifically excluded, and not in conflict with provisions of Chapter 19 shall apply to thin-shell structures.

19.1.3 — ***Thin shells*** — three-dimensional spatial structures made up of one or more curved slabs or folded plates whose thicknesses are small compared to their other dimensions. Thin shells are characterized by their three-dimensional load-carrying behavior that is determined by the geometry of their forms, by the manner in which they are supported, and by the nature of the applied load.

19.1.4 — ***Folded plates*** — a special class of shell structures formed by joining flat, thin slabs along their edges to create a three-dimensional spatial structure.

19.1.5 — ***Ribbed shells*** — spatial structures with material placed primarily along certain preferred rib lines, with the area between the ribs filled with thin slabs or left open.

19.1.6 — ***Auxiliary members*** — ribs or edge beams that serve to strengthen, stiffen, or support the shell; usually, auxiliary members act jointly with the shell.

19.1.7 — ***Elastic analysis*** — an analysis of deformations and internal forces based on equilibrium, compatibility of strains, and assumed elastic behavior, and representing to a suitable approximation the three-dimensional action of the shell together with its auxiliary members.

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R19.1.3 — Common types of thin shells are domes (surfaces of revolution),[19.7,19.8] cylindrical shells,[19.8] barrel vaults,[19.9] conoids,[19.9] elliptical paraboloids,[19.9] hyperbolic paraboloids,[19.10] and groined vaults.[19.10] Considerable information on the experience gained in the design, analysis, and construction of these shells may be found in the cited references. Less experience is available regarding other shell types or shapes, including free-form shells.

R19.1.4 — Folded plates may be prismatic,[19.8,19.11] nonprismatic,[19.11] or faceted. The first two types consist generally of planar thin slabs joined along their longitudinal edges to form a beam-like structure spanning between supports. Faceted folded plates are made up of triangular and/or polygonal planar thin slabs joined along their edges to form three-dimensional spatial structures.

R19.1.5 — Ribbed shells[19.7,19.12] generally have been used for larger spans where the increased thickness of the curved slab alone becomes excessive or uneconomical. Ribbed shells also have been used because of the construction techniques employed and to enhance the aesthetic impact of the completed structure.

R19.1.6 — Most thin-shell structures require ribs or edge beams at their boundaries to carry the shell boundary forces, to assist in transmitting them to the supporting structure, and to accommodate the increased amount of reinforcement in these areas.

R19.1.7 — Elastic analysis of thin shells and folded plates means any method of structural analysis that is based on assumptions that provide suitable approximations to the three-dimensional behavior of the structure. The method should determine the internal forces and displacements needed in the design of the shell proper, the rib or edge members, and the supporting structure. Equilibrium of internal forces and external loads and compatibility of deformations should be satisfied.

Methods of elastic analysis based on classical shell theory, simplified mathematical or analytical models, or numerical solutions using finite element,[19.10] finite differences,[19.7] or numerical integration techniques,[19.7] are described in the cited references.

The choice of the method of analysis and the degree of accuracy required depends on certain critical factors. These include: the size of the structure, the geometry of the thin shell or folded plate, the manner in which the structure is supported, the nature of the applied load, and, finally, the extent of personal or documented experience regarding the reliability of the given method of analysis in predicting the behavior of the specific type of shell[19.7] or folded plate.[19.11]

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19.1.8 — *Inelastic analysis* — an analysis of deformations and internal forces based on equilibrium, nonlinear stress-strain relations for concrete and reinforcement, consideration of cracking and time dependent effects, and compatibility of strains. The analysis shall represent to a suitable approximation three-dimensional action of the shell together with its auxiliary members.

19.1.9 — *Experimental analysis* — an analysis procedure based on the measurement of deformations or strains, or both, of the structure or its model; experimental analysis is based on either elastic or inelastic behavior.

19.2 — Analysis and design

19.2.1 — Elastic behavior shall be an accepted basis for determining internal forces and displacements of thin shells. This behavior shall be permitted to be established by computations based on an analysis of the uncracked concrete structure in which the material is assumed linearly elastic, homogeneous, and isotropic. Poisson's ratio of concrete shall be permitted to be taken equal to zero.

19.2.2 — Inelastic analyses shall not be permitted to be used unless it can be shown that such analysis methods provide a safe basis for design and provide for the necessary crack control for environmental engineering concrete structures at service loads.

19.2.3 — Equilibrium checks of internal resistances and external loads shall be made to ensure consistency of results.

19.2.4 — Experimental or numerical analysis procedures shall be permitted where it can be shown that such procedures provide a safe basis for design and provide for the necessary crack control for environmental engineering concrete structures at service loads.

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R19.1.8—Inelastic analysis of thin shells and folded plates can be performed using a refined method of analysis based on the specific nonlinear material properties, nonlinear behavior due to the cracking of concrete, and time-dependent effects such as creep, shrinkage, temperature, and load history. These effects are incorporated to trace the response and crack propagation of a reinforced concrete shell through the elastic, inelastic, and ultimate ranges. Such analyses usually require incremental loading and iterative procedures to converge on solutions that satisfy both equilibrium and strain compatibility.[19.13,19.14]

R19.2 — Analysis and design

R19.2.1 — For types of shell structures where experience, tests, and analyses have shown that the structure can sustain reasonable overloads without undergoing brittle failure, elastic analysis is a generally acceptable procedure. The designer may assume that reinforced concrete is ideally elastic, homogeneous, and isotropic, having identical properties in all directions. An analysis should be performed for the shell considering service-load conditions. The analysis of shells of unusual size, shape, or complexity should consider behavior through the elastic, cracking ranges, and inelastic ranges.

R19.2.2 — Because inelastic design assumes cracked sections and the addition of the environmental durability factor may not assure crack control for calculations of this complexity, inelastic design should be used only where it can be clearly demonstrated that cracking can be controlled.

Inelastic analysis procedures will generally require extensive use of computer procedures. References 19.13 and 19.14 indicate possible solution methods.

R19.2.4 — Experimental analysis of elastic models[19.15] has been used as a substitute for an analytical solution of a complex shell structure. Experimental analysis of reinforced microconcrete models through the elastic, cracking, inelastic, and ultimate ranges should be considered for important shells of unusual size, shape, and/or complexity.

For model analysis, only those portions of the structure that affect significantly the items under study need be simulated. Every attempt should be made to ensure that the experiments reveal the quantitative behavior of the prototype structure.

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Wind tunnel tests of a scaled-down model do not necessarily provide usable results, and should be conducted by a recognized expert in wind tunnel testing of structural models.

19.2.5 — Approximate methods of analysis not satisfying compatibility of strains either within the shell or between the shell and auxiliary members, or both, shall be permitted where it can be shown that such methods provide a safe basis for design and provide for the necessary crack control for environmental engineering concrete structures at service loads.

R19.2.5 — Solutions that include both membrane and bending effects and satisfy compatibility of strains and equilibrium are preferred. Approximate solutions that satisfy statics but not the compatibility of strains may be used only when extensive experience has proven that safe designs with crack control necessary to minimize the potential for corrosion have resulted from their use. Such methods include beam-type analysis for barrel shells and folded plates having large ratios of span to either width or radius of curvature, simple membrane analysis for shells of revolution, and others in which the equations of equilibrium are satisfied, while the strain compatibility equations are not. In complex structures where several shells join together, or where shells join auxiliary members, however, a more accurate analysis should be used. Also, a more accurate analysis is required where approximate solutions cannot assure crack control necessary for environmental engineering concrete structures at service loads.

19.2.6 — In prestressed shells, the analysis shall also consider behavior under loads induced during prestressing, at cracking load, and at factored load. Where tendons are draped within a shell, design shall take into account force components on the shell resulting from the tendon profile not lying in one plane.

R19.2.6 — If the shell is prestressed, the analysis must include its strength at factored loads as well as its adequacy under service loads, under the load that causes cracking, and under loads induced during prestressing. Axial forces due to draped tendons may not lie in one plane, and due consideration must be given to the resulting force components. The effects of post-tensioning of supporting members on the shell must be taken into account.

19.2.7 — The thickness of a thin shell or folded plate, and its reinforcement, shall be proportioned for the required strength, concrete cover over reinforcement, durability, and serviceability. All elements shall be proportioned by the same method, using either the strength design method of 8.1.1 or the alternate design method of 8.1.2.

The minimum shell or plate thickness shall be 4 in. and minimum reinforcing bar size shall be No. 4.

The provisions of Chapter 16 shall apply if elements are precast. If composite action is involved, the provisions of Chapter 17 shall be satisfied.

R19.2.7 — The thin shell's thickness and reinforcement are required to be proportioned to satisfy the strength provisions of this code, and to resist internal forces obtained from an analysis, an experimental study, or a combination thereof. The thickness of the shell is often dictated by the limitation of deflection of edge members by the requirements of 19.2.8, by required reinforcement cover, by being subjected to an environmental exposure, and by constraints of construction.

The necessary thickness and reinforcement may also be provided by using the alternate design method prescribed in 8.1.2. The design method chosen should be used consistently throughout the structure.

When shell or folded plate elements are precast and connected by cast-in-place segments, composite action is involved.

19.2.8 — Shell instability shall be investigated and shown by design to be precluded.

R19.2.8 — Thin shells, like other structures that experience in-plane membrane compressive forces, are subject to buckling when the applied load reaches a critical value. Because of the surface-like geometry of shells, the problem of calculating buckling load is complex. If one of the principal membrane forces is tensile, the shell is less likely to buckle than if both principal membrane forces are compressive. The kinds of membrane forces that develop in a shell depend on its initial shape and the manner in which the shell is supported and loaded. In some types of shells, post-buckling

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behavior must be considered in determining safety against instability.[19.2]

Investigation of thin shells for stability should consider the effect of the following factors: (1) anticipated deviation of the geometry of the shell surface as built from the idealized geometry; (2) large deflections; (3) creep and shrinkage of concrete; (4) inelastic properties of materials; (5) cracking of concrete; (6) location, amount, and orientation of reinforcement; and (7) possible deformation of supporting elements.

Measures to improve resistance to buckling successfully used in the past include the provision of two mats of reinforcement—one near each outer surface of the shell, a local increase of shell curvatures, the use of ribbed shells, and the use of concrete with high tensile strength and low creep.

A procedure for determining critical buckling loads of shells is given in the IASS recommendations.[19.2] Some recommendations for buckling design of domes used in industrial applications are given in References 19.5 and 19.16.

19.2.9 — Auxiliary members shall be designed according to the applicable provisions of this code. It shall be permitted to assume that a portion of the shell equal to the flange width, as specified in 8.10, acts with the auxiliary member. In such portions of the shell, the reinforcement perpendicular to the auxiliary member shall be at least equal to that required for the flange of a T-beam by 8.10.5.

R19.2.9 — Strength design can be used for the auxiliary members even though the alternate design method was used for the shell surface as long as serviceability requirements are also met. Portions of the shell may be utilized as flanges for transverse or longitudinal frames or arch-frames and beams.

19.2.10 — Strength design of shell slabs for membrane and bending forces shall be based on the distribution of stresses and strains as determined from either an elastic or an inelastic analysis.

R19.2.10 — The stresses and strains in the shell slab used for design are those determined by analysis (elastic or inelastic) multiplied by appropriate load factors. Because of detrimental effects of membrane cracking, the computed tensile strain in the reinforcement under factored loads should be limited.

19.2.11 — In a region where membrane cracking is predicted, the nominal compressive strength parallel to the cracks shall be taken as $0.4f_c'$.

R19.2.11 — When principal tensile stress produces membrane cracking in the shell, experiments indicate the attainable compressive strength in the direction parallel to the cracks is reduced.[19.17,19.18] For the alternate design method, the compressive strength f_c' parallel to the cracks should be replaced by $0.4f_c'$ in calculations involving I.3.1(a) or I.6.1.

19.3 — Design strength of materials

R19.3 — Design strength of materials

19.3.1 — Specified compressive strength of concrete f_c' at 28 days shall not be less than 3000 psi.

R19.3.1 — See Chapter 4 for minimum 28-day compressive strength f_c' requirements for concrete that is in contact with various exposure conditions.

19.3.2 — Specified yield strength of nonprestressed reinforcement f_y shall not exceed 60,000 psi.

19.4 — Shell reinforcement

R19.4 — Shell reinforcement

19.4.1 — Shell reinforcement shall be provided to resist tensile stresses from internal membrane forces,

R19.4.1 — At any point in a shell, two different kinds of internal forces may occur simultaneously: those associated

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to resist tension from bending and twisting moments, to limit shrinkage and temperature crack width and spacing, and as special reinforcement at shell boundaries, load attachments, and shell openings.

19.4.2 — Tensile reinforcement shall be provided in two or more directions and shall be proportioned such that its resistance in any direction equals or exceeds the component of internal forces in that direction.

Alternatively, reinforcement for the membrane forces in the slab shall be calculated as the reinforcement required to resist axial tensile forces plus the tensile force due to shear-friction required to transfer shear across any cross section of the membrane. The assumed coefficient of friction shall not exceed **1.0**λ where λ **= 1.0** for normalweight concrete, 0.85 for "sand-lightweight" concrete, and 0.75 for "all-lightweight" concrete. Linear interpolation shall be permitted when partial sand replacement is used.

19.4.3 — The area of shell reinforcement at any section as measured in two orthogonal directions shall not be less than the slab reinforcement required by 7.12 except the minimum area of reinforcement shall be not less than 0.0028 times the cross-sectional area.

19.4.4 — Reinforcement for shear and bending moments about axes in the plane of the shell slab shall be calculated in accordance with Chapters 10, 11, and 13.

19.4.5 — The area of shell tension reinforcement shall be limited so that the reinforcement will yield before either crushing of concrete in compression or shell buckling can take place.

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with membrane action, and those associated with bending of the shell. The membrane forces are assumed to act in the tangential plane midway between the surfaces of the shell, and are the two axial forces and the membrane shears. Flexural effects include bending moments, twisting moments, and the associated transverse shears. Control of membrane cracking due to shrinkage, temperature, and service load conditions is a major design consideration.

R19.4.2 — The requirement of ensuring strength in all directions is based on safety considerations. Any method that assures sufficient strength consistent with equilibrium is considered acceptable. The direction of the principal membrane tensile force at any point may vary depending on the direction, magnitudes, and combinations of the various applied loads.

The magnitude of the internal membrane forces, acting at any point due to a specific load, is generally calculated on the basis of an elastic theory in which the shell is assumed as uncracked. The computation of the required amount of reinforcement to resist the internal membrane forces has been traditionally based on the assumption that concrete cannot resist tension. The associated deflections, and the possibility of cracking, should be investigated in the serviceability phase of the design. Achieving this may require a working stress design for steel selection.

Where reinforcement is not placed in the direction of the principal tensile forces and where cracks at the service load level would be objectionable, the computation of reinforcement may have to be based on a more refined approach[19.17,19.20] that considers the existence of cracks. In the cracked state, the concrete is assumed to be unable to resist either tension or shear. Thus, equilibrium is attained by means of tensile resisting forces in reinforcement and compressive resisting forces in concrete.

The alternative method to calculate orthogonal reinforcement is the shear-friction method. It is based on the assumption that shear integrity of a shell should be maintained at factored loads. It is not necessary to calculate principal stresses if the alternative approach is used.

R19.4.3 — Minimum membrane reinforcement corresponding to slab shrinkage and temperature reinforcement are to be provided in at least two approximately orthogonal directions even if the calculated membrane forces are compressive in one or more directions.

R19.4.5 — The requirement that the tensile reinforcement yields before the concrete crushes anywhere is consistent with 10.3.3. Such crushing can also occur in regions near supports and for some shells where the principal membrane forces are approximately equal and opposite in sign.

19.4.6 — In regions of high tension, membrane reinforcement shall, if practical, be placed in the general directions of the principal tensile membrane forces. Where this is not practical, it shall be permitted to place membrane reinforcement in two or more component directions.

R19.4.6 — Generally, for all shells, and particularly in regions of substantial tension, the orientation of reinforcement approximates the directions of the principal tensile membrane forces. In some structures, however, it is not always possible or practical for the reinforcement to follow the stress trajectories. For such cases, orthogonal component reinforcement is allowed.

19.4.7 — If the direction of reinforcement varies more than 10 deg from the direction of principal tensile membrane force, the amount of reinforcement shall be reviewed in relation to cracking at service loads.

R19.4.7 — When the directions of reinforcement deviate significantly (more than 10 deg) from the directions of the principal membrane forces, higher strains in the shell occur to develop the capacity of the reinforcement. This might lead to the development of unacceptably wide cracks. The crack width should be estimated and controlled if necessary.

Permissible crack widths for service loads under different environmental conditions are given in the report of ACI Committee 224.[19.21] Crack control considerations for environmental structures are contained in 10.6. Crack width can be limited by an increase in the amount of reinforcement used by reducing the stress at the service load level, by providing reinforcement in three or more directions in the plane of the shell, or by using a closer spacing of smaller-diameter bars rather than a larger spacing of larger-diameter bars.

19.4.8 — Where the magnitude of the principal tensile membrane stress within the shell varies greatly over the area of the shell surface, reinforcement resisting the total tension shall be permitted to be concentrated in the regions of largest tensile stress where it can be shown that this provides a safe basis for design. However, the ratio of shell reinforcement in any portion of the tensile zone shall be not less than 0.0035 based on the overall thickness of the shell.

R19.4.8 — The practice of concentrating tensile reinforcement in the regions of maximum tensile stress has led to a number of successful and economical designs, primarily for long folded plates, barrel vault shells, and for domes. The requirement of providing the minimum reinforcement in the remaining tensile zone is intended to control cracking.

19.4.9 — Reinforcement required to resist shell bending moments shall be proportioned with due regard to the simultaneous action of membrane axial forces at the same location. Where shell reinforcement is required in only one face to resist bending moments, equal amounts shall be placed near both surfaces of the shell even though a reversal of bending moments is not indicated by the analysis.

R19.4.9 — The design method should assure that the concrete sections, including consideration of the reinforcement, are capable of developing the internal forces required to assure the equations of equilibrium are satisfied.[19.22] The sign of bending moments may change rapidly from point to point of a shell. For this reason, bending reinforcement, where required, is to be placed near both outer surfaces of the shell. In many cases, the thickness required to provide proper cover and spacing for the multiple layers of reinforcement may govern the design of the shell thickness.

19.4.10 — Shell reinforcement in any direction shall not be spaced farther apart than 12 in. nor farther apart than three times the shell thickness.

R19.4.10 — The value of ϕ to be used is that prescribed in 9.3.2.1 for axial tension.

19.4.11 — Shell reinforcement at the junction of the shell and supporting members or edge members shall be anchored in or extended through such members in accordance with the requirements of Chapter 12, except that the minimum development length shall be $1.2\ell_d$ but not less than 18 in.

R19.4.11 and R19.4.12 — On curved shell surfaces, it is difficult to control the alignment of precut reinforcement. This must be considered to avoid insufficient splice and development lengths. Sections 19.4.11 and 19.4.12 specify extra reinforcement length to maintain the minimum lengths on curved surfaces.

19.4.12 — Splice lengths of shell reinforcement shall be governed by the provisions of Chapter 12, except that the minimum splice length of tension bars shall be

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1.2 times the value required by Chapter 12 but not less than 18 in. The number of splices in principal tensile reinforcement shall be kept to a practical minimum. Where splices are necessary they shall be staggered at least ℓ_d with not more than one-third of the reinforcement spliced at any section.

19.5 — Construction

19.5.1 — When removal of formwork is based on a specific modulus of elasticity of concrete because of stability or deflection considerations, the value of the modulus of elasticity E_c shall be determined from flexural tests of field-cured beam specimens. The number of test specimens, the dimensions of test beam specimens, and test procedures shall be specified by the engineer.

19.5.2 — The engineer shall specify the tolerances for the shape of the shell. If construction results in deviations from the shape greater than the specified tolerances, an analysis of the effect of the deviations shall be made and any required remedial actions shall be taken to ensure safe behavior.

COMMENTARY

R19.5 — Construction

R19.5.1 — When early removal of forms is necessary, the magnitude of the modulus of elasticity at the time of proposed form removal must be investigated in order to ensure safety of the shell with respect to buckling, and to restrict deflections.[19.3,19.23] The value of the modulus of elasticity E_c should be obtained from a flexural test of field-cured specimens. It is not sufficient to determine the modulus from the formula in 8.5.1, even if f_c' is determined for the field-cured specimen.

R19.5.2 — In some types of shells, small local deviations from the theoretical geometry of the shell can cause relatively large changes in local stresses and in overall safety against instability. These changes can result in local cracking and yielding that may make the structure unsafe, cause corrosion of reinforcing steel, deterioration of concrete, or can greatly affect the critical load producing instability. The effect of such deviations should be evaluated, and any necessary remedial actions should be taken.

PART 6 — SPECIAL CONSIDERATIONS

CHAPTER 20 — STRENGTH EVALUATION OF EXISTING STRUCTURES

CODE

20.0 — Notation

D = dead loads or related internal moments and force

f_c' = specified compressive strength of concrete, psi

h = overall thickness of member, in.

L = live loads or related internal moments and force

ℓ_t = span of member under load test, in. (The shorter span for two-way slab systems.) Span is the smaller of: (a) distance between centers of supports; and (b) clear distance between supports plus thickness h of member. In Eq. (20-1), span for a cantilever shall be taken as twice the distance from support to cantilever end

Δ_{max} = measured maximum deflection, in. See Eq. (20-1)

Δ_{rmax} = measured residual deflection, in. See Eq. (20-2) and (20-3)

Δ_{fmax} = maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test, in. See Eq. (20-3)

20.1 — Strength evaluation — General

20.1.1 — If there is doubt that a part or all of a structure meets the safety requirements of this code, a strength evaluation shall be carried out as required by the engineer or building official.

COMMENTARY

R20.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip.

R20.1 — Strength evaluation — General

Chapter 20 does not cover load testing for the approval of new design or construction methods. (See 16.10 for recommendations on strength evaluation of precast concrete members.) Provisions of Chapter 20 may be used to evaluate whether a structure or a portion of a structure satisfies the safety requirements of this code. A strength evaluation may be required if the materials are considered to be deficient in quality, if there is evidence indicating faulty construction, if a structure has deteriorated, if a structure will be used for a new function, or if, for any reason, a structure or a portion of it does not appear to satisfy the requirements of the code. In such cases, Chapter 20 provides guidance for investigating the safety of the structure.

The demonstration of adequate strength in itself does not indicate that an environmental structure is adequate with regard to serviceability and durability. This chapter does not address acceptance criteria for serviceability and durability.

If the safety concerns are related to an assembly of elements or an entire structure, it is not feasible to load test every element and section to the maximum. In such cases, it is appropriate that an investigation plan be developed to address the specific safety concerns. If a load test is described as part of the strength evaluation process, it is desirable for all parties involved to come to an agreement about the region to be loaded, the magnitude of the load, the load test procedure, and acceptance criteria before any load tests are conducted.

20.1.2 — If the effect of the strength deficiency is well understood and if it is feasible to measure the dimensions and material properties required for analysis, analytical evaluations of strength based on those measurements shall suffice. Required data shall be determined in accordance with 20.2.

R20.1.2 — Strength considerations related to axial load, flexure, and combined axial load and flexure are well understood. There are reliable theories relating strength and short-term displacement to load in terms of dimensional and material data for the structure.

To determine the strength of the structure by analysis, calculations should be based on data gathered on the actual dimensions of the structure, properties of the materials in place, and all pertinent details. Requirements for data collection are in 20.2.

20.1.3 — If the effect of the strength deficiency is not well understood or if it is not feasible to establish the required dimensions and material properties by measurement, a load test shall be required if the structure is to remain in service.

R20.1.3 — If the shear or bond strength of an element is critical in relation to the doubt expressed about safety, a physical test may be the most efficient solution to eliminate or confirm the doubt. A physical test may also be appropriate if it is not possible or feasible to determine the material and dimensional properties required for analysis even if the cause of the concern relates to flexure or axial load.

Wherever possible and appropriate, it is desirable to support the results of the load test by analysis.

20.1.4 — If the doubt about safety of a part or all of a structure involves deterioration, and if the observed response during the load test satisfies the acceptance criteria, the structure or part of the structure shall be permitted to remain in service for a specified time period. If deemed necessary by the engineer, periodic reevaluations shall be conducted.

R20.1.4 — For a deteriorating structure, the acceptance provided by the load test may not be assumed to be without limits in terms of time. In such cases, a periodic inspection program is useful. A program that involves physical tests and periodic inspection can justify a longer period in service. Another option for maintaining the structure in service, while the periodic inspection program continues, is to limit the live load to a level determined to be appropriate.

The length of the specified time period should be based on consideration of: (a) the nature of the problem; (b) environmental and load effects; (c) service history of the structure; and (d) scope of the periodic inspection program. At the end of a specified time period, further strength evaluation is required if the structure is to remain in service.

With the agreement of all concerned parties, special procedures may be devised for periodic testing that do not necessarily conform to the loading and acceptance criteria specified in Chapter 20.

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20.2 — Determination of required dimensions and material properties

20.2.1 — Dimensions of the structural elements shall be established at critical sections.

20.2.2 — Locations and sizes of the reinforcing bars, welded wire fabric, or tendons shall be determined by measurement. It shall be permitted to base reinforcement locations on available drawings if spot checks are made confirming the information on the drawings.

20.2.3 — If required, concrete strength shall be based on results of cylinder tests or tests of cores removed from the part of the structure where the strength is in doubt. Concrete strengths shall be determined as specified in 5.5.4.

20.2.4 — If required, reinforcement or prestressing steel strength shall be based on tensile tests of representative samples of the material in the structure in question.

20.2.5 — If the required dimensions and material properties are determined through measurements and testing, and if calculations can be made in accordance with 20.1.2, it shall be permitted to increase the strength reduction factor in 9.3, but the strength reduction factor shall not be more than:

Tension-controlled sections, as defined in 10.3.4 .... 1.0

Compression-controlled sections, as defined in 10.3.3

Members with spiral reinforcement conforming to 10.9.3 .... 0.85

Other reinforced members 0.8

Shear and/or torsion 0.8

Bearing on concrete 0.8

20.3 — Load test procedure

20.3.1 — Load arrangement

The number and arrangement of spans or panels loaded shall be selected to maximize the deflection and stresses in the critical regions of the structural elements of which strength is in doubt. More than one test load arrangement shall be used if a single

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R20.2 — Determination of required dimensions and material properties

This section applies if it is decided to make an analytical evaluation (see 20.1.2).

R20.2.1 — Critical sections are those at which each type of stress calculated for the load in question reaches its maximum value.

R20.2.2 — For individual elements, amount, size, arrangement, and location must be determined at the critical sections for reinforcement and/or tendons designed to resist applied load. Nondestructive investigation methods are acceptable. In large structures, determination of these data for approximately 5 percent of the reinforcement or tendons in critical regions may suffice if these measurements confirm the data provided in the construction drawings.

R20.2.3 — The number of tests may depend on the size of the structure and the sensitivity of structural safety to concrete strength. In cases where the potential problem involves flexure only, investigation of concrete strength can be minimal for a lightly reinforced section ($\rho f_y / f_c' \leq 0.15$ for rectangular section).

R20.2.4 — The number of tests required depends on the uniformity of the material and is best determined by the engineer for the specific application.

R20.2.5 — Strength reduction factors given in 20.2.5 are larger than those specified in Chapter 9. These increased values are justified by the use of accurate field-obtained material properties, actual in-place dimensions, and well understood methods of analysis.

The strength reduction factors in 20.2.5 were changed for the 2006 edition to be compatible with the load combinations and strength reduction factors of Chapter 9, which were revised at that time.

R20.3 — Load test procedure

R20.3.1 — Load arrangement

It is important to apply the load at locations so that its effects on the suspected defect are a maximum and the probability of unloaded members sharing the applied load is a minimum. In cases where it is shown by analysis that adjoining unloaded elements will help carry some of the load, the load must be placed to develop effects consistent with the intent of the load factor.

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arrangement will not simultaneously result in maximum values of the effects (such as deflection, rotation, or stress) necessary to demonstrate the adequacy of the structure.

20.3.2 — Load intensity

The total test load (including dead load already in place) shall not be less than $\mathbf{0.85(1.4\mathit{D} + 1.7\mathit{L})}$. It shall be permitted to reduce $\boldsymbol{L}$ in accordance with the requirements of the applicable general building code.

20.3.3 — A load test shall not be made until that portion of the structure to be subject to load is at least 56 days old. If the owner of the structure, the contractor, and all involved parties agree, it shall be permitted to make the test at an earlier age.

20.4 — Loading criteria

20.4.1 — The initial value for all applicable response measurements (such as deflection, rotation, strain, slip, crack widths) shall be obtained not more than 1 hour before application of the first load increment. Measurements shall be made at locations where maximum response is expected. Additional measurements shall be made if required.

20.4.2 — Test load shall be applied in not less than four approximately equal increments.

20.4.3 — Uniform test load shall be applied in a manner to ensure uniform distribution of the load transmitted to the structure or portion of the structure being tested. Arching of the applied load shall be avoided.

20.4.4 — A set of response measurements shall be made after each load increment is applied and after the total load has been applied on the structure for at least 24 hours.

20.4.5 — Total test load shall be removed immediately after all response measurements defined in 20.4.4 are made.

20.4.6 — A set of final response measurements shall be made 24 hours after the test load is removed.

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R20.3.2 — Load intensity

The required load intensity follows previous load test practice. The live load $\boldsymbol{L}$ may be reduced as permitted by the general building code governing safety considerations for the structure. The live load should be increased to compensate for resistance provided by unloaded portions of the structure in questions. The increase in live load is determined from analysis of the loading conditions in relation to the selected pass/fail criterion for the test.

Although the load combinations and strength reduction factors of Chapter 9 were revised for the 2006 edition, the test load intensity remained the same. It is considered appropriate for designs using the load combinations and strength reduction factors of Chapter 9 or Appendix C.

R20.4 — Loading criteria

R20.4.2 — It is advisable to inspect the structure after each load increment.

R20.4.3 — "Arching" refers to the tendency for the load to be transmitted nonuniformly to the flexural element being tested. For example, if a slab is loaded by a uniform arrangement of bricks with the bricks in contact, arching would result in reduction of the load on the slab near the midspan of the slab.

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20.5 — Acceptance criteria

20.5.1 — The portion of the structure tested shall show no evidence of failure. Spalling and crushing of compressed concrete shall be considered an indication of failure.

20.5.2 — Measured maximum deflections shall satisfy one of the following conditions

$$\Delta_{max} \leq \frac{\ell_t^2}{20{,}000h} \quad (20\text{-}1)$$

$$\Delta_{rmax} \leq \frac{\Delta_{max}}{4} \quad (20\text{-}2)$$

If the measured maximum and residual deflections do not satisfy Eq. (20-1) or (20-2), it shall be permitted to repeat the load test.

The repeat test shall be conducted not earlier than 72 hours after removal of the first test load. The portion of the structure tested in the repeat test shall be considered acceptable if deflection recovery satisfies the condition

$$\Delta_{rmax} \leq \frac{\Delta_{fmax}}{5} \quad (20\text{-}3)$$

where Δ_{fmax} is the maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test.

20.5.3 — Structural members tested shall not have cracks indicating the imminence of shear failure.

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R20.5 — Acceptance criteria

R20.5.1 — A general acceptance criterion for the behavior of a structure under the test load is that it shall not show "evidence of failure." Evidence of failure includes cracking, spalling, or deflection of such magnitude and extent that the observed result is obviously excessive and incompatible with the safety requirements of the structure. No simple rules can be developed for application to all types of structures and conditions. If sufficient damage has occurred so that the structure is considered to have failed that test, retesting is not permitted because it is considered that damaged members should not be put into service even at a lower rating.

Local spalling or flaking of the compressed concrete in flexural elements related to casting imperfections need not indicate overall structural distress. Crack widths are good indicators of the state of the structure and should be observed to help determine whether the structure is satisfactory. Exact prediction or measurement of crack widths in reinforced concrete elements, however, is not likely to be achieved under field conditions. Establish criteria before the test, relative to the types of cracks anticipated, where the cracks will be measured, how they will be measured, and to establish approximate limits or criteria to evaluate new cracks or limits for the changes in crack width.

R20.5.2 — The deflection limits and the retest option follow past practice. If the structure shows no evidence of failure, recovery of deflection after removal of the test load is used to determine whether the strength of the structure is satisfactory. In the case of a very stiff structure, however, the errors in measurements under field conditions may be of the same order as the actual deflections and recovery. To avoid penalizing a satisfactory structure in such a case, recovery measurements are waived if the maximum deflection is less than $\ell_t^2/(20{,}000h)$. The residual deflection Δ_{rmax} is the difference between the initial and final (after load removal) deflections for the load test or the repeat load test.

R20.5.3 — Forces are transmitted across a shear crack plane by a combination of aggregate interlock at the interface of the crack that is enhanced by clamping action of transverse stirrup reinforcing and by dowel action of stirrups crossing the crack. As crack lengths increase to approach a horizontal projected length equal to the depth of the member

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and concurrently widen to the extent that aggregate interlock cannot occur, and as transverse stirrups if present begin to yield or display loss of anchorage so as to threaten their integrity, the member is assumed to be approaching imminent shear failure.

20.5.4 — In regions of structural members without transverse reinforcement, appearance of structural cracks inclined to the longitudinal axis and having a horizontal projection longer than the depth of the member at midpoint of the crack shall be evaluated.

R20.5.4 — The intent of 20.5.4 is to make the professionals in charge of the test pay attention to the structural implication of observed inclined cracks that may lead to brittle collapse in members without transverse reinforcement.

20.5.5 — In regions of anchorage and lap splices, the appearance along the line of reinforcement of a series of short inclined cracks or horizontal cracks shall be evaluated.

R20.5.5 — Cracking along the axis of the reinforcement in anchorage zones may be related to high stresses associated with the transfer of forces between the reinforcement and the concrete. These cracks may be indicators of pending brittle failure of the element if they are associated with the main reinforcement. It is important that their causes and consequences be evaluated.

20.6 — Provision for lower load rating

If the structure under investigation does not satisfy conditions or criteria of 20.1.2, 20.5.2, or 20.5.3, the structure shall be permitted for use at a lower load rating based on the results of the load test or analysis, if approved by the building official.

R20.6 — Provision for lower load rating

Except for load tested members that have failed under a test (see 20.5), the building official may permit the use of a structure or member at a lower load rating that is judged to be safe and appropriate on the basis of the test results.

20.7 — Safety

20.7.1 — Load tests shall be conducted in such a manner as to provide for safety of life and structure during the test.

20.7.2 — No safety measures shall interfere with load test procedures or affect results.

CHAPTER 21 — SPECIAL PROVISIONS FOR SEISMIC DESIGN

CODE

21.0 — Notation

A_{ch} = cross-sectional area of a structural member measured out-to-out of transverse reinforcement, in.^{2}

A_{cp} = area of concrete section, resisting shear, of an individual pier or horizontal wall segment, in.^{2}

A_{cv} = gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, in.^{2}

A_g = gross area of section, in.^{2}

A_j = effective cross-sectional area within a joint, see 21.5.3.1, in a plane parallel to plane of reinforcement generating shear in the joint, in.^{2} The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of:

(a) beam width plus the joint depth;

(b) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. See 21.5.3.1

A_s = area of nonprestressed tension reinforcement, in.^{2}

A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension h_c, in.^{2}

A_v = area of shear reinforcement within a distance s, in.^{2}

A_{vd} = total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam, in.^{2}

b = effective compressive flange width of a structural member, in.

b_w = web width, or diameter of circular section, in.

c = distance from the extreme compression fiber to neutral axis, see 10.2.7, calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth, in.

c_1 = size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, in.

c_t = dimension equal to the distance from the interior

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R21.0 — Notation

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as ft or kip. Nevertheless, care must be taken to properly convert constants in equations, which may include factors to account for units of defined variables.

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face of the column to the slab edge measured parallel to c_1, but not exceeding c_1, in.

d = effective depth of section, in.

d_b = bar diameter, in.

D_T = inside diameter of a circular tank, ft

E = load effects of earthquake, or related internal moments and forces

f'_c = specified compressive strength of concrete, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi

f_y = specified yield strength of reinforcement, psi

f_{yh} = specified yield strength of transverse reinforcement, psi

h = overall thickness of member, in.

h_c = cross-sectional dimension of column core measured center-to-center of confining reinforcement, in.

h_w = height of entire wall or of the segment of wall considered, in.

h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the column, in.

H_L = depth of stored liquid, ft

ℓ_d = development length for a straight bar, in.

ℓ_{dh} = development length for a bar with a standard hook as defined in Eq. (21-6), in.

ℓ_n = clear span measured face-to-face of supports, in.

ℓ_o = minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, in.

ℓ_w = length of entire wall or of segment of wall considered in direction of shear force, in.

L = live loads, or related internal moments and force

L_T = length of rectangular tank (inside dimension) parallel to the design earthquake direction being evaluated, ft

M_c = moment at the face of the joint, corresponding to the nominal flexural strength of the column framing into that joint, calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength, in.-lb. See 21.4.2.2

M_g = moment at the face of the joint, corresponding to the nominal flexural strength of the girder including slab where in tension, framing into that joint, in.-lb. See 21.4.2.2

M_n = nominal moment strength at section, in-lb.

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M_{pr} = probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least $1.25f_y$ and a strength reduction factor ϕ of 1.0, in.-lb

M_s = portion of slab moment balanced by support moment, in.-lb

M_u = factored moment at section, in.-lb

P_o = nominal axial load strength at zero eccentricity, lb

R = numerical coefficient representing the combined effect of the structure's ductility, energy-dissipating capacity, and structural redundancy

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, in.

s_o = maximum spacing of transverse reinforcement, in.

s_x = longitudinal spacing of transverse reinforcement within the length ℓ_o, in.

S = snow load, or related internal moments and forces

S_d = environmental durability factor. See 9.2.6

S_e = moment, shear, or axial force at connection corresponding with development of probable strength at intended yield locations, based on the governing mechanism of inelastic lateral deformation, considering both gravity and earthquake load effects

S_n = nominal flexural, shear, or axial strength of the connection

S_y = yield strength of connection, based on f_y, for moment, shear, or axial force

U = required strength to resist factored loads or related internal moments and forces

v_n = nominal shear stress, psi. See 11.12.6.2

V_c = nominal shear strength provided by concrete, lb

V_e = design shear force determined from 21.3.4.1 or 21.4.5.1, lb

V_n = nominal shear strength, lb

V_u = factored shear force at section, lb

α = angle between the diagonal reinforcement and the longitudinal axis of a diagonally reinforced coupling beam

α_c = coefficient defining the relative contribution of concrete strength to wall strength. See Eq. (21-7)

δ_u = design displacement, in.

ρ = ratio of nonprestressed tension reinforcement

= A_s/bd

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ρ_g = ratio of total reinforcement area to cross-sectional area of column

ρ_n = ratio of area of distributed reinforcement parallel to the plane of A_{cv} to gross concrete area perpendicular to that reinforcement

ρ_s = ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out)

ρ_v = ratio of area of distributed reinforcement perpendicular to the plane of A_{cv} to gross concrete area A_{cv}

ϕ = strength reduction factor

21.1 — Definitions

Base of structure — Level at which earthquake motions are assumed to be imparted to a building.

This level does not necessarily coincide with the ground level.

Boundary elements — Portions along structural wall and structural diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms shall be provided with boundary elements as required by 21.7.6 or 21.9.8.

Collector elements — Elements that serve to transmit the inertial forces within structural diaphragms to members of the lateral-force-resisting systems.

Convective pressures — The hydrodynamic pressure on a liquid-containing structure during an earthquake, due to the upper (sloshing) portion of its contents.

Connection — A region that joins two or more members, of which one or more is precast.

Ductile connection — Connection that experiences yielding as a result of the design displacements.

Strong connection — Connection that remains elastic while adjoining members experience yielding as a result of the design displacements.

Crosstie — A continuous reinforcing bar having a seismic hook at one end and a hook not less than 90 deg with at least a six-diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90-deg hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.

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R21.1 — Definitions

Convective pressures — When a liquid-containing structure is accelerated horizontally, that acceleration induces oscillations in which the upper, sloshing portion of the contained liquid responds as an oscillating mass flexibly connected to the walls. The hydrodynamic pressures on the structure due to that oscillating mass are known as the convective pressures.

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Design displacement — Total lateral displacement expected for the design-basis earthquake, as required by the governing code for earthquake-resistant design.

Design load combinations — Combinations of factored loads and forces in 9.2.

Development length for a bar with a standard hook — The shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent to the outer edge of the 90-deg hook.

Factored loads and forces — Loads and forces multiplied by appropriate load factors in 9.2.

Hoop — A closed tie or continuously wound tie. A closed tie can be made up of several reinforcement elements each having seismic hooks at both ends. A continuously wound tie shall have a seismic hook at both ends.

Impulsive pressures — The hydrodynamic pressure on a liquid-containing structure during an earthquake, due to the lower portion of its contents.

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Design displacement — The design displacement is an index of the maximum lateral displacement expected in design for the design-basis earthquake. In documents such as the National Earthquake Hazards Reduction Provisions (NEHRP),[21.1] ASCE 7-95,[21.2] the Uniform Building Code (UBC),[21.3] the BOCA/National Building Code (BOCA)[21.4] published by Building Officials and Code Administrators International, or the Standard Building Code (SBC)[21.5] published by Southern Building Code Congress International, the design-basis earthquake has approximately a 90 percent probability of nonexceedance in 50 years. In documents such as the International Building Code (IBC)[21.6] and the NFPA 5000,[21.7] the design-basis earthquake for most regions is taken as 2/3 of an earthquake that has approximately a 98 percent probability of nonexceedance in 50 years. In those documents, the design displacement is calculated using static or dynamic linear elastic analysis under code specified actions considering effects of cracked sections, effects of torsion, effects of vertical forces acting through lateral displacements, and modification factors to account for expected inelastic response. The design displacement generally is larger than the displacement calculated from design-level forces applied to a linear-elastic model of the building, within the limitations of R21.2.1 for liquid containing structures.

Impulsive pressures — When a liquid-containing structure is accelerated horizontally, the lower portion of the contained liquid that does not slosh responds as a solid mass rigidly attached to the container walls. The hydrodynamic pressures due to this mass are known as the impulsive pressures.

For schematic representation of convective and impulsive pressures see References 21.8 through 21.10.

The provisions of 21.6 are intended to result in a special moment frame constructed using precast concrete having minimum strength and toughness equivalent to that for a special moment frame of cast-in-place concrete.

The provisions of 21.13 are intended to result in an intermediate precast structural wall having minimum strength and

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toughness equivalent to that for an ordinary reinforced concrete structural wall of cast-in-place concrete. A precast concrete wall satisfying only the requirements of Chapters 1 through 18 and not the additional requirements of 21.13 or 21.8 is considered to have ductility and structural integrity less than that for an intermediate precast structural wall.

The provisions of 21.8 are intended to result in a special precast structural wall having minimum strength and toughness equivalent to that for a special reinforced concrete structural wall of cast in-place concrete.

Joint — Portion of structure common to intersecting members. The effective area of the joint for shear strength computations is defined in 21.0 (See A_j).

Lateral-force resisting system — That portion of the structure composed of members proportioned to resist forces related to earthquake effects.

Lightweight aggregate concrete — All-lightweight or sand-lightweight aggregate concrete made with lightweight aggregates conforming to 3.3.

Liquid-containing structure — A primary or secondary containment structure that is designed to contain liquids of fluidized materials and gases. The structure may have any shape in plan and may incorporate various floor or roof designs.

Moment frame — Frame in which members and joints resist forces through flexure, shear, and axial force. Moment frames shall be categorized as follows:

Intermediate moment frame — A cast-in-place frame complying with the requirements of 21.2 and 21.12 in addition to the requirements for ordinary moment frames.

Ordinary moment frame — A cast-in-place or precast concrete frame complying with the requirements of Chapters 1 through 18.

Special moment frame — A cast-in-place frame complying with the requirements of 21.2 through 21.5 or a precast frame complying with the requirements of 21.2 through 21.6.

Plastic hinge region — Length of frame element over which flexural yielding is intended to occur due to design displacements, extending not less than a distance h from the critical section where flexural yielding initiates.

Seismic hook — A hook on a stirrup, hoop, or crosstie having a bend not less than 135 deg, except that circular hoops shall have a bend not less than 90 deg. Hooks shall have a six-diameter (but not less than 3 in.) extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

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Special boundary elements — Boundary elements required by 21.7.6.2 or 21.7.6.3.

Specified lateral forces — Lateral forces corresponding to the appropriate distribution of the design base shear force prescribed by the governing code for earthquake-resistant design.

Structural diaphragms — Structural members, such as floor and roof slabs, that transmit inertial forces to lateral-force resisting members.

Structural trusses — Assemblages of reinforced concrete members subjected primarily to axial forces.

Structural walls — Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shearwall is a structural wall. Structural walls shall be categorized as follows:

Intermediate precast structural wall — A wall complying with all applicable requirements of 21.2 and 21.13.

Ordinary reinforced concrete structural wall — A wall complying with the requirements of Chapters 1 through 18.

Special precast structural wall — A precast wall complying with the requirements of 21.2 and 21.8.

Special reinforced concrete structural wall — A cast-in-place wall complying with the requirements of 21.2 and 21.7.

Strut — An element of a structural diaphragm used to provide continuity around an opening in the diaphragm.

Tie elements — Elements that serve to transmit inertia forces and prevent separation of building components such as footings and walls.

21.2 — General requirements

21.2.1 — Scope

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R21.2 — General requirements

R21.2.1 — Scope

For nonliquid-containing members, Chapter 21 contains provisions considered to be the minimum requirements for a cast-in-place or precast concrete structure capable of sustaining a series of oscillations into the inelastic range of response without critical deterioration in strength. The integrity of the structure in the inelastic range of response should be maintained because the design forces defined in documents such as the IBC,[21.6] the UBC,[21.3] and the NEHRP[21.1] provisions are considered less than those corresponding to linear response at the anticipated earthquake intensity.[21.1,21.11-21.13]

For liquid-containing structures, lateral design forces are normally calculated on the basis of a linearly elastic model of the uncracked section. In reality, these models are not always completely accurate, as most reinforced concrete

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structures, particularly nonprestressed structures, undergo some amount of cracking, or at least microcracking. While the resultant decrease in effective stiffness and accompanying increase in energy dissipation may reduce the lateral inertia forces, these effects are neglected in the analysis. Accordingly, ***R*** factors must be selected that, in combination with the load and strength reduction factors, will keep the structure close to the elastic range, and maintain its serviceability under actual seismic loads.[21.14] In the case of secondary components that may be allowed to enter the inelastic range without adversely affecting the strength or the serviceability of the main component, larger ***R*** factors may be used in accordance with the provisions of Reference 21.1, Section 1E.2.a.

As a properly detailed reinforced cast-in-place or precast concrete structure responds to strong ground motion, its effective stiffness decreases and its energy dissipation increases. These developments tend to reduce the response accelerations and lateral inertia forces relative to values that would occur were the structure to remain a linearly elastic and lightly damped.[21.15] Thus, the use of design forces representing earthquake effects such as those in Reference 21.3 requires that the lateral-force resisting system retain a substantial portion of its strength into the inelastic range under displacement reversals. Toughness of the structure is an essential property for earthquake resistance.

The provisions of Chapter 21 relate detailing requirements to type of structural framing, earthquake risk level at the site, level of inelastic deformation intended in structural design, and use and occupancy of the structure. Earthquake risk levels traditionally have been classified as low, moderate, and high. The seismic risk level of a region or the seismic performance or design category of a structure is regulated by the legally adopted general building code or determined by local authority (see 1.1.8.2, R1.1.8.2, and Table R1.1.8.2). The 2003 IBC[21.6] and the 2000 NEHRP[21.15] provisions use the same terminology as the 1997 NEHRP[21.1] provisions.

The design and detailing requirements should be compatible with the level of energy dissipation (or toughness) assumed in the computation of the design seismic loads. The terms ordinary, intermediate, and special are specifically used to facilitate this compatibility. The degree of required toughness, and therefore, the level of required detailing, increases for structures progressing from ordinary through intermediate to special categories. It is essential that structures in higher seismic zones or assigned to higher seismic performance or design categories possess a higher degree of toughness. It is permitted, however, to design for higher toughness in the lower seismic zones or design categories and take advantage of the lower design force levels.

The provisions of Chapters 1 through 18 are intended to provide adequate toughness for structures in regions of low seismic risk, or assigned to ordinary categories. Therefore, it is not required to apply the provisions of Chapter 21 to

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lateral-force-resisting systems consisting of ordinary structural walls.

Chapter 21 requires special details for reinforced concrete structures in regions of moderate seismic risk, or assigned to intermediate seismic performance or design categories. These requirements are contained in 21.2.1.4, 21.12, and 21.13. Although new provisions are provided in 21.13 for design of intermediate precast structural walls, general building codes that address seismic performance or design categories currently do not include intermediate structural walls.

Structures in regions of high seismic risk, or assigned to high seismic performance or design categories, may be subjected to strong ground shaking. Structures designed using seismic forces based upon response modification factors for special moment frames or special reinforced concrete structural walls are likely to experience multiple cycles of lateral displacements well beyond the point where reinforcement yields should the design earthquake ground shaking occur. The provisions of 21.2 through 21.11 have been developed to provide the structure with adequate toughness for this special response.

The requirements of Chapter 21 as they apply to various components of structures in regions of various levels of seismic risk are summarized in Table R21.2.1.

The special proportioning and detailing requirements in Chapter 21 are based predominantly on field and laboratory experience with monolithic reinforced concrete building structures and precast concrete building structures designed and detailed to behave like monolithic building structures. Extrapolation of these requirements to other types of cast-in-place or precast concrete structures should be based on evidence provided by field experience, tests, or analysis. ACI T1.1-01, “Acceptance Criteria for Moment Frames Based on Structural Testing,” can be used in conjunction with Chapter 21 to demonstrate that the strength and toughness of a proposed frame system equals or exceeds that provided by a comparable monolithic concrete system.

TABLE R21.2.1—SECTIONS OF CHAPTER 21 TO BE SATISFIED*

	Level of seismic risk or assigned seismic performance or design categories (as defined in code section)		
Component resisting earthquake effect, unless otherwise noted	Low (21.2.1.3)	Moderate/ intermediate (21.2.1.4)	High (21.2.1.5)
Frame members	None	21.12	21.3, 21.4, 21.5, 21.6
Structural walls and coupling beams	None	None	21.7
Precast structural walls	None	21.13	21.8
Structural diaphragm and trusses	None	None	21.9
Foundations	None	None	21.10
Frame members not proportioned to resist forces induced by earthquake motions	None	None	21.11

*Structural members under all seismic levels must satisfy 21.2.

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The toughness requirements in 21.2.1.5 refer to the concern for the structural integrity of the entire lateral-force-resisting system at lateral displacements anticipated for ground motions corresponding to the design earthquake. Depending on the energy-dissipation characteristics of the structural system used, such displacements may be larger than for a monolithic reinforced concrete structure.

21.2.1.1 — Chapter 21 contains special requirements for design and construction of reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response. For liquid-containing structures, serviceability considerations preclude significant excursions into the nonlinear range under unfactored loads.

R21.2.1.1 — This section provides directions to the designer of liquid-containing concrete structures for computing seismic forces that are to be applied to the particular structure. The designer should also consider the effects of seismic forces on piping, equipment (for example, clarifier mechanisms), and connecting walkways, where vertical or horizontal movements between adjoining structures or surrounding backfill could adversely influence the ability of the structure to function properly.[21.16] Moreover, seismic forces applied at the interface of piping or walkways with the structure may also introduce appreciable flexural or shear stresses at these connections.

21.2.1.2 — The provisions of Chapters 1 through 19 shall apply except as modified by the provisions of this chapter. The provisions of 21.3 through 21.13 need only be applied where specifically required by the code. The requirements of 21.2 apply to all elements designed in accordance with this Chapter. Where the design seismic loads are computed using provisions for intermediate or special concrete systems, the requirements of Chapter 21 for intermediate or special systems, as applicable, shall be satisfied. Horizontal frames and trusses providing lateral support for liquid-containing walls are not required to meet the provisions for special moment resisting frames. Liquid-containing walls with in-plane factored shear force less than $2A_{cv}\sqrt{f'_c}$ are not required to meet the provisions for special structural walls.

R21.2.1.2 — Horizontal frames and trusses providing lateral support for liquid-containing walls are not required to meet the provisions for special moment resisting frames for the following reasons:

(a) Because of the relatively low R used for liquid containing structures, there is low ductility demand;

(b) These provisions were intended to prevent progressive collapse of building type structures, which are not applicable to liquid-containing walls;

(c) Because the horizontal frame or truss is designed for the full lateral load to the contained liquid, the increase in stress due to seismic loading is relatively low compared to building type structures;

(d) In many cases, the lateral deflection of the horizontal frame or truss controls the design, rather than stress.

Liquid-containing walls with in-plane factored shear force less than $\phi 2A_{cv}\sqrt{f'_c}$ are not required to meet the provisions for special structural walls for the following reasons:

(a) Because of the relatively low R used for liquid containing structures, there is low ductility demand;

(b) These provisions were intended to prevent progressive collapse of building type structures, which are not applicable to liquid-containing walls. For in-plane factored shear forces below this level, shear is resisted by concrete alone. Therefore, additional reinforcement above Section 7.12 requirements and special development and splices are not required.

21.2.1.3 — In regions of low seismic risk or for structures assigned to low seismic performance or design categories, liquid-containing structures shall be designed by the lateral force procedures prescribed in

R21.2.1.3 — Lateral-force procedures in local governing codes normally do not account for the dynamic behavior of tanks at grade with an L_T/H_L or D_T/H_L ratio less than 2.0 or of pedestal-mounted tanks.

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the governing codes or ACI 350.3; except tanks at grade with an L_T/H_L or D_T/H_L ratio less than 2.0 and pedestal-mounted tanks shall be designed by the procedures prescribed in ACI 350.3.

21.2.1.4 — In regions of moderate seismic risk or for structures assigned to intermediate seismic performance or design categories, the design seismic loads acting on liquid-containing structures shall be computed using the procedures prescribed in ACI 350.3. For structures in such regions or seismic performance or design categories, forces induced by earthquake motions shall be resisted by members conforming to the following: (a) ordinary structural walls; (b) special moment frames conforming to 21.3, 21.4, 21.5, and 21.6, as applicable; (c) special structural walls conforming to 21.7 and 21.8, as applicable; (d) intermediate moment frames conforming to 21.12; and (e) intermediate precast structural walls conforming to 21.13. Where the design seismic loads are computed using provisions for special concrete systems, the requirements of Chapter 21 for special systems, as applicable, shall be satisfied.

21.2.1.5 — In regions of high seismic risk or for structures assigned to high seismic performance or design categories, the design seismic loads acting on liquid-containing structures shall be computed using the procedures prescribed in ACI 350.3. For structures in such regions or seismic performance or design categories, forces induced by earthquake motions shall be resisted by members conforming to the following: (a) special moment frames conforming to 21.3, 21.4, 21.5, and 21.6, as applicable; (b) special structural walls conforming to 21.7 and 21.8, as applicable; (c) structural diaphragms and trusses conforming to 21.9; and (d) foundations conforming to 21.10. Frame members not proportioned to resist earthquake forces shall comply with 21.11.

21.2.1.6 — A reinforced concrete structural system not satisfying the requirements of this chapter shall be permitted if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this chapter.

21.2.1.7 — For liquid-containing structures, alternative methods of analysis based on generally accepted theory that is more rigorous than ACI 350.3 shall be permitted to be used provided that:

(a) Liquid-tightness is not compromised due to inelastic action; and

(b) The resulting values from the total lateral force and total base overturning moment are not less than 80 percent of the values that would be obtained using ACI 350.3.

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21.2.1.8 — Liquid-containing structures subjected to earthquake-induced forces shall be designed in accordance with the strength design method or the alternate design method (Appendix I) subject to the following:

(a) When a liquid-containing structure is designed in accordance with the strength design method, it shall be proportioned for the required strength $\boldsymbol{U}$ as defined in 9.2.1. The environmental durability factor ($\boldsymbol{S_d}$) defined in 9.2.6 and in C9.2.9 need not be applied to load combinations that include earthquake loads;

(b) When a liquid-containing structure is designed in accordance with the alternate design method, allowable stresses in Appendix I shall be permitted to be increased by one-third, when permitted by the governing building code.

21.2.2 — Analysis and proportioning of structural members

21.2.2.1 — The interaction of all structural and nonstructural members that materially affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

21.2.2.2 — Rigid members assumed not to be a part of the lateral-force resisting system shall be permitted provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of failure of structural and nonstructural members, which are not a part of the lateral-force resisting system, shall also be considered.

21.2.2.3 — Structural members below base of structure that are required to transmit to the foundation forces resulting from earthquake effects shall also comply with the requirements of Chapter 21.

21.2.2.4 — Liquid-containing structures shall be designed for the forces, shears, and moments resulting from horizontal and vertical accelerations of the containment structure and its contents.

The calculations of the lateral and vertical seismic forces, shears, and moments shall take into account the seismicity of the area; the overall ductility and

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R21.2.1.8 — For a classification of liquid-containing structures, see ACI 350.3.

Liquid-containing structures founded at or below grade are nonbuilding structures as defined in Reference 21.17, Sections 11.1 and 11.4. The most widely accepted basis for designing such structures considers impulsive and convective modes of fluid response to earthquakes. ACI 350.3 has been adapted from references that use this design basis.

The environmental durability factor specified in 9.2.6 in connection with static loadings during normal operations is not automatically applicable to seismic loads, which are normally infrequent and of short duration. Typically, it is assumed that structures designed for the specified maximum-level seismic forces will sustain some level of damage but will not collapse. For structures that require a higher level of liquid-tightness during and after an earthquake, or structures that must remain in continued operation with only minor repairable damage after such an event, the level of damage may be limited by the use of selected importance factor $\boldsymbol{I}$ specified in the applicable codes or standards, such as ACI 350.3. The importance factor is applied to calculated seismic loads where added resistance to structural damage is desired.

Alternately, if the potential for even minor damage to the structure is to be minimized, the engineer may select the applicable environmental durability factor $\boldsymbol{S_d}$ based on the definitions in 9.2.6.

R21.2.2 — Analysis and proportioning of structural members

It is assumed that the distribution of required strength to the various components of a lateral-force-resisting system will be guided by the analysis of a linearly elastic model of the system acted upon by the factored forces specified by the governing code. If nonlinear response history analyses for nonliquid-containing members are to be used, base motions should be selected after a detailed study of the site conditions and local seismic history.

Because the design basis for nonliquid-containing structural members admits nonlinear response, it is necessary to investigate the stability of the lateral-force resisting system as well as its interaction with other structural and nonstructural members at displacements larger than those indicated by linear analysis. To handle this without having to resort to nonlinear response analysis, one option is to multiply by a factor of at least two the displacements from linear analysis for the factored lateral forces, unless the governing code specifies the factors to be used as in References 21.1 and 21.3. For lateral displacement calculations, assuming all the horizontal structural members to be fully cracked is likely to lead to better estimates of the possible drift than using uncracked stiffness for all members.

The main concern of Chapter 21 is the safety of the structure.

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energy-dissipating capacity of the structure (factor ***R***); the importance of the structure; and the dynamic amplification factors.

21.2.3 — Strength reduction factors

Strength reduction factors shall be as given in 9.3.4.

21.2.4 — Concrete in members resisting earthquake-induced forces

21.2.4.1 — The specified compressive strength f'_c of the concrete shall be governed by Chapter 4 as applicable.

21.2.4.2 — Compressive strength of lightweight aggregate concrete used in design shall not exceed 5000 psi. Lightweight aggregate concrete with higher design compressive strength shall be permitted if demonstrated by experimental evidence that structural members made with that lightweight aggregate concrete provide strength and toughness equal to or exceeding those of comparable members made with normalweight aggregate concrete of the same strength.

21.2.5 — Reinforcement in members resisting earthquake-induced forces

Reinforcement resisting earthquake-induced flexural and axial forces in special moment frame members and in special structural wall boundary elements shall comply with ASTM A 706. ASTM A 615 Grades 40 and 60 reinforcement shall be permitted in these members if:

(a) The actual yield strength based on mill tests does not exceed the specified yield strength by more than 18,000 psi (retests shall not exceed this value by more than an additional 3000 psi); and

(b) The ratio of the actual ultimate tensile strength to the actual tensile yield strength is not less than 1.25.

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The intent of 21.2.2.1 and 21.2.2.2 is to draw attention to the influence of nonstructural members on structural response and to hazards from falling objects.

Section 21.2.2.3 alerts the designer to the fact that the base of the structure as defined in analysis may not necessarily correspond to the foundation or ground level.

In selecting member sizes for earthquake-resistant structures, it is important to consider problems related to congestion of reinforcement. The designer should ensure that all reinforcement can be assembled and placed and that concrete can be cast and consolidated properly. Use of upper limits of reinforcement ratios permitted is likely to lead to insurmountable construction problems especially at frame joints.

R21.2.4 — Concrete in members resisting earthquake-induced forces

Requirements of this section refer to concrete quality in frames, trusses, or walls proportioned to resist earthquake-induced forces. The maximum design compressive strength of lightweight aggregate concrete to be used in structural design calculations is limited to 5000 psi primarily because of paucity of experimental and field data on the behavior of nonliquid-containing members made with lightweight aggregate concrete subjected to displacement reversals in the nonlinear range. If convincing evidence is developed for a specific application, the limit on maximum compressive strength of lightweight concrete may be increased to a level justified by the evidence.

R21.2.5 — Reinforcement in members resisting earthquake-induced forces

Use of longitudinal reinforcement with strength substantially higher than that assumed in design will lead to higher shear and bond stresses at the time of development of yield moments. These conditions may lead to brittle failures in shear or bond and should be avoided even if such failures may occur at higher loads than those anticipated in design. Therefore, a ceiling is placed on the actual yield strength of the steel (see 21.2.5(a)).

The requirement for an ultimate tensile strength larger than the yield strength of the reinforcement (21.2.5(b)) is based on the assumption that the capability of a structural member to develop inelastic rotation capacity is a function of the length of the yield region along the axis of the member. In interpreting experimental results, the length of the yield region has been related to the relative magnitudes of ultimate and yield moments.[21.18] According to this interpretation, the larger the ratio of ultimate to yield moment, the longer the yield region. Chapter 21 requires that the ratio of

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actual tensile strength to actual yield strength is not less than 1.25. Members with reinforcement not satisfying this condition can also develop inelastic rotation, but their behavior is sufficiently different to exclude them from direct consideration on the basis of rules derived from experience with members reinforced with strain-hardening steel.

21.2.6 — Mechanical splices

21.2.6.1 — Mechanical splices shall be classified as either Type 1 or Type 2 mechanical splices, as follows:

(a) Type 1 mechanical splices shall conform to 12.14.3.2;

(b) Type 2 mechanical splices shall conform to 12.14.3.2 and shall develop the specified tensile strength of the spliced bar.

21.2.6.2 — In regions of high seismic risk or for structures assigned to high seismic performance design categories, Type 1 mechanical splices shall not be used within a distance equal to twice the member depth from the column or beam face, from the base of wall, or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements. Type 2 mechanical splices shall be permitted to be used at any location.

R21.2.6 — Mechanical splices

In a structure undergoing inelastic deformations during an earthquake, the tensile stresses in reinforcement may approach the tensile strength of the reinforcement. The requirements for Type 2 mechanical splices are intended to avoid a splice failure when the reinforcement is subjected to expected stress levels in yielding regions. Type 1 splices are not required to satisfy the more stringent requirements for Type 2 splices, and may not be capable of resisting the stress levels expected in yielding regions. The locations of Type 1 splices are restricted because tensile stresses in reinforcement in yielding regions can exceed the strength requirements of 12.14.3.2.

Recommended detailing practice would preclude the use of splices in regions of potential yield in members resisting earthquake effects. If use of mechanical splices in regions of potential yielding cannot be avoided, the designer should have documentation on the actual strength characteristics of the bars to be spliced, on the force-deformation characteristics of the spliced bar, and on the ability of the Type 2 splice to be used to meet the specified performance requirements.

21.2.7 — Welded splices

21.2.7.1 — In regions of high seismic risk or for structures assigned to high seismic performance design categories, welded splices in reinforcement resisting earthquake-induced forces shall conform to 12.14.3.4 and shall not be used within a distance equal to twice the member depth from the column or beam face, from the base of wall, or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements.

21.2.7.2 — Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement that is required by design shall not be permitted.

R21.2.7 — Welded splices

R21.2.7.1 — Welding of reinforcement should be according to ANSI/AWS D1.4 as required in Chapter 3. The locations of welded splices are restricted because reinforcement tension stresses in yielding regions can exceed the strength requirements of 12.14.3.4.

R21.2.7.2 — Welding of crossing reinforcing bars can lead to local embrittlement of the steel. If welding of crossing bars is used to facilitate fabrication or placement of reinforcement, it should be done only on bars added for such purposes. The prohibition of welding crossing reinforcing bars does not apply to bars that are welded with welding operations under continuous, competent control as in the manufacture of welded wire reinforcement.

21.2.8 — Anchoring to concrete

21.2.8.1 — Anchors resisting earthquake-induced forces in structures in regions of moderate or high seismic risk, or assigned to intermediate or high seismic performance or design categories shall conform to the additional requirements of D.3.3 of Appendix D.

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21.3 — Flexural members of special moment frames

21.3.1 — Scope

Requirements of 21.3 apply to special moment frame members: (a) resisting earthquake-induced forces; and (b) proportioned primarily to resist flexure. These frame members shall also satisfy the conditions of 21.3.1.1 through 21.3.1.4.

21.3.1.1 — Factored axial compressive force on the member shall not exceed **($A_g f'_c$/10)**.

21.3.1.2 — Clear span for the member shall not be less than four times its effective depth.

21.3.1.3 — The width-to-depth ratio shall not be less than 0.3.

21.3.1.4 — The width shall not be:

(a) Less than 10 in.; and

(b) More than the width of the supporting member (measured on a plane perpendicular to the longitudinal axis of the flexural member) plus distances on each side of the supporting member not exceeding three-fourths of the depth of the flexural member.

21.3.2 — Longitudinal reinforcement

21.3.2.1 — At any section of a flexural member, except as provided in 10.5.3, for top as well as for bottom reinforcement, the amount of reinforcement shall not be less than that given by Eq. (10-3) but not less than $200 b_w d/f_y$, and the reinforcement ratio ρ shall not exceed 0.025. At least two bars shall be provided continuously both top and bottom.

21.3.2.2 — Positive moment strength at joint face shall be not less than one-half of the negative moment strength provided at that face of the joint. Neither the

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R21.3 — Flexural members of special moment frames

R21.3.1 — Scope

This section refers to beams of special moment frames resisting lateral loads induced by earthquake motions. If any frame member subjected to a factored axial compressive force exceeding **($A_g f'_c$/10)** is to be proportioned and detailed as described in 21.4.

Experimental evidence[21.19] indicates that, under reversals of displacement into the nonlinear range, behavior of continuous members having length-to-depth ratios of less than four is significantly different from the behavior of relatively slender members. Design rules derived from experience with relatively slender members do not apply directly to members with length-to-depth ratios less than four, especially with respect to shear strength.

Geometric constraints indicated in 21.3.1.3 and 21.3.1.4 were derived from practice with reinforced concrete frames resisting earthquake-induced forces.[21.17]

R21.3.2 — Longitudinal reinforcement

Section 10.3.5 limits net tensile strain ε_t, thereby indirectly limiting the tensile reinforcement ratio in a flexural member to a fraction of the amount that would produce balanced conditions. For a section subjected to bending only and loaded monotonically to yielding, this approach is feasible because the likelihood of compressive failure can be estimated reliably with the behavioral model assumed for determining the reinforcement ratio corresponding to balanced failure. The same behavioral model (because of incorrect assumptions such as linear strain distribution, well-defined yield point for the steel, limiting compressive strain in the concrete of 0.003, and compressive stresses in the shell concrete) fails to describe the conditions in a flexural member subjected to reversals of displacements well into the inelastic range. Thus, there is little rationale for continuing to refer to balanced conditions in earthquake-resistant design of reinforced concrete structures.

R21.3.2.1 — The limiting reinforcement ratio of 0.025 is based primarily on considerations of steel congestion and, indirectly, on limiting shear stresses in girders of typical proportions. The requirement of at least two bars, top and bottom, refers again to construction rather than behavioral requirements.

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negative nor the positive moment strength at any section along member length shall be less than one-fourth the maximum moment strength provided at face of either joint.

21.3.2.3 — Lap splices of flexural reinforcement shall be permitted only if hoop or spiral reinforcement is provided over the lap length. Maximum spacing of the transverse reinforcement enclosing the lapped bars shall not exceed ***d*/4** or 4 in. Lap splices shall not be used

(a) Within the joints;

(b) Within a distance of twice the member depth from the face of the joint; and

(c) At locations where analysis indicates flexural yielding caused by inelastic lateral displacements of the frame.

21.3.2.4 — Mechanical splices shall conform to 21.2.6 and welded splices shall conform to 21.2.7.

21.3.3 — Transverse reinforcement

21.3.3.1 — Hoops shall be provided in the following regions of frame members:

(a) Over a length equal to twice the member depth measured from the face of the supporting member toward midspan, at both ends of the flexural member;

(b) Over lengths equal to twice the member depth on both sides of a section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame.

21.3.3.2 — The first hoop shall be located not more than 2 in. from the face of a supporting member. Maximum spacing of the hoops shall not exceed (a), (b), (c), and (d):

(a) ***d*/4**;

(b) Eight times the diameter of the smallest longitudinal bars;

(c) 24 times the diameter of the hoop bars; and

(d) 12 in.

21.3.3.3 — Where hoops are required, longitudinal bars on the perimeter shall have lateral support conforming to 7.10.5.3.

21.3.3.4 — Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than ***d*/2** throughout the length of the member.

21.3.3.5 — Stirrups or ties required to resist shear shall be hoops over lengths of members in 21.3.3, 21.4.4, and 21.5.2.

COMMENTARY

R21.3.2.3 — Lap splices of reinforcement are prohibited at regions where flexural yielding is anticipated because such splices are not reliable under conditions of cyclic loading into the inelastic range. Transverse reinforcement for lap splices at any location is mandatory because of the likelihood of loss of shell concrete.

R21.3.3 — Transverse reinforcement

Transverse reinforcement is required primarily to confine the concrete and maintain lateral support for the reinforcing bars in regions where yielding is expected. Examples of hoops suitable for flexural members of frames are shown in Fig. R21.3.3.

In the case of members with varying strength along the span or members for which the permanent load represents a large proportion of the total design load, concentrations of inelastic rotation may occur within the span. If such a condition is anticipated, transverse reinforcement also should be provided in regions where yielding is expected.

Because spalling of the concrete shell is anticipated during

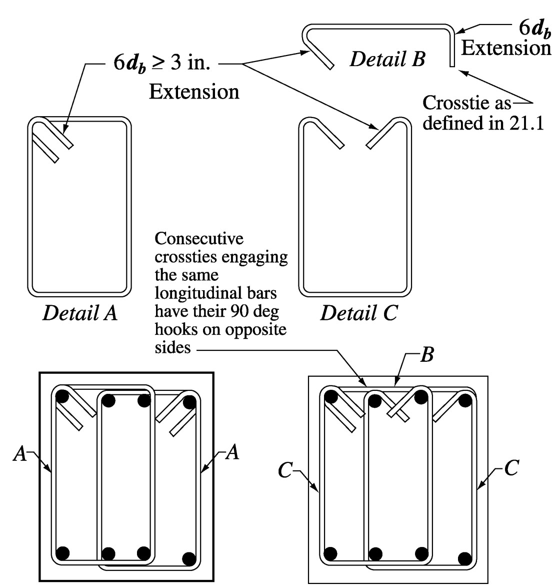

Fig. R21.3.3—Examples of overlapping hoops.

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21.3.3.6 — Hoops in flexural members shall be permitted to be made up of two pieces of reinforcement: a stirrup having seismic hooks at both ends and closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90-deg hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the crossties are confined by a slab on only one side of the flexural frame member, the 90-deg hooks of the crossties shall be placed on that side.

21.3.4 — Shear strength requirements

21.3.4.1 — Design forces

The design shear force V_e shall be determined from consideration of the statical forces on the portion of the member between faces of the joints. It shall be assumed that moments of opposite sign corresponding to probable flexural moment strength M_{pr} act at the joint faces and that the member is loaded with the factored tributary gravity load along its span.

21.3.4.2 — Transverse reinforcement

Transverse reinforcement over the lengths identified in 21.3.3.1 shall be proportioned to resist shear assuming $V_c = 0$ when both of the following conditions occur:

(a) The earthquake-induced shear force calculated in accordance with 21.3.4.1 represents one-half or more of the maximum required shear strength within those lengths;

(b) The factored axial compressive force including earthquake effects is less than $A_g f_c'/20$.

COMMENTARY

strong motion, especially at and near regions of flexural yielding, all web reinforcement should be provided in the form of closed hoops as defined in 21.3.3.5.

R21.3.4 — Shear strength requirements

R21.3.4.1 — Design forces

In determining the equivalent lateral forces representing earthquake effects for the type of frames considered, it is assumed that frame members will dissipate energy in the nonlinear range of response. Unless a frame member possesses a strength that is a multiple on the order of 3 or 4 of the design forces, it should be assumed that it will yield in the event of a major earthquake. The design shear force should be a good approximation of the maximum shear that may develop in a member. Therefore, required shear strength for frame members is related to flexural strengths of the designed member rather than to factored shear forces indicated by lateral load analysis. The conditions described by 21.3.4.1 are illustrated in Fig. R21.3.4.

Because the actual yield strength of the longitudinal reinforcement may exceed the specified yield strength and because strain hardening of the reinforcement is likely to take place at a joint subjected to large rotations, required shear strengths are determined using a stress of at least $1.25f_y$ in the longitudinal reinforcement.

R21.3.4.2 — Transverse reinforcement

Experimental studies[21.20,21.21] of reinforced concrete members subjected to cyclic loading have demonstrated that more shear reinforcement is required to ensure a flexural failure if the member is subjected to alternating nonlinear displacements than if the member is loaded in only one direction: the necessary increase of shear reinforcement being higher in the case of no axial load. This observation is reflected in the specifications (21.3.4.2) by eliminating the term representing the contribution of concrete to shear strength. The added conservatism on shear is deemed necessary in locations where potential flexural hinging may occur. This stratagem, however, chosen for its relative simplicity, should not be interpreted to mean that no concrete is required to resist shear. On the contrary, it may be argued that the concrete core resists all of the shear with the shear (transverse) reinforcement confining and thus strengthening the concrete. The confined concrete core plays an important role in the behavior of the beam and should not be reduced to a minimum just because the design expression does not explicitly recognize it.

CODE

21.4 — Special moment frame members subjected to bending and axial load

21.4.1 — Scope

The requirements of this section apply to special moment frame members: (a) resisting earthquake-induced forces; and (b) having a factored axial force exceeding $A_g f_c'/10$. These frame members shall also satisfy the conditions of 21.4.1.1 and 21.4.1.2.

21.4.1.1 — The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 12 in.

21.4.1.2 — The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

COMMENTARY

R21.4 — Special moment frame members subjected to bending and axial load

R21.4.1 — Scope

Section 21.4.1 is intended primarily for columns of special moment frames. Frame members, other than columns, that do not satisfy 21.3.1 are to be proportioned and detailed according to this section.

The geometric constraints in 21.4.1.1 and 21.4.1.2 follow from previous practice.[21.17]

Notes on Fig. R21.3.4:

1. Direction of shear force V_e depends on relative magnitudes of gravity loads and shear generated by end moments.

2. End moments M_{pr} based on steel tensile stress of $1.25 f_y$, where f_y is specified yield strength. (Both end moments should be considered in both directions, clockwise and counter-clockwise).

3. End moment M_{pr} for columns need not be greater than moments generated by the M_{pr} of the beams framing into the beam-column joints. V_e should not be less than that required by analysis of the structure.

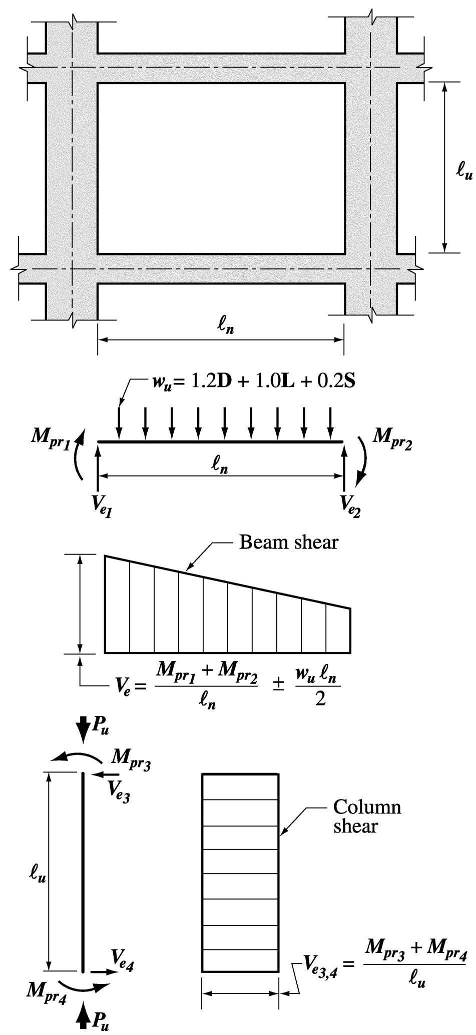

Fig. R21.3.4—Design shears for girders and columns.

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21.4.2 — Minimum flexural strength of columns

21.4.2.1 — Flexural strength of any column proportioned to resist a factored axial compressive force exceeding $A_g f_c'/10$ shall satisfy 21.4.2.2 or 21.4.2.3.

Lateral strength and stiffness of columns not satisfying 21.4.2.2 shall be ignored in determining the calculated strength and stiffness of the structure, but such columns shall conform to 21.11.

21.4.2.2 — The flexural strengths of the columns shall satisfy Eq. (21-1)

$$\Sigma M_c \geq (6/5)\Sigma M_g \qquad (21\text{-}1)$$

ΣM_c = sum of moments at the faces of the joint corresponding to the nominal flexural strength of the columns framing into that joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.

ΣM_g = sum of moments at the faces of the joint corresponding to the nominal flexural strengths of the girders framing into that joint. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width defined in 8.10 shall be assumed to contribute to flexural strength if the slab reinforcement is developed at the critical section for flexure.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Equation (21-1) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

21.4.2.3 — If 21.4.2.2 is not satisfied at a joint, columns supporting reactions from that joint shall be provided with transverse reinforcement as specified in 21.4.4.1 through 21.4.4.3 over their full height.

21.4.3 — Longitudinal reinforcement

21.4.3.1 — The reinforcement ratio ρ_g shall not be less than 0.01 and shall not exceed 0.06.

21.4.3.2 — Mechanical splices shall conform to 21.2.6 and welded splices shall conform to 21.2.7. Lap splices shall be permitted only within the center half of the member length shall be designed as tension lap splices and shall be enclosed within transverse reinforcement conforming to 21.4.4.2 and 21.4.4.3.

COMMENTARY

R21.4.2 — Minimum flexural strength of columns

The intent of 21.4.2.2 is to reduce the likelihood of yielding in columns that are considered as part of the lateral force resisting system. If columns are not stronger than beams framing into a joint, there is likelihood of inelastic action. In the worst case of weak columns, flexural yielding can occur at both ends of all columns in a given story, resulting in a column failure mechanism that can lead to collapse.

In 21.4.2.2, the nominal strengths of the girders and columns are calculated at the joint faces, and those strengths are compared directly using Eq. (21-1). The ACI 318-95 code required design strengths to be compared at the center of the joint, which typically produced similar results but with added computational effort.

When determining the nominal flexural strength of a girder section in negative bending (top in tension), longitudinal reinforcement contained within an effective flange width of a top slab that acts monolithically with the girder increases the girder strength. Research[21.22] on beam-column subassemblies under lateral loading indicates that using the effective flange widths defined in 8.10 gives reasonable estimates of girder negative bending strengths of interior connections at interstory displacement levels approaching 2 percent of story height. This effective width is conservative where the slab terminates in a weak spandrel.

If 21.4.2.2 cannot be satisfied at a joint, any positive contribution of the column or columns involved to the lateral strength and stiffness of the structure is to be ignored. Negative contributions of the column or columns should not be ignored. For example, ignoring the stiffness of the columns ought not be used as a justification for reducing the design base shear. If inclusion of those columns in the analytical model of the building results in an increase in torsional effects, the increase should be considered as required by the governing code.

R21.4.3 — Longitudinal reinforcement

The lower limit of the reinforcement ratio is to control time-dependent deformations and to have the yield moment exceed the cracking moment. The upper limit of the section reflects concern for steel congestion, load transfer from floor elements to column especially in low-rise construction, and the development of high shear stresses.

Spalling of the shell concrete, which is likely to occur near the ends of the column in frames of typical configuration, makes lap splices in those locations vulnerable. If lap splices are to be used at all, they should be located near the midheight where stress reversal is likely to be limited to a smaller stress range than at locations near the joints. Special transverse reinforcement is required along the lap-splice length because of the uncertainty in moment distributions along the height and the need for confinement of lap splices subjected to stress reversals.[21.23]

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21.4.4 — Transverse reinforcement

21.4.4.1 — Transverse reinforcement as required below in (a) through (e) shall be provided unless a larger amount is required by 21.4.3.2 or 21.4.5.

(a) The volumetric ratio of spiral or circular hoop reinforcement ρ_s shall not be less than that required by Eq. (21-2)

$$\rho_s = 0.12 f_c'/f_{yh} \quad (21\text{-}2)$$

and shall not be less than that required by Eq. (10-5);

(b) The total cross-sectional area of rectangular hoop reinforcement shall not be less than that required by Eq. (21-3) and (21-4)

$$A_{sh} = 0.3(sh_c f_c'/f_{yh})\lfloor(A_g/A_{ch}) - 1\rfloor \quad (21\text{-}3)$$

$$A_{sh} = 0.09 sh_c f_c'/f_{yh} \quad (21\text{-}4)$$

(c) Transverse reinforcement shall be provided by either single or overlapping hoops. Crossties of the same bar size and spacing as the hoops shall be permitted. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement;

(d) If the design strength of member core satisfies the requirement of the design loading combinations including earthquake effect, Eq. (21-3) and (10-7) need not be satisfied;

(e) If the thickness of the concrete outside the confining transverse reinforcement exceeds 4 in., additional transverse reinforcement shall be provided a spacing not exceeding 12 in. Concrete cover the additional reinforcement shall not exceed 4 in.

21.4.4.2 — Transverse reinforcement shall be spaced at a distance not exceeding: (a) one-quarter the minimum member dimension; (b) six times diameter of the longitudinal reinforcement; and (c) s_x, as defined by Eq. (21-5)

$$s_x = 4 + \left(\frac{14 - h_x}{3}\right) \quad (21\text{-}5)$$

The value of s_x shall not exceed 6 in. and need not be taken less than 4 in.

21.4.4.3 — Crossties or legs of overlapping hoops shall not be spaced more than 14 in. on center in the direction perpendicular to the longitudinal axis of structural member.

21.4.4.4 — Transverse reinforcement in amount specified in 21.4.4.1 through 21.4.4.3 shall be provided over a length ℓ_o from each joint face and on

COMMENTARY

R21.4.4 — Transverse reinforcement

Requirements of this section are concerned with confining the concrete and providing lateral support to the longitudinal reinforcement.

The effect of helical (spiral) reinforcement and adequately configured rectangular hoop reinforcement on strength and ductility of columns is well established.[21.24] While analytical procedures exist for calculation of strength and ductility capacity of columns under axial and moment reversals,[21.25] the axial load and deformation demands required during earthquake loading are not known with sufficient accuracy to justify calculation of required transverse reinforcement as a function of design earthquake demands. Instead, Eq. (10-7) and (21-3) are required, with the intent that spalling of shell concrete will not result in a loss of axial load strength of the column. Equation (21-2) and (21-4) govern for large-diameter columns, and are intended to ensure adequate flexural curvature capacity in yielding regions.

Figure R21.4.4 shows an example of transverse reinforcement provided by one hoop and three crossties. Crossties with a 90-deg hook are not as effective as either crossties with 135-deg hooks or hoops in providing confinement. Tests show that if crosstie ends with 90-deg hooks are alternated, confinement will be sufficient.

Sections 21.4.4.2 and 21.4.4.3 are interrelated requirements for configuration of rectangular hoop reinforcement. The requirement that spacing not exceed one-quarter of the minimum member dimension is to obtain adequate concrete confinement. The requirement that spacing not exceed six bar diameters is intended to restrain longitudinal reinforcement buckling after spalling. The 4 in. spacing is for concrete confinement; 21.4.4.2 permits this limit to be relaxed to a maximum of 6 in. if the spacing of crossties or legs of overlapping hoops is limited to 8 in.

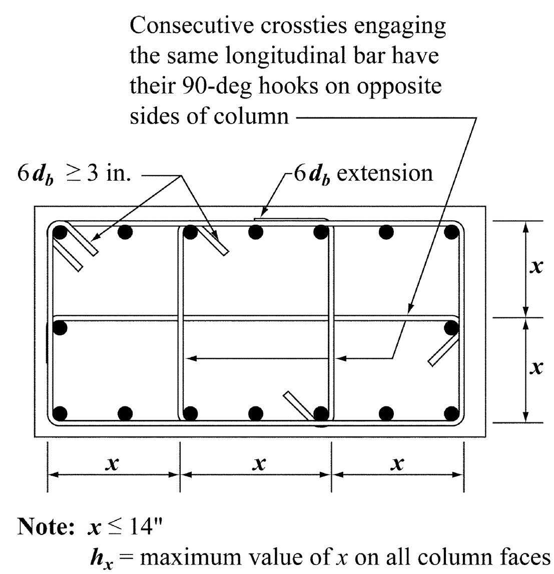

Fig. R21.4.4—Example of transverse reinforcement in columns.

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both sides of any section where flexural yielding is likely to occur as result of inelastic lateral displacements of the frame. The length ℓ_o shall not be less than (a), (b), and (c):

(a) The depth of the member at the joint face or the section where flexural yielding is likely to occur;

(b) one-sixth of the clear span of the member; and

(c) 18 in.

21.4.4.5 — Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with transverse reinforcement as required in 21.4.4. through 21.4.4.3 over their full height beneath the level that the discontinuity occurs if the factored axial compressive force in these members, related to earthquake effect, exceeds $(A_g f_c'/10)$. Transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 shall extend into discontinued member for at least the development length of the largest longitudinal reinforcement in the column in accordance with 21.5.4. If the lower end of the column terminates on a wall, transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 shall extend into the wall for at least the development length of the largest longitudinal bar in the column at the point of termination. If the column terminates on a footing or mat, transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 shall extend at least 12 in. into the footing or mat.

21.4.4.6 — Where transverse reinforcement, as specified in 21.4.4.1 through 21.4.4.3, is not provided throughout the full length of the column, the remainder of the column length shall contain spiral or hoop reinforcement with center-to-center spacing not exceeding the smaller of six times the diameter of the longitudinal column bars or 6 in.

21.4.5 — Shear strength requirements

21.4.5.1 — Design forces

The design shear force V_e shall be determined from consideration of the maximum forces that can be generated at the faces of the joints at each end of the member. These joint forces shall be determined using the maximum probable moment strengths M_{pr} of the member associated with the range of factored axial loads on the member. The member shears need not exceed those determined from joint strengths based on the probable moment strength M_{pr} of the transverse members framing into the joint. In no case shall V_e be less than the factored shear determined by analysis of the structure.

COMMENTARY

The unreinforced shell may spall as the column deforms to resist earthquake effects. Separation of portions of the shell from the core caused by local spalling creates a falling hazard. The additional reinforcement is required to reduce the risk of portions of the shell falling away from the column.

Section 21.4.4.4 stipulates a minimum length over which to provide closely-spaced transverse reinforcement at the member ends, where flexural yielding normally occurs. Research results indicate that the length should be increased by 50 percent or more in locations, such as the base of the building, where axial loads and flexural demands may be especially high.[21.26]

Columns supporting discontinued stiff members, such as walls or trusses, may develop considerable inelastic response. Therefore, it is required that these columns have special transverse reinforcement throughout their length. This covers all columns beneath the level at which the stiff member has been discontinued, unless the factored forces corresponding to earthquake effect are low (see 21.4.4.5).

Field observations have shown significant damage to columns in the unconfined region near the midheight. The requirements of 21.4.4.6 are to ensure a relatively uniform toughness of the column along its length.

R21.4.4.6 — The provisions of 21.4.4.6 were added to the 1989 318 code to provide reasonable protection and ductility to the midheight of columns between transverse reinforcement. Observations after earthquakes have shown significant damage to columns in the nonconfined region, and the minimum ties or spirals required should provide a more uniform toughness of the column along its length.

R21.4.5 — Shear strength requirements

R21.4.5.1 — Design forces

The provisions of 21.3.4.1 also apply to members subjected to axial loads (for example, columns). Above the ground floor, the moment at a joint may be limited by the flexural strength of the beams framing into the joint. Where beams frame into opposite sides of a joint, the combined strength may be the sum of the negative moment strength of the beam on one side of the joint and the positive moment strength of the beam on the other side of the joint. Moment strengths are to be determined using a strength reduction factor of 1.0 and reinforcing steel stress equal to at least $1.25f_y$. Distribution of the combined moment strength of the beams to the columns above and below the joint should be based on analysis. The value of M_{pr} in Fig. R21.3.4 may be computed from the flexural member strengths at the beam-column joints.

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21.4.5.2 — Transverse reinforcement over the lengths ℓ_o, identified in 21.4.4.4, shall be proportioned to resist shear assuming $V_c = 0$ when both the following conditions occur:

(a) The earthquake-induced shear force, calculated in accordance with 21.4.5.1, represents one-half or more of the maximum required shear strength within those lengths;

(b) The factored axial compressive force including earthquake effects is less than $(A_g f'_c/20)$.

21.5 — Joints of special moment frames

21.5.1 — General requirements

21.5.1.1 — Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural tensile reinforcement is $1.25f_y$.

21.5.1.2 — Strength of joint shall be governed by the appropriate strength reduction factors in 9.3.

21.5.1.3 — Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension according to 21.5.4 and in compression according to Chapter 12.

21.5.1.4 — Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall not be less than 20 times the diameter of the largest longitudinal bar for normalweight concrete. For lightweight concrete, the dimension shall be not less than 26 times the bar diameter.

21.5.2 — Transverse reinforcement

21.5.2.1 — Transverse hoop reinforcement in 21.4.4 shall be provided within the joint, unless the joint is confined by structural members in 21.5.2.2.

21.5.2.2 — Within the depth of the shallowest framing member, transverse reinforcement equal to at least one-half the amount required by 21.4.4.1 shall be provided where members frame into all four sides of

COMMENTARY

R21.5 — Joints of special moment frames

R21.5.1 — General requirements

Development of inelastic rotations at the faces of joints of reinforced concrete frames is associated with strains in the flexural reinforcement well in excess of the yield strain. Consequently, joint shear force generated by the flexural reinforcement is calculated for a stress of $1.25f_y$ in the reinforcement (see 21.5.1.1). A detailed explanation of the reasons for the possible development of stresses in excess of the yield strength in girder tensile reinforcement is provided in Reference 21.18.

R21.5.1.4 — Research[21.27-21.31] has shown that straight beam bars may slip within the beam-column joint during a series of large moment reversals. The bond stresses on these straight bars may be very large. To substantially reduce slip during the formation of adjacent beam hinging, it would be necessary to have a ratio of column dimension to bar diameter of approximately 1/32, which would result in very large joints. On reviewing the available tests, the limit of 1/20 of the column depth in the direction of loading for the maximum size of beam bars for normalweight concrete, and a limit of 1/26 for lightweight concrete were chosen. Due to the lack of specific data, the modification for lightweight concrete used a factor of 1.3 from Chapter 12. Committee 318 feels that these limits provide reasonable control on the amount of potential slip of the beam bars in a beam-column joint considering the number of anticipated inelastic excursions of the building frames during a major earthquake. A thorough treatment of this topic is given in Reference 21.32.

R21.5.2 — Transverse reinforcement

However low the calculated shear force in a joint of a frame resisting earthquake-induced forces, confining reinforcement (see 21.4.4) should be provided through the joint around the column reinforcement (21.5.2.1). In 21.5.2.2, confining reinforcement may be reduced if horizontal members frame into all four sides of the joint. The 318-89 code provided a maximum limit on spacing to these areas based on available data.[21.33-21.36]

CODE

the joint and where each member width is at least three-fourths the column width. At these locations, the spacing required in 21.4.4.2 shall be permitted to be increased to 6 in.

21.5.2.3 — Transverse reinforcement as required by 21.4.4 shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement is not provided by a beam framing into the joint.

21.5.3 — Shear strength

21.5.3.1 — The nominal shear strength of the joint shall not be taken as greater than the values specified below for normalweight aggregate concrete.

For joints confined on all four faces .... $20\sqrt{f_c'}\,A_j$

For joints confined on three faces or on two opposite faces .... $15\sqrt{f_c'}\,A_j$

For others .... $12\sqrt{f_c'}\,A_j$

A member that frames into a face is considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. A joint is considered to be confined if such confining members frame into all faces of the joint.

21.5.3.2 — For lightweight aggregate concrete, the nominal shear strength of the joint shall not exceed three-quarters of the limits given in 21.5.3.1.

21.5.4 — Development length of bars in tension

21.5.4.1 — The development length ℓ_{dh} for a bar with a standard 90-deg hook in normalweight aggregate concrete shall not be less than the largest of $8d_b$, 6 in., and the length required by Eq. (21-6)

$$\ell_{dh} = f_y d_b/(65\sqrt{f_c'}) \qquad (21\text{-}6)$$

for bar sizes No. 3 through No. 11.

For lightweight aggregate concrete, the development length for a bar with a standard 90 deg hook shall not be less than the largest of $10d_b$, 7-1/2 in., and 1.25 times that required by Eq. (21-6).

The 90-deg hook shall be located within the confined core of a column or of a boundary element.

21.5.4.2 — For bar sizes No. 3 through No. 11, the development length ℓ_d for a straight bar shall not be less than (a) and (b):

(a) 2.5 times the length required by 21.5.4.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 12 in.; and

COMMENTARY

Section 21.5.2.3 refers to a joint where the width of the girder exceeds the corresponding column dimension. In that case, girder reinforcement not confined by the column reinforcement should be provided lateral support either by a girder framing into the same joint or by transverse reinforcement.

R21.5.3 — Shear strength

The requirements in Chapter 21 for proportioning joints are based on Reference 21.18 in that behavioral phenomena within the joint are interpreted in terms of a nominal shear strength of the joint. Because tests of joints[21.27] and deep beams[21.19] indicated that shear strength was not as sensitive to joint (shear) reinforcement as implied by the expression developed by ACI Committee 326[21.37] for beams and adopted to apply to joints by ACI Committee 352,[21.18] Committee 318 set the strength of the joint as a function of only the compressive strength of the concrete (see 21.5.3) and to require a minimum amount of transverse reinforcement in the joint (see 21.5.2). The effective area of joint A_j is illustrated in Fig. R21.5.3. In no case is A_j greater than the column cross-sectional area.

The three levels of shear strength required by 21.5.3.1 are based on the recommendation of ACI Committee 352.[21.18] Test data reviewed by the committee[21.35] indicate that the lower value given in 21.5.3.1 of the 1983 318 code was unconservative when applied to corner joints.

R21.5.4 — Development length of bars in tension

Minimum development length for deformed bars with standard hooks embedded in normalweight concrete is determined

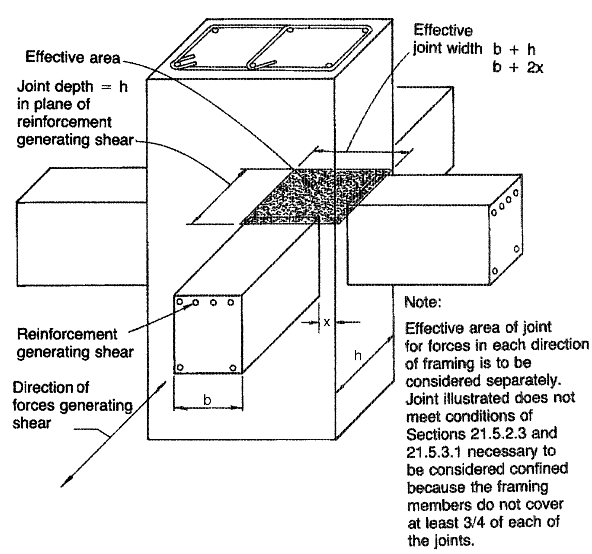

Fig. R21.5.3—Effective joint area.

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(b) 3.5 times the length required by 21.5.4.1 if the depth of the concrete cast in one lift beneath the bar exceeds 12 in.

21.5.4.3 — Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary element. Any portion of the straight embedment length not within the confined core shall be increased by a factor of 1.6.

21.5.4.4 — If epoxy-coated reinforcement is used, the development lengths in 21.5.4.1 through 21.5.4.3 shall be multiplied by the applicable factor in 12.2.4 or 12.5.2.

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using Eq. (21-6). Equation (21-6) is based on the requirements of 12.5. Because Chapter 21 stipulates that the hook is to be embedded in confined concrete, the coefficients 0.7 (for concrete cover) and 0.8 (for ties) have been incorporated in the constant used in Eq. (21-6). The development length that would be derived directly from 12.5 is increased to reflect the effect of load reversals.

The development length in tension for a reinforcing bar with a standard hook is defined as the distance, parallel to the bar, from the critical section (where the bar is to be developed) to a tangent drawn to the outside edge of the hook. The tangent is to be drawn perpendicular to the axis of the bar (Fig. R12.5).

Factors such as the actual stress in the reinforcement being more than the yield force and the effective development length not necessarily starting at the face of the joint were implicitly considered in the development of the expression for basic development length that has been used as the basis for Eq. (21-6).

For lightweight aggregate concrete, the length required by Eq. (21-6) is to be increased by 25 percent to compensate for variability of bond characteristics of reinforcing bars in various types of lightweight aggregate concrete.

Section 21.5.4.2 specifies the minimum development length for straight bars as a multiple of the length indicated by 21.5.4.1. Section 21.5.4.2(b) refers to "top" bars.

If the required straight embedment length of a reinforcing bar extends beyond the confined volume of concrete (as defined in 21.3.3, 21.4.4, or 21.5.2), the required development length is increased on the premise that the limiting bond stress outside the confined region is less than that inside

$$\ell_{dm} = 1.6(\ell_d - \ell_{dc}) + \ell_{dc}$$

or

$$\ell_{dm} = 1.6\ell_d - 0.6\ell_{dc}$$

where

ℓ_{dm} = required development length if bar is not entirely embedded in confined concrete;

ℓ_d = required development length for straight bar embedded in confined concrete (see 21.5.4.3);

ℓ_{dc} = length of bar embedded in confined concrete.

Lack of reference to No. 14 and No. 18 bars in 21.5.4 is due to the paucity of information on anchorage of such bars subjected to load reversals simulating earthquake effects.

21.6 — Special moment frames constructed using precast concrete

21.6.1 — Special moment frames with ductile connections constructed using precast concrete shall satisfy the requirements of (a) and (b) and all requirements

R21.6 — Special moment frames constructed using precast concrete

The detailing provisions in 21.6.1 and 21.6.2 are intended to produce frames that respond to design displacements essentially like monolithic special moment frames.

CODE

for special moment frames constructed with cast-in-place concrete:

(a) The nominal shear strength for connections V_n, computed according to 11.7.4, shall be greater than or equal to $2V_e$, where V_e is calculated according to 21.3.4.1 or 21.4.5.1; and

(b) Mechanical splices of beam reinforcement shall be located not closer than $h/2$ from the joint face and shall meet the requirements of 21.2.6.

21.6.2 — Special moment frames with strong connections constructed using precast concrete shall satisfy all requirements for special moment frames constructed with cast-in-place concrete, as well as the requirements of (a), (b), (c), and (d).

(a) Provisions of 21.3.1.2 shall apply to segments between locations where flexural yielding is intended to occur due to design displacements;

(b) Design strength of the strong connection ϕS_n shall be not less than S_e;

(c) Primary longitudinal reinforcement shall be made continuous across connections and shall be developed outside both the strong connection and the plastic hinge region; and

(d) Column-to-column connections shall have design strength ϕS_n not less than $1.4S_e$. At column-to-column connections, the design flexural strength ϕM_n shall be not less than 0.4 times the maximum probable flexural strength M_{pr} for the column within the story height, and the design shear strength ϕV_n of the connection shall be not less than that determined by 21.4.5.1.

COMMENTARY

Precast frame systems composed of concrete elements with ductile connections are expected to experience flexural yielding in connection regions. Reinforcement in ductile connections can be made continuous by using Type 2 mechanical splices or any other technique that provides development in tension or compression of at least 125 percent of the specified yield strength f_y of bars and the specified tensile strength of bars.[21.38-21.41] Requirements for mechanical splices are in addition to those in 21.2.6 and are intended to avoid strain concentrations over a short length of reinforcement adjacent to a splice device. Additional requirements for shear strength are provided in 21.6.1 to prevent sliding on connection faces. Precast frames composed of elements with ductile connections may be designed to promote yielding at locations not adjacent to the joints. Therefore, design shear V_e, as computed according to 21.3.4.1 or 21.4.5.1, may be conservative.

Precast concrete frame systems composed of elements joined using strong connections are intended to experience flexural yielding outside the connections. Strong connections include the length of the coupler hardware as shown in Fig. R21.6.2. Capacity-design techniques are used in 21.6.2(b) to ensure the strong connection remains elastic following formation of plastic hinges. Additional column requirements are provided to avoid hinging and strength deterioration of column-to-column connections.

Strain concentrations have been observed to cause brittle fracture of reinforcing bars at the face of mechanical splices in laboratory tests of precast beam-column connections.[21.42] Designers should carefully select locations of strong connections or take other measures, such as debonding of reinforcing bars in highly stressed regions, to avoid strain concentrations that can result in premature fracture of reinforcement.

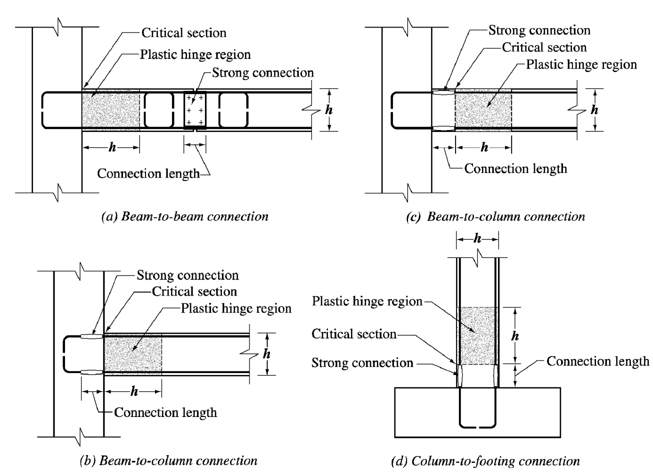

Fig. R21.6.2—Strong connection examples.

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21.6.3 — Special moment frames constructed using precast concrete and not satisfying the requirements of 21.6.1 or 21.6.2 shall satisfy the requirements of ACI T1.1, "Acceptance Criteria for Moment Frames Based on Structural Testing," and the requirements of (a) and (b):

(a) Details and materials used in the test specimens shall be representative of those used in the structure; and

(b) The design procedure used to proportion the test specimens shall define the mechanism by which the frame resists gravity and earthquake effects, and shall establish acceptance values for sustaining that mechanism. Portions of the mechanism that deviate from code requirements shall be contained in the test specimens and shall be tested to determine upper bounds for acceptance values.

21.7 — Special reinforced concrete structural walls and coupling beams

21.7.1 — Scope

The requirements of this section apply to special reinforced concrete structural walls and coupling beams serving as part of the earthquake force-resisting system.

21.7.2 — Reinforcement

21.7.2.1 — The distributed web reinforcement ratio ρ_v and ρ_n, for structural walls shall not be less than 0.0030 along the longitudinal and transverse axes. If the design shear force does not exceed $A_{cv}\sqrt{f_c'}$, the minimum reinforcement for structural walls shall be in conformance with 14.3. Reinforcement spacing each way in structural walls shall not exceed 12 in. Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane.

21.7.2.2 — At least two curtains of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $2A_{cv}\sqrt{f_c'}$.

21.7.2.3 — All continuous reinforcement in structural walls shall be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in 21.5.4.

21.7.2.4 — Tension lap splices for circumferential reinforcing bars in circular reinforced concrete tanks shall be staggered as required in 12.15.2.

COMMENTARY

R21.6.3 — Precast frame systems not satisfying the prescriptive requirements of Chapter 21 have been demonstrated in experimental studies to provide satisfactory seismic performance characteristics.[21.43,21.44] ACI T1.1 defines a protocol for establishing a design procedure, validated by analysis and laboratory tests, for such frames. The design procedure should identify the load path or mechanism by which the frame resists gravity and earthquake effects. The tests should be configured to test critical behaviors, and the measured quantities should establish upper-bound acceptance values for components of the load path, which may be in terms of limiting stresses, forces, strains, or other quantities. The design procedure used for the structure should not deviate from that used to design the test specimens, and acceptance values should not exceed values that were demonstrated by the tests to be acceptable. Materials and components used in the structure should be similar to those used in the tests. Deviations may be acceptable if the engineer can demonstrate that those deviations do not adversely affect the behavior of the framing system.

R21.7 — Special reinforced concrete structural walls and coupling beams

R21.7.1 — Scope

This section contains requirements for the dimensions and details of special reinforced concrete structural walls and coupling beams. In the ACI 318-95 code, 21.6 also contained provisions for diaphragms. Provisions for diaphragms are in 21.9.

R21.7.2 — Reinforcement

Minimum reinforcement requirements (see 21.7.2.1) follow from preceding codes. The uniform distribution requirement of the shear reinforcement is related to the intent to control the width of inclined cracks. The requirement for two layers of reinforcement in walls carrying substantial design shears (21.7.2.2) is based on the observation that, under ordinary construction conditions, the probability of maintaining a single layer of reinforcement near the middle of the wall section is quite low. Furthermore, presence of reinforcement close to the surface tends to inhibit fragmentation of the concrete in the event of severe cracking during an earthquake.

Because the actual forces in longitudinal reinforcing bars of stiff members may exceed the calculated forces, it is required (21.7.2.3) that all continuous reinforcement be developed fully.

CODE

21.7.3 — Design forces

The design shear force V_u shall be obtained from the lateral load analysis in accordance with the factored load combinations.

21.7.4 — Shear strength

21.7.4.1 — Nominal shear strength of structural walls shall not exceed

$$V_n = A_{cv}(\alpha_c\sqrt{f_c'} + \rho_n f_y) \qquad (21\text{-}7)$$

where the coefficient α_c is 3.0 for $h_w/\ell_w \leq 1.5$, is 2.0 for $h_w/\ell_w \leq 2.0$, and varies linearly between 3.0 and 2.0 for h_w/ℓ_w between 1.5 and 2.0.

21.7.4.2 — In 21.7.4.1, the value of ratio (h_w/ℓ_w) used for determining V_n for segments of a wall shall be the larger of the ratios for the entire wall and the segment of wall considered.

21.7.4.3 — Walls shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. If the ratio (h_w/ℓ_w) does not exceed 2.0, reinforcement ratio ρ_v shall not be less than reinforcement ratio ρ_n.

21.7.4.4 — Nominal shear strength of all wall piers sharing a common lateral force shall not be assumed to exceed $8A_{cv}\sqrt{f_c'}$, where A_{cv} is the total cross-sectional area, and the nominal shear strength of any one of the individual wall piers shall not be assumed to exceed $10A_{cp}\sqrt{f_c'}$, where A_{cp} represents the cross-sectional area of the pier considered.

21.7.4.5 — Nominal shear strength of horizontal wall segments and coupling beams shall be assumed not to exceed $10A_{cp}\sqrt{f_c'}$, where A_{cp} represents the cross-sectional area of a horizontal wall segment or coupling beam.

COMMENTARY

R21.7.3 — Design forces

Design shears for structural walls are obtained from lateral load analysis with the appropriate load factors. The designer should, however, consider the possibility of yielding in components of such structures, as in the portion of a wall between two window openings, in which case the actual shear may be in excess of the shear indicated by lateral load analysis based on factored design forces.

R21.7.4 — Shear strength

Equation (21-7) recognizes the higher shear strength of walls with high shear-to-moment ratios.[21.18,21.37,21.45] The nominal shear strength is given in terms of the net area of the section resisting shear. For a rectangular section without openings, the term A_{cv} refers to the gross area of the cross section rather than to the product of the width and the effective depth. The definition of A_{cv} in Eq. (21-7) facilitates design calculations for walls with uniformly distributed reinforcement and walls with openings.

A wall segment refers to a part of a wall bounded by openings or by an opening and an edge. Traditionally, a vertical wall segment bounded by two window openings has been referred to as a pier.

The ratio h_w/ℓ_w may refer to overall dimensions of a wall, or of a segment of the wall bounded by two openings, or an opening and an edge. The intent of 21.7.4.2 is to make certain that any segment of a wall is not assigned a unit strength larger than that for the whole wall. However, a wall segment with a ratio of h_w/ℓ_w higher than that of the entire wall should be proportioned for the unit strength associated with the ratio h_w/ℓ_w based on the dimensions for that segment.

To restrain the inclined cracks effectively, reinforcement included in ρ_n and ρ_v should be appropriately distributed along the length and height of the wall (see 21.7.4.3). Chord reinforcement provided near wall edges in concentrated amounts for resisting bending moment is not to be included in determining ρ_n and ρ_v. Within practical limits, shear reinforcement distribution should be uniform and at a small spacing.

If the factored shear force at a given level in a structure is resisted by several walls or several piers of a perforated wall, the average unit shear strength assumed for the total available cross-sectional area is limited to $8\sqrt{f_c'}$, with the additional requirement that the unit shear strength assigned to any single pier does not exceed $10\sqrt{f_c'}$. The upper limit of strength to be assigned to any one member is imposed to limit the degree of redistribution of shear force.

"Horizontal wall segments" in 21.7.4.5 refers to wall sections between two vertically aligned openings (see Fig. R21.7.4.5). It is, in effect, a pier rotated through 90 deg. A horizontal wall segment is also referred to as a coupling beam when the openings are aligned vertically over the building height.

CODE

21.7.5 — Design for flexure and axial loads

21.7.5.1 — Structural walls and portions of such walls subject to combined flexural and axial loads shall be designed in accordance with 10.2 and 10.3 except that 10.3.7 and the nonlinear strain requirements of 10.2.2 shall not apply. Concrete and developed longitudinal reinforcement within effective flange widths, boundary elements, and the wall web shall be considered effective. The effects of openings shall be considered.

21.7.5.2 — Unless a more detailed analysis is performed, effective flange widths of flanged sections shall extend from the face of the web a distance equal to the smaller of one-half the distance to an adjacent wall web and 25 percent of the total wall height.

21.7.6 — Boundary elements of special reinforced concrete structural walls

21.7.6.1 — The need for special boundary elements at the edges of structural walls shall be evaluated in accordance with 21.7.6.2 or 21.7.6.3. The requirements of 21.7.6.4 and 21.7.6.5 also shall be satisfied.

COMMENTARY

R21.7.5 — Design for flexure and axial loads

R21.7.5.1 — Flexural strength of a wall or wall segment is determined according to procedures commonly used for columns. Strength should be determined considering the applied axial and lateral forces. Reinforcement concentrated in boundary elements and distributed in flanges and webs should be included in the strength computations based on a strain compatibility analysis. The foundation supporting the wall should be designed to develop the wall boundary and web forces. For walls with openings, the influence of the opening or openings on flexural and shear strengths is to be considered and a load path around the opening or openings should be verified. Capacity design concepts and strut-and-tie models may be useful for this purpose.[21.46]

R21.7.5.2 — Where wall sections intersect to form L-, T-, C-, or other cross-sectional shapes, the influence of the flange on the behavior of the wall should be considered by selecting appropriate flange widths. Tests[21.47] show that effective flange width increases with increasing drift level and the effectiveness of a flange in compression differs from that for a flange in tension. The value used for the effective compression flange width has little impact on the strength and deformation capacity of the wall; therefore, to simplify design, a single value of effective flange width based on an estimate of the effective tension flange width is used in both tension and compression.[21.47]

R21.7.6 — Boundary elements of special reinforced concrete structural walls

R21.7.6.1 — Two design approaches for evaluating detailing requirements at wall boundaries are included in 21.7.6.1. Section 21.7.6.2 allows the use of displacement-based design of walls, in which the structural details are determined directly on the basis of the expected lateral displacements of the wall. The provisions of 21.7.6.3 are similar to those of the 318-95 code, and have been retained because they are conservative for assessing required transverse reinforcement at wall boundaries for many walls. Requirements of 21.7.6.4 and 21.7.6.5 apply to structural walls designed by either 21.7.6.2 or 21.7.6.3.

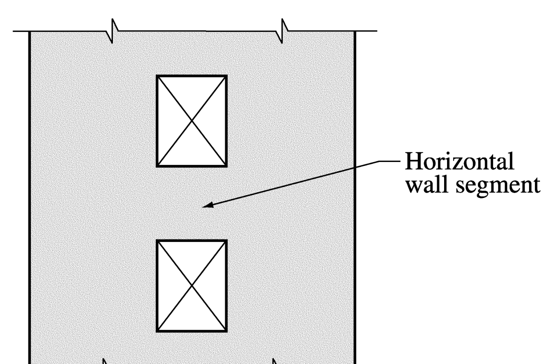

Fig. R21.7.4.5—Wall with openings.

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21.7.6.2 — This section applies to walls or wall piers that are effectively continuous from the base of structure to top of wall and designed to have a single critical section for flexure and axial loads. Walls not satisfying these requirements shall be designed by 21.7.6.3.

(a) Compression zones shall be reinforced with special boundary elements where

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)} \qquad (21\text{-}8)$$

The quantity δ_u/h_w in Eq. (21.8) shall not be taken less than 0.007;

(b) Where special boundary elements are required by 21.7.6.2(a), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of ℓ_w or $M_u/4V_u$.

21.7.6.3 — Structural walls not designed to the provisions of 21.7.6.2 shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to factored forces including earthquake effect, exceeds $0.2f_c'$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f_c'$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in 21.7.5.2 shall be used.

21.7.6.4 — Where special boundary elements are required by 21.7.6.2 or 21.7.6.3, (a) through (f), shall be satisfied:

(a) The boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of $c - 0.1\ell_w$ and $c/2$;

(b) In flanged sections, the boundary element shall include the effective flange width in compression and shall extend at least 12 in. into the web;

(c) Special boundary element transverse reinforcement

COMMENTARY

R21.7.6.2 — Section 21.7.6.2 is based on the assumption that inelastic response of the wall is dominated by flexural action at a critical, yielding section. The wall should be proportioned so that the critical section occurs where intended.

Equation (21-8) follows from a displacement-based approach.[21.48,21.49] The approach assumes that special boundary elements are required to confine the concrete where the strain at the extreme compression fiber of the wall exceeds a critical value when the wall is displaced to the design displacement. The horizontal dimension of the special boundary element is intended to extend at least over the length where the compression strain exceeds the critical value. The height of the special boundary element is based on upper bound estimates of plastic hinge length and extends beyond the zone over which concrete spalling is likely to occur. The lower limit of 0.007 on the quantity δ_u/h_w requires moderate wall deformation capacity for stiff buildings.

The neutral axis depth c in Eq. (21-8) is the depth calculated according to 10.2, except the nonlinear strain requirements of 10.2.2 need not apply, corresponding to development of nominal flexural strength of the wall when displaced in the same direction as δ_u. The axial load is the factored axial load that is consistent with the design load combination that produces the displacement δ_u.

R21.7.6.3 — By this procedure, the wall is considered to be acted on by gravity loads W and the maximum shear and moment induced by earthquake in a given direction. Under this loading, the compressed boundary at the critical section resists the tributary gravity load plus the compressive resultant associated with the bending moment.

Recognizing that this loading condition may be repeated many times during the strong motion, the concrete is to be confined where the calculated compressive stresses exceed a nominal critical value equal to $0.2f_c'$. This stress is calculated for the factored forces on the section assuming linear response of the gross concrete section. The compressive stress of $0.2f_c'$ is used as an index value and does not necessarily describe the actual state of stress that may develop at the critical section under the influence of the actual inertia forces for the anticipated earthquake intensity.

R21.7.6.4 — The value of $c/2$ in 21.7.6.4(a) is to provide a minimum length of the special boundary element. Where flanges are heavily stressed in compression, the web-to-flange interface is likely to be heavily stressed and may sustain local crushing failure unless special boundary element reinforcement extends into the web. Equation (21-3) does not apply to walls.

Because horizontal reinforcement is likely to act as web reinforcement in walls requiring boundary elements, it should be fully anchored in boundary elements that act as flanges (21.7.6.4). Achievement of this anchorage is difficult when large transverse cracks occur in the boundary

CODE

shall satisfy the requirements of 21.4.4.1 through 21.4.4.3, except Eq. (21-3) need not be satisfied;

(d) Special boundary element transverse reinforcement at the wall base shall extend into the support at least the development length of the largest longitudinal reinforcement in the special boundary element unless the special boundary element terminates on a footing or mat, where special boundary element transverse reinforcement shall extend at least 12 in. into the footing or mat;

(e) Horizontal reinforcement in the wall web shall be anchored to develop the specified yield strength f_y within the confined core of the boundary element;

(f) Mechanical splices of longitudinal reinforcement of boundary elements shall conform to 21.2.6. Welded splices of longitudinal reinforcement of boundary elements shall conform to 21.2.7.

21.7.6.5 — Where special boundary elements are not required by 21.7.6.2 or 21.7.6.3, (a) and (b) shall be satisfied:

(a) If the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$, boundary transverse reinforcement shall satisfy 21.4.4.1(c), 21.4.4.3, and 21.7.6.4(a). The maximum longitudinal spacing of transverse reinforcement in the boundary shall not exceed 8 in.;

(b) Except when V_u in the plane of the wall is less than $A_{cv}\sqrt{f_c'}$, horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

21.7.6.6 — Mechanical and welded splices of longitudinal reinforcement of boundary elements shall conform to 21.2.6 and 21.2.7.

COMMENTARY

elements. Therefore, standard 90-deg hooks or mechanical anchorage schemes are recommended instead of straight bar development.

R21.7.6.5 — Cyclic load reversals may lead to buckling of boundary longitudinal reinforcement even in cases where the demands on the boundary of the wall do not require special boundary elements. For walls with moderate amounts of boundary longitudinal reinforcement, ties are required to inhibit buckling. The longitudinal reinforcement ratio is intended to include only the reinforcement at the wall boundary as indicated in Fig. R21.7.6.5. A larger spacing of ties relative to 21.7.6.4(c) is allowed due to the lower deformation demands on the walls.

The addition of hooks or U-stirrups at the ends of horizontal wall reinforcement provides anchorage so that the reinforcement will be effective in resisting shear forces. It will also tend to inhibit the buckling of the vertical edge reinforcement. In walls with low in-plane shear, the development of horizontal reinforcement is not necessary.

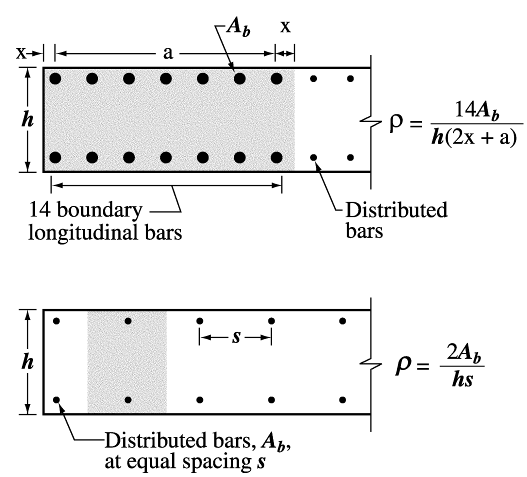

Fig. R21.7.6.5—Longitudinal reinforcement ratios for typical wall boundary conditions.

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21.7.7 — Coupling beams

21.7.7.1 — Coupling beams with aspect ratio $\ell_n/h \geq 4$ shall satisfy the requirements of 21.3. The provisions of 21.3.1.3 and 21.3.1.4(a) shall not be required if it can be shown by analysis that the beam has adequate lateral stability.

21.7.7.2 — Coupling beams with aspect ratio $\ell_n/h \geq 4$ shall be permitted to be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan.

21.7.7.3 — Coupling beams with aspect ratio $\ell_n/h < 2$ and with factored shear force V_u exceeding $4\sqrt{f_c'}A_{cp}$ shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load carrying capacity of the structure, or the egress from the structure, or the integrity of nonstructural components and their connections to the structure.

21.7.7.4 — Coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan shall satisfy the following:

(a) Each group of diagonally placed bars shall consist of a minimum of four bars assembled in a core having sides measured to the outside of transverse reinforcement no smaller than $b_w/2$ perpendicular to the plane of the beam and $b_w/5$ in the plane of the beam and perpendicular to the diagonal bars;

(b) The nominal shear strength V_n shall be determined by

$$V_n = 2A_{vd}f_y\sin\alpha \leq 10\sqrt{f_c'}A_{cp} \quad (21\text{-}9)$$

(c) Each group of diagonally placed bars shall be enclosed in transverse reinforcement satisfying 21.4.4.1 through 21.4.4.3. For the purpose of computing A_g for use in Eq. (10-7) and (21-3), the minimum concrete cover as required in 7.7 shall be assumed on all four sides of each group of diagonally placed reinforcing bars;

(d) The diagonally placed bars shall be developed for tension in the wall;

(e) The diagonally placed bars shall be considered to contribute to nominal flexural strength of the coupling beam;

(f) Reinforcement parallel and transverse to the longitudinal axis shall be provided and, as a minimum, shall conform to 11.8.9 and 11.8.10.

COMMENTARY

R21.7.7 — Coupling beams

Coupling beams connecting structural walls can provide stiffness and energy dissipation. In many cases, geometric limits result in coupling beams that are deep in relation to their clear span. Deep coupling beams may be controlled by shear and may be susceptible to strength and stiffness deterioration under earthquake loading. Test results[21.50,21.51] have shown that confined diagonal reinforcement provides adequate resistance in deep coupling beams.

Experiments show that diagonally oriented reinforcement is effective only if the bars are placed with a large inclination. Therefore, diagonally reinforced coupling beams are restricted to beams having an aspect ratio $\ell_n/h < 4$.

Each diagonal element consists of a cage of longitudinal and transverse reinforcement as shown in Fig. R21.7.7. The cage contains at least four longitudinal bars and confines a concrete core. The requirement on side dimensions of the cage and its core is to provide adequate toughness and stability to the cross section when the bars are loaded beyond yielding. The minimum dimensions and required reinforcement clearances may control the wall width.

When coupling beams are not used as part of the lateral force resisting system, the requirements for diagonal reinforcement may be waived. Nonprestressed coupling beams are permitted at locations where damage to these beams does not impair vertical load-carrying capacity or egress of the structure, or integrity of the nonstructural components and their connections to the structure.

Tests in Reference 21.52 demonstrated that beams reinforced as described in Section 21.7.7 have adequate ductility at shear forces exceeding $10\sqrt{f_c'}b_wd$. Consequently, the use of a limit of $10\sqrt{f_c'}b_wd$ provides an acceptable upper limit.

When the diagonally oriented reinforcement is used, additional reinforcement in 21.7.7.4(f) is to contain the concrete outside the diagonal cores if the concrete is damaged by earthquake loading (Fig. R21.7.7).

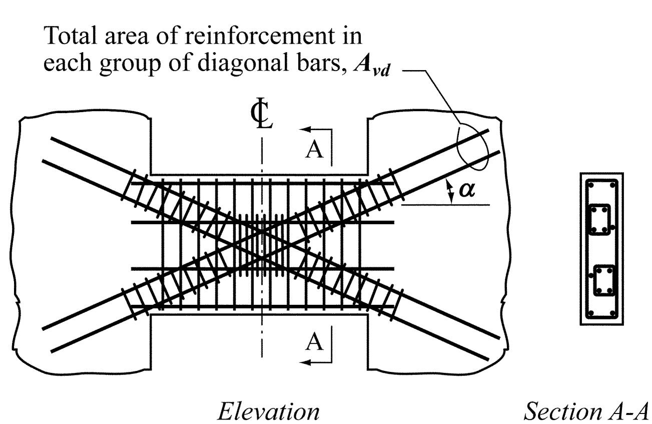

Fig. R21.7.7—Coupling beam with diagonally oriented reinforcement.

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21.7.8 — Construction joints

All construction joints in structural walls shall conform to 6.4 and contact surfaces shall be roughened as specified in 11.7.9.

21.7.9 — Discontinuous walls

Columns supporting discontinuous structural walls shall be reinforced in accordance with 21.4.4.5.

21.7.10 — Wall-to-slab joints

21.7.10.1 — The stresses induced by the transfer of forces between the diaphragms and walls shall be considered.

21.7.10.2 — At joints between diaphragms and walls where shear is transferred, shear reinforcement $A_v f_y$ shall be provided in accordance with Chapter 11, unless other shear transfer mechanisms are available or provided.

21.7.10.3 — Provisions shall be made to accommodate the maximum wave oscillation generated by earthquake acceleration.

21.8 — Special structural walls constructed using precast concrete

21.8.1 — Special structural walls constructed using precast concrete shall satisfy all requirements of 21.7 for cast-in-place special structural walls in addition to 21.13.2 and 21.13.3.

21.9 — Structural diaphragms and trusses

21.9.1 — Scope

Floor and roof slabs acting as structural diaphragms to transmit design actions induced by earthquake ground motions shall be designed in accordance with this section. This section also applies to struts, ties, chords, and collector elements that transmit forces induced by earthquakes, as well as trusses serving as parts of the earthquake force-resisting systems.

COMMENTARY

R21.7.8 — Construction joints

For details of control joints and construction joints in walls of liquid-containing structures, see Reference 21.35 and Chapter 6.

R21.7.10 — Wall-to-slab joints

R.21.7.10.1 — For liquid-containing structures such as tanks, particular attention must be given to wall-to-floor and wall-to-roof joints where forces induced by earthquakes may cause cracking and leakage.

R21.7.10.3 — The horizontal earthquake acceleration causes the container fluid to slosh, causing a vertical displacement of the liquid surface. When the maximum displacement is such that an upward pressure is created on the roof slab, the resulting stresses must be accounted for. The guidelines given in Reference 21.52 for computing the maximum sloshing height may be used.

R21.9 — Structural diaphragms and trusses

R21.9.1 — Scope

Diaphragms as used in building construction are structural elements (such as a floor or roof) that provide some or all of the following functions:

(a) Support for building elements (such as walls, partitions, and cladding) resisting horizontal forces but not acting as part of the building vertical lateral-force-resisting system;

(b) Transfer of lateral forces from the point of application to the building vertical lateral-force-resisting system;

(c) Connection of various components of the building vertical lateral-force-resisting system with appropriate strength, stiffness, and toughness so the building responds as intended in the design.[21.53]

CODE

21.9.2 — Cast-in-place composite-topping slab diaphragms

A composite-topping slab cast-in place on a precast floor or roof shall be permitted to be used as a structural diaphragm provided the topping slab is reinforced and its connections are proportioned and detailed to provide for a complete transfer of forces to chords, collector elements, and the lateral-force-resisting system. The surface of the previously hardened concrete on which the topping slab is placed shall be clean, free of laitance, and intentionally roughened.

21.9.3 — Cast-in-place topping slab diaphragms

A cast-in-place noncomposite topping on a precast floor or roof shall be permitted to serve as a structural diaphragm, provided the cast-in-place topping acting alone is proportioned and detailed to resist the design forces.

21.9.4 — Minimum thickness of diaphragms

Concrete diaphragms and composite topping slabs serving as structural diaphragms used to transmit earthquake forces shall not be less than 2 in. thick. Topping slabs placed over precast floor or roof elements, acting as structural diaphragms and not relying on composite action with the precast elements to resist the design seismic forces, shall have thickness not less than 2-1/2 in.

21.9.5 — Reinforcement

21.9.5.1 — The minimum reinforcement ratio for structural diaphragms shall be in conformance with 7.12. Reinforcement spacing each way in non-post-tensioned floor or roof systems shall not exceed 12 in. Where welded wire fabric is used as the distributed reinforcement to resist shear in topping slabs placed over precast floor and roof elements, the wires parallel to the span of the precast elements shall be spaced not less than 10 in. on center. Reinforcement provided for shear strength shall be continuous and shall be distributed uniformly across the shear plane.

21.9.5.2 — Bonded prestressing steel used as primary reinforcement in diaphragm chords or collectors shall be proportioned such that the stress due to design seismic forces does not exceed 60,000 psi. Precompression from unbonded prestressing steel shall be permitted to resist diaphragm design forces if a complete load path is provided.

21.9.5.3 — Structural truss elements, struts, ties, diaphragm chords, and collector elements with compressive stresses exceeding $0.2 f_c'$ at any section shall have transverse reinforcement, as given in 21.4.4.1 through 21.4.4.3, over the length of the element. The special transverse reinforcement is

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R21.9.2 — Cast-in-place composite-topping slab diaphragms

A bonded topping slab is required so that the floor or roof system can provide restraint against slab buckling. Reinforcement is required to ensure the continuity of the shear transfer across precast joints. The connection requirements are introduced to promote a complete system with necessary shear transfers.

R21.9.3 — Cast-in-place topping slab diaphragms

Composite action between the topping slab and the precast floor elements is not required, provided that the topping slab is designed to resist the design seismic forces.

R21.9.4 — Minimum thickness of diaphragms

The minimum thickness of concrete diaphragms reflects current practice in joist and waffle systems and composite topping slabs on precast floor and roof systems. Thicker slabs are required when the topping slab does not act compositely with the precast system to resist the design seismic forces.

R21.9.5 — Reinforcement

Minimum reinforcement ratios for diaphragms correspond to the required amount of temperature and shrinkage reinforcement (7.12). The maximum spacing for web reinforcement is intended to control the width of inclined cracks. Minimum average prestress requirements (7.12.3) are considered to be adequate to limit the crack widths in post-tensioned floor systems; therefore, the maximum spacing requirements do not apply to these systems.

The minimum spacing requirement for welded wire reinforcement in topping slabs on precast floor systems (see 21.9.5.1) is to avoid fracture of the distributed reinforcement during an earthquake. Cracks in the topping slab open immediately above the boundary between the flanges of adjacent precast members, and the wires crossing those cracks are restrained by the transverse wires.[21.54] Therefore, all the deformation associated with cracking should be accommodated in a distance not greater than the spacing of the transverse wires. A minimum spacing of 10 in. for the transverse wires is required in 21.9.5.1 to reduce the likelihood of fracture of the wires crossing the critical cracks during a design earthquake. The minimum spacing requirements do not apply to diaphragms reinforced with individual bars, because strains are distributed over a longer length.

Compressive stress calculated for the factored forces on a

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allowed to be discontinued at a section where the calculated compressive strength is less than $0.15f_c'$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross-section properties of the elements considered.

21.9.5.4 — All continuous reinforcement in diaphragms, trusses, struts, ties, chords, and collector elements shall be anchored or spliced in accordance with the provisions for reinforcement tension as specified in 21.5.4.

21.9.5.5 — Type 2 splices are required where mechanical splices are used to transfer forces between the diaphragm and the vertical components of the lateral-force-resisting system.

21.9.6 — Design forces

The seismic design forces for structural diaphragms shall be obtained from the lateral load analysis in accordance with the design load combinations.

21.9.7 — Shear strength

21.9.7.1 — Nominal shear strength V_n of structural diaphragms shall not exceed

$$V_n = A_{cv}(2\sqrt{f_c'} + \rho_n f_y) \qquad (21\text{-}10)$$

21.9.7.2 — Nominal shear strength V_n of cast-in-place composite-topping slab diaphragms and cast-in-place noncomposite topping slab diaphragms on a precast floor or roof shall not exceed the shear force

$$V_n = A_{cv}\rho_n f_y \qquad (21\text{-}11)$$

where A_{cv} is based on the thickness of the topping slab. The required web reinforcement shall be distributed uniformly in both directions.

21.9.7.3 — Nominal shear strength shall not exceed $8A_{cv}\sqrt{f_c'}$ where A_{cv} is the gross cross-sectional area of the diaphragm.

21.9.8 — Boundary elements of structural diaphragms

21.9.8.1 — Boundary elements of structural diaphragms shall be proportioned to resist the sum of the factored axial forces acting in the plane of the diaphragm and the force obtained from dividing the factored moment at the section by the distance between the boundary elements of the diaphragm at that section.

21.9.8.2 — Splices of tensile reinforcement in the chords and collector elements of diaphragms shall develop the yield strength of the reinforcement.

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linearly elastic model based on gross section of the structural member is used as an index value to determine whether confining reinforcement is required. A calculated compressive stress of $0.2f_c'$ in a member is assumed to indicate that integrity of the entire structure is dependent on the ability of that member to resist substantial compressive force under severe cyclic loading. Therefore, transverse reinforcement in 21.4.4 is required in such members to provide confinement for the concrete and the reinforcement (21.9.5.3).

The dimensions of typical structural diaphragms often preclude the use of transverse reinforcement along the chords. Reducing the calculated compressive stress by reducing the span of the diaphragm is considered to be a solution.

R21.9.7 — Shear strength

The shear strength requirements for monolithic diaphragms, Eq. (21-10) in 21.9.7.1, are the same as those for slender structural walls. The term A_{cv} refers to the thickness times the width of the diaphragm. This corresponds to the gross area of the effective deep beam that forms the diaphragm. The shear reinforcement should be placed perpendicular to the span of the diaphragm.

The shear strength requirements for topping slab diaphragms are based on a shear friction model, and the contribution of the concrete to the nominal shear strength is not included in Eq. (21-9) for topping slabs placed over precast floor elements. Following typical construction practice, the topping slabs are roughened immediately above the boundary between the flanges of adjacent precast floor members to direct the paths of shrinkage cracks. As a result, critical sections of the diaphragm are cracked under service loads, and the contribution of the concrete to the shear capacity of the diaphragm may have already been reduced before the design earthquake occurs.

R21.9.8 — Boundary elements of structural diaphragms

For structural diaphragms, the design moments are assumed to be resisted entirely by chord forces acting at opposite edges of the diaphragm. Reinforcement located at the edges of collectors should be fully developed for its yield strength. Adequate confinement of lap splices is also required. If chord reinforcement is located within a wall, the joint between the diaphragm and the wall should be provided with adequate shear strength to transfer the shear forces.

Section 21.9.8.3 is intended to reduce the possibility of chord buckling in the vicinity of splices and anchorage zones.

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Mechanical and welded splices shall conform to 21.2.6 and 21.2.7, respectively.

21.9.8.3 — Reinforcement for chords and collectors at splices and anchorage zones shall have either:

(a) A minimum center-to-center spacing of three longitudinal bar diameters, but not less than 1-1/2 in., and a minimum concrete clear cover of two and one-half longitudinal bar diameters, but not less than 2 in.; or

(b) Transverse reinforcement as required by 11.5.5.3, except as required in 21.9.5.3.

21.9.9 — Construction joints

All construction joints in diaphragms shall conform to 6.4 and contact surfaces shall be roughened as in 11.7.9.

21.10 — Foundations

21.10.1 — Scope

21.10.1.1 — Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground shall comply with 21.10 and other applicable code provisions.

21.10.1.2 — The provisions in this section for piles, drilled piers, caissons, and slabs on grade shall supplement other applicable code design and construction criteria. See 1.1.5 and 1.1.6.

21.10.2 — Footings, foundation mats, and pile caps

21.10.2.1 — Longitudinal reinforcement of columns and structural walls resisting forces induced by earthquake effects shall extend into the footing, mat, or pile cap, and shall be fully developed for tension at the interface.

21.10.2.2 — Columns designed assuming fixed-end conditions at the foundation shall comply with 21.10.2.1 and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90-deg hooks near the bottom of the foundation with the free end of the bars oriented towards the center of the column.

21.10.2.3 — Columns or boundary elements of special reinforced concrete structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with 21.4.4 provided below the top of the footing. This reinforcement shall extend into the footing a distance no less than the smaller of the depth of the footing, mat, or pile cap, or the development length in tension of the longitudinal reinforcement.

21.10.2.4 — Where earthquake effects create uplift forces in boundary elements of special reinforced concrete structural walls or structural moment frames, flexural reinforcement shall be provided in the top of

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R21.10 — Foundations

R21.10.1 — Scope

Requirements for foundations supporting buildings assigned to high seismic performance or design categories were added to the ACI 318-99 code. They represent a consensus of a minimum level of good practice in designing and detailing concrete foundations including piles, drilled piers, and caissons. It is desirable that inelastic response in strong ground shaking occurs above the foundations, as repairs to foundations can be extremely difficult and expensive.

R21.10.2 — Footings, foundation mats, and pile caps

R21.10.2.2 — Tests[21.55] have demonstrated that flexural members terminating in a footing, slab, or beam (a T-joint) should have their hooks turned inwards toward the axis of the member for the joint to be able to resist the flexure in the member forming the stem of the T.

R21.10.2.3 — Columns or boundary members supported close to the edge of the foundation, as often occurs near property lines, should be detailed to prevent an edge failure of the footing, pile cap, or mat.

R21.10.2.4 — The purpose of 21.10.2.4 is to alert the designer to provide top reinforcement as well as other required reinforcement.

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the footing, mat, or pile cap to resist the design load combinations, and shall not be less than required by 10.5.

21.10.3 — Grade beams and slabs-on-grade

21.10.3.1 — Grade beams designed to act as horizontal ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supported column or anchored within the pile cap or footing at all discontinuities.

21.10.3.2 — Grade beams designed to act as horizontal ties between pile caps or footings shall be proportioned such that the smallest cross-sectional dimension shall be equal to or greater than the clear spacing between connected columns divided by 20, but need not be greater than 18 in. Closed ties shall be provided at a spacing not to exceed the lesser of one-half the smallest orthogonal cross-sectional dimension or 12 in.

21.10.3.3 — Grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the lateral-force-resisting system shall conform to 21.3.

21.10.3.4 — Slabs-on-grade that resist seismic forces from walls or columns that are part of the lateral-force-resisting system shall be designed as structural diaphragms in accordance with 21.9. The design drawings shall clearly state that the slab on grade is a structural diaphragm and part of the lateral-force-resisting system.

21.10.4 — Piles, piers, and caissons

21.10.4.1 — Provisions of 21.10.4 shall apply to concrete piles, piers, and caissons supporting structures designed for earthquake resistance.

21.10.4.2 — Piles, piers, or caissons resisting tension loads shall have continuous longitudinal reinforcement over the length resisting design tension forces. The longitudinal reinforcement shall be detailed to transfer tension forces within the pile cap to supported structural members.

21.10.4.3 — Where tension forces induced by earthquake effects are transferred between pile cap or mat foundation and precast pile by reinforcing bars grouted or post-installed in the top of the pile, the grouting system shall have been demonstrated by test to develop at least 125 percent of the specified yield strength of the bar.

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R21.10.3 — Grade beams and slabs on grade

For seismic conditions, slabs on grade (soil-supported slabs) are often part of the lateral-force-resisting system and should be designed in accordance with this code as well as other appropriate standards or guidelines. See 1.1.6.

R21.10.3.2 — Grade beams between pile caps or footings can be separate beams beneath the slab on grade or can be a thickened portion of the slab on grade. The cross-sectional limitation and minimum tie requirements provide reasonable proportions.

R21.10.3.3 — Grade beams resisting seismic flexural stresses from column moments should have reinforcing details similar to the beams of the frame above the foundation.

R21.10.3.4 — Slabs-on-grade often act as a diaphragm to hold the structure together at the ground level and minimize the effects of out-of-phase ground motion that may occur over the footprint of the structure. In these cases, the diaphragm should be adequately reinforced and detailed. The design drawings should clearly state that these diaphragms are structural members so as to prohibit sawcutting of the slab.

R21.10.4 — Piles, piers, and caissons

Adequate performance of piles and caissons for seismic loadings requires that these provisions be met in addition to other applicable standards or guidelines. See R1.1.5.

R21.10.4.2 — A load path is necessary at pile caps to transfer tension forces from the reinforcing bars in the column or boundary member through the pile cap to the reinforcement of the pile or caisson.

R21.10.4.3 — Grouted dowels in a blockout in the top of a precast concrete pile need to be developed, and testing is a practical means of demonstrating capacity. Alternatively, reinforcing bars can be cast in the upper portion of the pile, exposed by chipping of concrete and mechanically connected or welded to an extension.

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21.10.4.4 — Piles, piers, or caissons shall have transverse reinforcement in accordance with 21.4.4 at locations (a) and (b):

(a) At the top of the member for at least five times the member cross-sectional dimension, but not less than 6 ft below the bottom of the pile cap;

(b) For the portion of piles in soil that is not capable of providing lateral support, or in air and water, along the entire unsupported length plus the length required in 21.10.4.4(a).

21.10.4.5 — For precast concrete driven piles, the length of transverse reinforcement provided shall be sufficient to account for potential variations in the elevation in pile tips.

21.10.4.6 — Pile caps incorporating batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns. The slenderness effects of batter piles shall be considered for the portion of the piles in soil that is not capable of providing lateral support, or in air or water.

21.11 — Frame members not proportioned to resist forces induced by earthquake motions

21.11.1 — Frame members assumed not to contribute to lateral resistance shall be detailed according to 21.11.2 or 21.11.3 depending on the magnitude of moments induced in those members if subjected to the design displacement. If effects of design displacements are not explicitly checked, it shall be permitted to apply the requirements of 21.11.3.

21.11.2 — When the induced moments and shears under design displacements of 21.11.1 combined with the factored gravity moments and shears do not exceed the design moment and shear strength of the frame member, the conditions of 21.11.2.1, 21.11.2.2, and 21.11.2.3 shall be satisfied. The gravity load combinations of **(1.2*D* + 1.0*L* + 0.2*S*)** or **0.9*D***, whichever is critical, shall be used. The load factor on ***L*** shall be permitted to be reduced to 0.5 where it can be justified that no greater than 50 percent of the design live load is expected to be present during normal operating conditions. Reduction of the load factor shall not be permitted in places of public assembly, and all areas where the live load ***L*** is greater than 100 lb/ft^2.

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R21.10.4.4 — During earthquakes, piles can be subjected to extremely high flexural demands at points of discontinuity, especially just below the pile cap and near the base of a soft or loose soil deposit. The ACI 318-99 code requirement for confinement reinforcement at the top of the pile is based on numerous failures observed at this location in recent earthquakes. Transverse reinforcement is required in this region to provide ductile performance. The designer should also consider possible inelastic action in the pile at abrupt changes in soil deposits, such as changes from soft to firm or loose to dense soil layers. Where precast piles are to be used, the potential for the pile tip to be driven to an elevation different than that specified in the drawings needs to be considered when detailing the pile. If the pile reaches refusal at a shallower depth, a longer length of pile will need to be cut off. If this possibility is not foreseen, the length of transverse reinforcement required by 21.10.4.4 may not be available after the excess pile length is cut off.

R21.10.4.6 — Extensive structural damage has often been observed at the junction of batter piles and the buildings. The pile cap and surrounding structure should be designed for the potentially large forces that can be developed in batter piles.

R21.11 — Frame members not proportioned to resist forces induced by earthquake motions

The detailing requirements for members that are part of the lateral-force resisting system assume that the members may undergo deformations that exceed the yield limit of the member without significant loss of strength. Members that are not part of the designated lateral-force-resisting system are not required to meet all the detailing requirements of members that are relied on to resist lateral forces. They should, however, be able to resist the gravity loads at lateral displacements corresponding to the design level prescribed by the governing code for earthquake-resistant design. The design displacement is defined in 21.1.

Section 21.11 recognizes that actual displacements resulting from earthquake forces may be larger than the displacements calculated using the design forces and commonly used analysis models. Section 21.1 defines a nominal displacement for the purpose of prescribing detailing requirements. This section has been revised from the ACI 318-95 code to reflect changes from a working stress design approach to a strength design approach in governing codes for earthquake-resistant design.[21.3] Actual displacements may exceed the value of 21.1. Section 21.11.2 prescribes

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21.11.2.1 — Members with factored gravity axial forces not exceeding $A_g f_c'/10$ shall satisfy 21.3.2.1. Stirrups shall be spaced not more than $d/2$ throughout the length of the member.

21.11.2.2 — Members with factored gravity axial forces exceeding $A_g f_c'/10$ shall satisfy 21.4.3, 21.4.4.1(c), 21.4.4.3, and 21.4.5. The maximum longitudinal spacing of ties shall be s_o for the full column height. The spacing s_o shall not be more than six diameters of the smallest longitudinal bar enclosed or 6 in., whichever is smaller.

21.11.2.3 — Members with factored gravity axial forces exceeding $0.35P_o$ shall satisfy 21.11.2.2 and the amount of transverse reinforcement provided shall be one-half of that required by 21.4.4.1 not to exceed a spacing s_o for the full height of the column.

21.11.3 — If the induced moment or shear under design displacements of 21.11.1 exceed the design moment or shear strength of the frame member, or if induced moments are not calculated, the conditions of 21.11.3.1, 21.11.3.2, and 21.11.3.3 shall be satisfied.

21.11.3.1 — Materials shall satisfy 21.2.4 and 21.2.5. Mechanical splices shall satisfy 21.2.6 and welded splices shall satisfy 21.2.7.1.

21.11.3.2 — Members with factored gravity axial forces not exceeding $A_g f_c'/10$ shall satisfy 21.3.2.1 and 21.3.4. Stirrups shall be spaced at not more than $d/2$ throughout the length of the member.

21.11.3.3 — Members with factored gravity axial forces exceeding $A_g f_c'/10$ shall satisfy 21.4.3, 21.4.4, 21.4.5, and 21.5.2.1.

21.11.4 — Precast concrete frame members assumed not to contribute to lateral resistance, including their connections, shall satisfy (a), (b), and (c), in addition to 21.11.1 through 21.11.3:

(a) Ties specified in 21.11.2.2 shall be provided over the entire column height, including the depth of the beams;

(b) Structural integrity reinforcement, as specified in 16.5, shall be provided; and

(c) Bearing length at support of a beam shall be at least 2 in. longer than determined from calculations using bearing strength values from 10.17.

21.12 — Requirements for intermediate moment frames

21.12.1 — The requirements of this section apply to intermediate moment frames.

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detailing requirements intended to provide a system capable of sustaining gravity loads under moderate excursions into the inelastic range. Section 21.11.3 prescribes detailing requirements intended to provide a system capable of sustaining gravity loads under larger displacements.

Models used to determine design deflections of buildings should be chosen to produce results that conservatively bound the values expected during the design earthquake considering vertical, horizontal, and diaphragm systems as appropriate.

For gravity load factors, see R9.2.

The poor performance of some buildings with precast concrete gravity systems during the Northridge Earthquake was attributed to several factors addressed in 21.11.4. Columns should contain ties over their entire height, frame members not proportioned to resist earthquake forces should be tied together, and longer bearing lengths should be used to maintain integrity of the gravity system during shaking. The 2 in. increase in bearing length is based on an assumed 4 percent story drift ratio and 50 in. beam depth, and is considered to be conservative for the ground motions expected in high seismic zones. In addition to the provisions of 21.11.4, precast frame members assumed not to contribute to lateral resistance should also satisfy 21.11.1 through 21.11.3.

R21.12 — Requirements for intermediate moment frames

The objective of the requirements in 21.12.3 is to reduce the risk of failure in shear during an earthquake. The designer is given two options by which to determine the factored shear force.

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21.12.2 — Reinforcement details in a frame member shall satisfy 21.12.4 if the factored compressive axial load for the member does not exceed $A_g f_c'/10$. If the factored compressive axial load is larger, frame reinforcement details shall satisfy 21.12.5 unless the member has spiral reinforcement according to Eq. (10-5). If a two-way slab system without beams is treated as part of a frame resisting earthquake effect, reinforcement details in any span resisting moments caused by lateral force shall satisfy 21.12.6.

21.12.3 — Design shear strength of beams, columns, and two-way slabs resisting earthquake effect shall not be less than either (a) or (b):

(a) The sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for factored gravity loads;

(b) The maximum shear obtained from design load combinations that include earthquake effect E, with E assumed to be twice that prescribed by the governing code for earthquake-resistant design.

21.12.4 — Beams

21.12.4.1 — The positive moment strength at the face of the joint shall be not less than one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the member shall be less than one-fifth the maximum moment strength provided at the face of either joint.

21.12.4.2 — At both ends of the member, hoops shall be provided over lengths equal to twice the member depth measured from the face of the supporting member toward midspan. The first hoop shall be located at not more than 2 in. from the face of the supporting member. Maximum hoop spacing shall not exceed the smallest of (a), (b), (c), or (d):

(a) $d/4$;

(b) Eight times the diameter of the smallest longitudinal bar enclosed;

(c) 24 times the diameter of the hoop bar;

(d) 12 in.

21.12.4.3 — Stirrups shall be placed at not more than $d/2$ throughout the length of the member.

21.12.5 — Columns

21.12.5.1 — Columns shall be spirally reinforced in accordance with 7.10.4 or shall conform with 21.12.5.2 through 21.12.5.4. Section 21.12.5.5 shall apply to all columns.

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According to option (a) of 21.12.3, the factored shear force is determined from the nominal moment strength of the member and the gravity load on it. Examples for a beam and a column are illustrated in Fig. R21.12.3.

To determine the maximum beam shear, it is assumed that its nominal moment strengths $\phi = 1.0$ are developed simultaneously at both ends of its clear span. As indicated in Fig. R21.12.3, the shear associated with this condition $[(M_{nl} + M_{nr})/\ell_n]$ added algebraically to the effect of the factored gravity loads indicates the design shear of the beam. For this example, both the dead load w_D and the live load w_L have been assumed to be uniformly distributed.

Determination of the design shear for a column is also illustrated for a particular example in Fig. R21.12.3. The factored design axial load P_u should be chosen to develop the largest moment strength of the column.

In all applications of option (a) of 21.12.3, shears are required to be calculated for moment, acting clockwise and counterclockwise. Figure R21.12.3 demonstrates only one

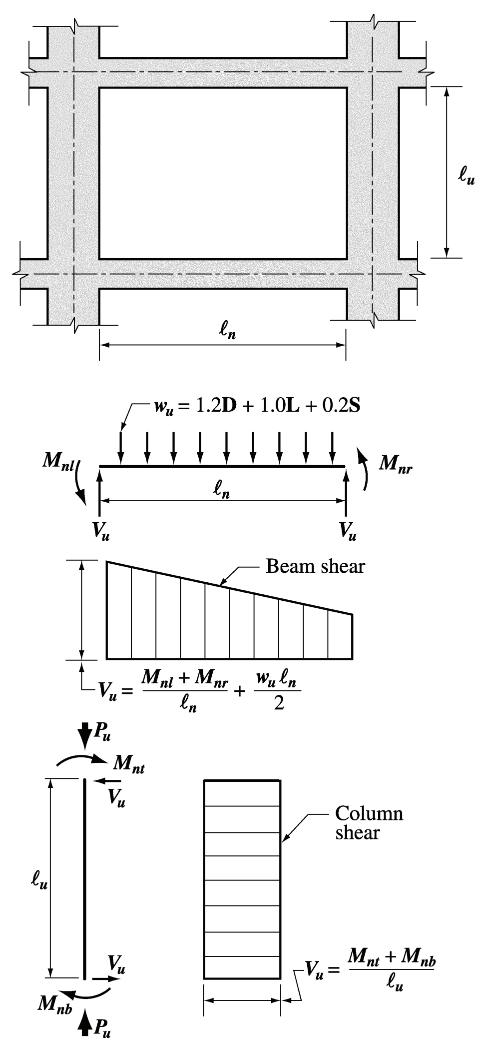

Fig. R21.12.3—Design shears for frames in regions of moderate seismic risk (see 21.12).

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21.12.5.2 — At both ends of the member, hoops shall be provided at spacing s_o over a length ℓ_o measured from the joint face. Spacing s_o shall not exceed the smallest of (a), (b), (c), and (d):

(a) Eight times the diameter of the smallest longitudinal bar enclosed;

(b) 24 times the diameter of the hoop bar;

(c) One-half of the smallest cross-sectional dimension of the frame member;

(d) 12 in.

Length ℓ_o shall not be less than the largest of (e), (f), and (g):

(e) One-sixth of the clear span of the member;

(f) Maximum cross-sectional dimension of the member;

(g) 18 in.

21.12.5.3 — The first hoop shall be located at not more than $s_o/2$ from the joint face.

21.12.5.4 — Outside the length ℓ_o, spacing of transverse reinforcement shall conform to 7.10 and 11.5.4.1.

21.12.5.5 — Joint reinforcement shall conform to 11.11.2.

21.12.6 — Two-way slabs without beams

21.12.6.1 — Factored slab moment at support related to earthquake effect shall be determined for load combinations given in Eq. (9-5) and (9-7). All reinforcement provided to resist M_s, the portion of slab moment balanced by support moment, shall be placed within the column strip defined in 13.2.1.

21.12.6.2 — The fraction, defined by Eq. (13-1), of moment M_s shall be resisted by reinforcement placed within the effective width given in 13.5.3.2. Effective slab width for exterior and corner connections shall not extend beyond the column face a distance greater than c_t measured perpendicular to the slab span.

21.12.6.3 — Not less than one-half of the reinforcement in the column strip at support shall be placed within the effective slab width given in 13.5.3.2.

21.12.6.4 — Not less than one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span.

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of the two conditions that are to be considered for every member. Option (b) bases V_u on the load combination including the earthquake effect E, which should be doubled. For example, the load combination defined by Eq. (9-5) would be

$$U = 1.2D + 2.0E + 1.0L + 0.2S$$

where E is the value specified by the governing code.

Section 21.12.4 contains requirements for providing beams with a threshold level of toughness. Transverse reinforcement at the ends of the beam should be hoops. In most cases, stirrups required by 21.12.3 for design shear force will be more than those required by 21.12.4. Requirements of 21.12.5 serve the same purpose for columns.

Section 21.12.6 applies to two-way slabs without beams, such as flat plates, subject to earthquake effect.

Using load combinations of Eq. (9-5) and (9-7) may result in moments requiring both top and bottom reinforcement at the supports.

The moment M_s refers, for a given design load combination with E acting in one horizontal direction, to that portion of the factored slab moment that is balanced by the supporting members at a joint. It is not necessarily equal to the total design moment at support for a load combination including earthquake effect. In accordance with 13.5.3.2, only a fraction ($\gamma_f M_s$) of the moment M_s is assigned to the slab effective width. For edge and corner connections, flexural reinforcement perpendicular to the edge is not considered fully effective unless it is placed within the effective slab width.[21.56,21.57] See Fig. 21.12.6.1.

Application of the provisions of 21.12.6 is illustrated in Fig. R21.12.6.2 and R21.12.6.3.

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21.12.6.5 — Continuous bottom reinforcement in the column strip shall be not less than one-third of the top reinforcement at the support in the column strip.

21.12.6.6 — Not less than one-half of all bottom middle strip reinforcement and all bottom column strip reinforcement at midspan shall be continuous and shall develop its yield strength at face of support as defined in 13.6.2.5.

21.12.6.7 — At discontinuous edges of the slab all top and bottom reinforcement at support shall be developed at the face of support as defined in 13.6.2.5.

21.12.6.8 — At the critical sections for columns defined in 11.12.1.2, two-way shear caused by factored gravity loads shall not exceed $0.4\phi V_c$, where V_c shall be calculated as defined in 11.12.2.1 for nonprestressed slabs and in 11.12.2.2 for prestressed slabs. It shall be permitted to waive this requirement if the contribution of the earthquake-induced factored two-way shear stress transferred by eccentricity of shear in accordance with 11.12.6.1 and 11.12.6.2 at the point of maximum stress does not exceed one-half of the stress ϕv_n permitted by 11.12.6.2.

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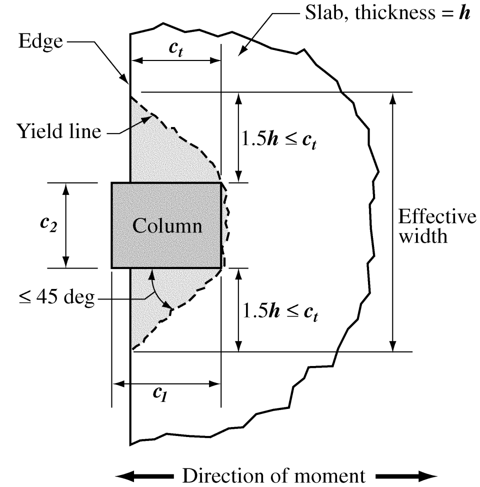
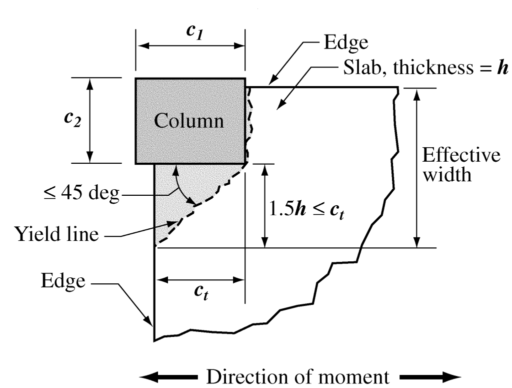
R21.12.6.8 — The requirements apply to two-way slabs that are part of the primary lateral-force-resisting system. Slab-column connections in laboratory tests[21.57] exhibited reduced lateral displacement ductility when the shear at the column connection exceeded the recommended limit.



(a) Edge connection



(b) Corner connection

Fig. R21.12.6.1—Effective width for reinforced placement in edge and corner connections.

CODE

21.13 — Intermediate precast structural walls

21.13.1 — The requirements of this section apply to intermediate precast structural walls used to resist forces induced by earthquake motions.

21.13.2 — In connections between wall panels, or between wall panels and the foundation, yielding shall be restricted to steel elements or reinforcement.

21.13.3 — Elements of the connection that are not designed to yield shall develop at least $1.5S_y$.

COMMENTARY

R21.13 — Intermediate precast structural walls

Connections between precast wall panels or between wall panels and the foundation are required to resist forces induced by earthquake motions and to provide for yielding in the vicinity of connections. When Type 2 mechanical splices are used to directly connect primary reinforcement, the probable strength of the splice should be at least 1-1/2 times the specified yield strength of the reinforcement.

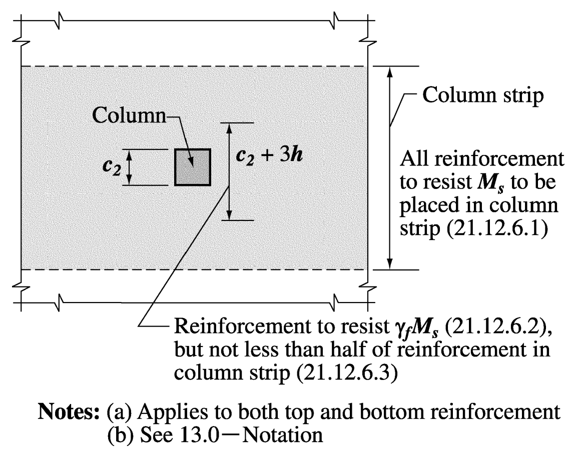


Fig. R21.12.6.2—Location of reinforcement in slabs.

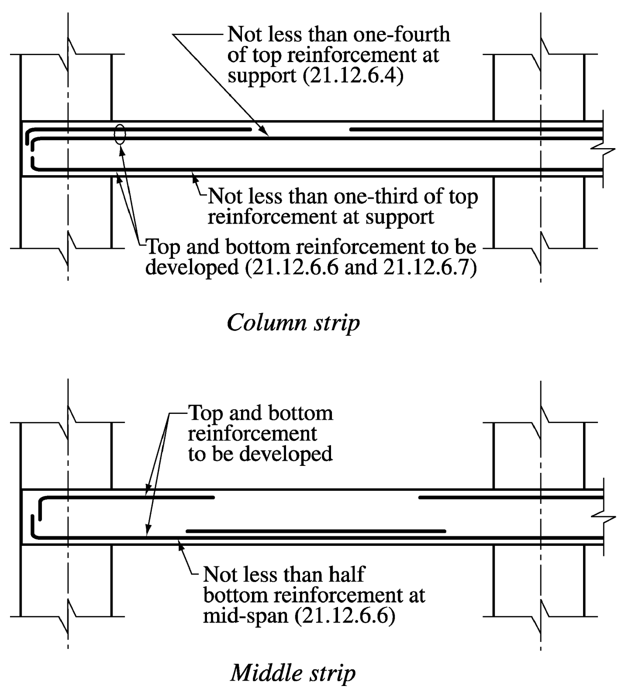


Fig. R21.12.6.3—Arrangement of reinforcement in slabs.

PART 7 — STRUCTURAL PLAIN CONCRETE

CHAPTER 22 — STRUCTURAL PLAIN CONCRETE

CODE

Structural plain concrete is not permitted in environmental engineering concrete structures designed under the requirements of ACI 350.

COMMENTARY

CODE

COMMENTARY

Notes

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Notes

APPENDIX A — NOT USED

CODE

COMMENTARY

CODE

COMMENTARY

Notes

APPENDIX B — ALTERNATIVE PROVISIONS FOR REINFORCED AND PRESTRESSED CONCRETE FLEXURAL AND COMPRESSION MEMBERS

CODE

B.0 — Notation

A_g = gross area of section, in.^{2}

f_c' = specified compressive strength of concrete, psi

f_y = specified yield strength of nonprestressed reinforcement, ksi

d = distance from extreme compression fiber to centroid of tension reinforcement, in.

P_b = nominal axial load strength at balanced strain conditions, lb. See 10.3.2

P_n = nominal axial load strength at given eccentricity, lb

β_1 = factor defined in 10.2.7.3

ρ = reinforcement ratio for nonprestressed tension reinforcement

= A_s/bd

ρ' = reinforcement ratio for nonprestressed compression reinforcement

= A_s'/bd

ρ_b = reinforcement ratio producing balanced strain conditions. See 10.3.2.

ϕ = strength reduction factor

B.1 — Scope

Design for flexure and axial load by provisions of Appendix B shall be permitted. When Appendix B is used in design, B.8.4, B.8.4.1, B.8.4.2, and B.8.4.3 shall replace the corresponding numbered sections in Chapter 8; B.10.3.3 shall replace 10.3.3, 10.3.4, and 10.3.5; if any section in this appendix is used, all sections in this appendix shall be substituted in the body of the code, and all other sections in the body of the code shall be applicable.

B.8.4 — Redistribution of negative moments in continuous nonprestressed flexural members

For criteria on moment redistribution for prestressed concrete members, see 18.10.4.

B.8.4.1 — Except where approximate values for moments are used, it shall be permitted to increase or

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RB.1 — Scope

Reinforcement limits and moment redistribution in Appendix B differ from those in the main body of the code. Appendix B contains the reinforcement limits and moment redistribution used in the code for many years. Designs using the provisions of Appendix B satisfy the code, and are equally acceptable.

When this appendix is used, the corresponding commentary sections apply. The load factors and strength reduction factors of either Chapter 9 or Appendix C are applicable.

RB.8.4 — Redistribution of negative moments in continuous nonprestressed flexural members

Moment redistribution is dependent on adequate ductility in plastic hinge regions. These plastic hinge regions develop at points of maximum moment and cause a shift in the elastic moment diagram. The usual results are reduction in the values of negative moments in the plastic hinge region and an increase in the values of positive moments from those computed by elastic analysis. Because negative moments

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decrease negative moments calculated by elastic theory at supports of continuous flexural members for any assumed loading arrangement by not more than

$$20\left(1 - \frac{\rho - \rho'}{\rho_b}\right) \text{ percent}$$

B.8.4.2 — The modified negative moments shall be used for calculating moments at sections within the spans.

B.8.4.3 — Redistribution of negative moments shall be made only when the section at which moment is reduced is so designed that ρ or $\rho - \rho'$ is not greater than $0.50\rho_b$, where

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{87{,}000}{87{,}000 + f_y}\right) \qquad \text{(B-1)}$$

COMMENTARY

are determined for one loading arrangement and positive moments for another, each section has a reserve capacity that is not fully utilized for any one loading condition. The plastic hinges permit the utilization of the full capacity of more cross sections of a flexural member at ultimate loads. Using conservative values of ultimate concrete strains and lengths of plastic hinges derived from extensive tests, flexural members with small rotation capacity were analyzed for moment redistribution up to 20 percent, depending on the reinforcement ratio. The results were found to be conservative (see Fig. RB.8.4). Studies by Cohn[B.1] and Mattock[B.2] support this conclusion and indicate that cracking and deflection of beams designed for moment redistribution are not significantly greater at service loads than for beams designed by the elastic theory distribution of moments. These studies also indicated that adequate rotation capacity for the moment redistribution allowed by the code is available if the members satisfy the code requirements. This appendix maintains the same limit on redistribution as used in previous code editions.

Moment redistribution does not apply to members designed by the alternate design method of Appendix I; nor may it be used for slab systems designed by the direct design method (see 13.6.1.7).

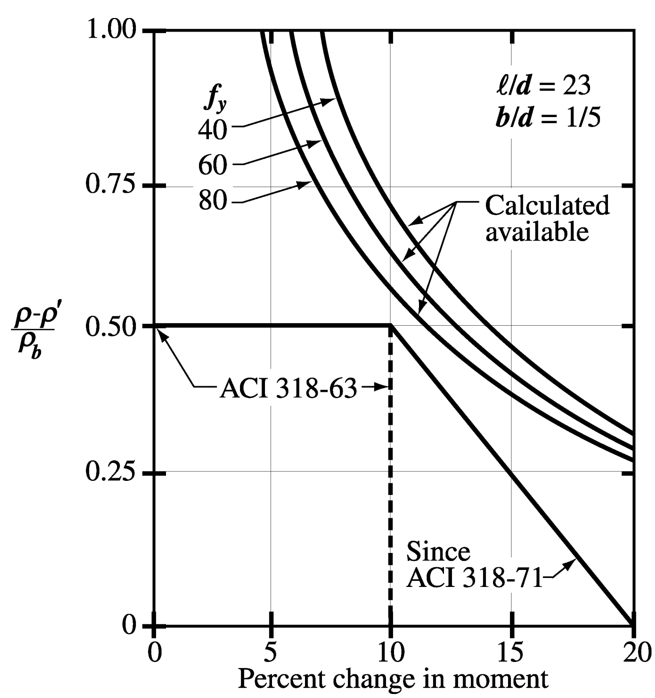


Fig. RB8.4—Permissible moment redistribution for minimum rotation capacity.

CODE

B.10.3 — General principles and requirements

B.10.3.3 — For flexural members and members subject to combined flexure and compressive axial load when the design axial load strength ϕP_n is less than the smaller of $0.10 f_c' A_g$ or ϕP_b, the ratio of reinforcement ρ provided shall not exceed 0.75 of the ratio ρ_b that would produce balanced strain conditions for the section under flexure without axial load. For members with compression reinforcement, the portion of ρ_b equalized by compression reinforcement need not be reduced by the 0.75 factor.

COMMENTARY

RB.10.3 — General principles and requirements

RB.10.3.3 — The maximum amount of tension reinforcement in flexural members is limited to ensure a level of ductile behavior.

The nominal flexural strength of a section is reached when the strain in the extreme compression fiber reaches the limiting strain in the concrete. At ultimate strain of the concrete, the strain in the tension reinforcement could just reach the strain at first yield, be less than the yield strain (elastic), or exceed the yield strain (inelastic). The steel strain that exists at limiting concrete strain depends on the relative proportion of steel to concrete and material strengths f_c' and f_y.

If $\rho(f_y/f_c')$ is sufficiently low, the strain in the tension steel will greatly exceed the yield strain when the concrete strain reaches its limiting value, with large deflection and ample warning of impending failure (ductile failure condition). With a larger $\rho(f_y/f_c')$, the strain in the tension steel may not reach the yield strain when the concrete strain reaches its limiting value, with consequent small deflection and little warning of impending failure (brittle failure condition). For design, it is considered more conservative to restrict the nominal strength condition so that a ductile failure mode can be expected.

Unless unusual amounts of ductility are required, the $0.75\rho_b$ limitation will provide ductile behavior for most designs. One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames. Section B.8.4 permits negative moment redistribution. Because moment redistribution is dependent on adequate ductility in hinge regions, the amount of tension reinforcement in hinging regions is limited to $0.5\rho_b$.

For ductile behavior of beams with compression reinforcement, only that portion of the total tension steel balanced by compression in the concrete need be limited; that portion of the total tension steel where force is balanced by compression reinforcement need not be limited by the 0.75 factor.

CODE

COMMENTARY

Notes

APPENDIX C — ALTERNATIVE LOAD FACTORS, STRENGTH REDUCTION FACTORS, AND DISTRIBUTION OF FLEXURAL REINFORCEMENT

CODE

C.1 — General

C.1.1 — Structural concrete shall be permitted to be designed using the load factors, strength reduction factors, and distribution of flexural reinforcement of Appendix C. When Appendix C is used in design, Sections C.9.2 and C.9.3 shall replace Sections 9.2 and 9.3 in Chapter 9 of the code and Section C.10.6 shall replace Section 10.6 in Chapter 10 of the code. If any section in this appendix is used, all sections in this appendix shall be applied.

C.1.2 — It shall be permitted to use the provisions of Appendix B in conjunction with the provisions of Appendix C.

C.9.2 — Required strength

C.9.2.1 — Required strength U to resist dead load D and live load L shall be at least equal to

$$U = 1.4D + 1.7L \tag{C-1}$$

C.9.2.2 — If resistance to structural effects of a specified wind load W is included in design, the following combinations of D, L, and W shall be investigated to determine the greatest required strength U

$$U = 0.75\,(1.4D + 1.7L) + (1.6W) \tag{C-2}$$

where load combinations shall include both full value and zero value of L to determine the more severe condition, and

$$U = 0.9D + 1.6W \tag{C-3}$$

but for any combination of D, L, and W, required strength U shall not be less than Eq. (C-1). Where wind load W has not been reduced by a directionality factor, it shall be permitted to use $1.3W$ in place of $1.6W$ in Eq. (C-2) and (C-3).

C.9.2.3 — If resistance to specified earthquake loads or forces E are included in the design, the following combinations of D, L, H, F, and E shall be investigated to determine the greatest required strength U

$$U = 0.75(1.4D + 1.7L + 1.7H + 1.7F) + 1.0E \tag{C-4}$$

Except that where H or F reduce the effect of D, L, or each other, $0.8H$ or $1.3F$ shall be substituted for $1.7H$ or $1.7F$ as applicable. The greatest required strength shall be used to determine U.

And

$$U = 0.9D + 0.6H + 1.4F + 1.0E \tag{C-5a}$$

COMMENTARY

RC.1 — General

RC.1.1 — The load factors, strength reduction factors, and distribution of flexural reinforcement formerly in Chapter 9 were revised and moved to this appendix. Designs using the provisions of Appendix C satisfy the code, and are equally acceptable.

When this appendix is used, the corresponding commentary sections apply.

RC.9.2 — Required strength

RC.9.2.2 — The wind load equation in ASCE 7-98 and IBC 2000[C.1] includes a factor for wind directionality that is equal to 0.85 for buildings. The corresponding load factor for wind in the load combination equations was increased accordingly (1.3/0.85 = 1.53, rounded up to 1.6). The code allows use of the previous wind load factor of 1.3 when the design wind load is obtained from other sources that do not include the wind directionality factor.

RC.9.2.3 — The load E represents strength-level earthquake forces. Recent model building codes and design load references have converted earthquake forces to strength level, and reduced the earthquake load factor to 1.0. The code requires use of a load factor of 1.4 for earthquake loads when service-level earthquake forces from earlier editions of such codes are used.

The load combinations in Eq. (C-5a) and (C-5b) are included for the case where higher dead load reduces the effect of earthquake loads combined with static earth or fluid pressure.

Due to the significant uncertainty in determining soil pressures,

CODE

Or

$$U = 0.9D + 1.4H + 1.0F + 1.0E \qquad \text{(C-5b)}$$

But U shall not be less than Eq. (9-1).

The above load combinations shall include both full value and zero value of L and F to determine the most severe condition.

Estimations of earth pressures shall be permitted to be used to reduce other load effects only if investigation and analysis shows that structure movement and soil characteristics are appropriate to develop the pressure. Where earthquake load U is based on service-level seismic forces, $1.4E$ shall be used in place of $1.0E$ in Eq. (C-4), (C-5a), and (C-5b).

C.9.2.4 — If resistance to earth pressure H is included in design, required strength U shall be at least equal to

$$U = 1.4D + 1.7L + 1.7H \qquad \text{(C-6)}$$

except that where D or L reduce the effect of H, $0.9D$ shall be substituted for $1.4D$, and zero value of L shall be used to determine the greatest required strength U. For any combination of D, L, and H, required strength U shall not be less than Eq. (C-1).

C.9.2.5 — If resistance to loadings due to weight and pressure of fluids F is included in design, such loading shall have a load factor of 1.7, and be added to all governing loading combinations, except as shown in C.9.2.3, so that the effect of L or W does not reduce the effect of F.

C.9.2.6 — If resistance to impact effects is taken into account in design, such effects shall be included with live load L.

C.9.2.7 — Where structural effects T of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change are significant in design, required strength U shall be at least equal to

$$U = 0.75(1.4D + 1.4T + 1.7L) \qquad \text{(C-7)}$$

but required strength U shall not be less than

COMMENTARY

it is conservative to disregard earth pressures as a balancing force. It may be appropriate, however, for some load cases to consider earth pressures as a balancing force. When doing so, the magnitude of earth pressure used should be developed conservatively by a geotechnical engineer.

RC.9.2.4 — If effects H caused by earth pressure, ground-water pressure, or pressure caused by granular materials are included in design, the required strength equations become

$$U = 1.4D + 1.7L + 1.7H$$

and where D or L reduce the effect of H

$$U = 0.9D + 1.7H$$

but for any combination of D, L, or H

$$U = 1.4D + 1.7L$$

RC.9.2.5 — This section addresses the need to consider loading due to weight of liquid or liquid pressure.

For well-defined fluid pressures, the required strength equations become

$$U = 1.4D + 1.7L + 1.7F$$

and where D or L reduce the effect of F

$$U = 0.9D + 1.7F$$

but for any combination of D, L, or F

$$U = 1.4D + 1.7L$$

RC.9.2.6 — If the live load is applied rapidly, as may be the case for vehicle loads, cranes, etc., impact effects should be considered. In all equations, substitute (L + impact) for L when impact must be considered.

RC.9.2.7 — The designer should consider the effects of differential settlement, creep, shrinkage, temperature, and shrinkage-compensating concrete. The term "realistic assessment" is used to indicate that the most probable values, rather than the upper bound values, of the variables should be used.

Equation (C-8) is to prevent a design for load

$$U = 0.75(1.4D + 1.4T + 1.7L)$$

CODE

$$U = 1.4(D + T) \qquad \text{(C-8)}$$

Estimations of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change shall be based on realistic assessment of such effects occurring in service.

C.9.2.8 — For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum prestressing steel jacking force.

C.9.2.9 — Required strength U shall be multiplied by the following environmental durability factors (S_d) in portions of an environmental engineering concrete structure where durability, liquid-tightness, or similar serviceability are considerations. These durability factors shall not be used for prestressed reinforcement or for designs using service loads and permissible service load stresses per the alternate design method in Appendix I. For applicable use of the environmental durability factor (S_d) in conjunction with load combinations that include earthquake loads, see Section 21.2.1.8.

C.9.2.9.1 — Flexural stress: $S_d = 1.3$.

C.9.2.9.2 — Direct tensile stress (including hoop tension): $S_d = 1.65$.

C.9.2.9.3 — Excess shear stress carried by shear reinforcement: $S_d = 1.3$.

C.9.3 — Design strength

C.9.3.1 — Design strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this code, multiplied by the strength reduction factors ϕ in C.9.3.2 and C.9.3.4.

COMMENTARY

to approach

$$U = 1.05(D + T)$$

when live load is negligible.

RC.9.2.8 — The load factor of 1.2 applied to the maximum prestressing steel jacking force results in a design load of 113 percent of the specified prestressing steel yield strength, but not more than 96 percent of the nominal ultimate strength of the tendon. This compares well with a maximum attainable jacking force, which is limited by the anchor efficiency factor.

RC.9.3 — Design strength

RC.9.3.1 — The term "design strength" of a member refers to the nominal strength calculated in accordance with the requirements stipulated in this code multiplied by a strength reduction factor ϕ that is always less than one.

The purposes of the strength reduction factor ϕ are: (1) to allow for the probability of understrength members due to variations in material strengths and dimensions; (2) to allow for inaccuracies in the design equations; (3) to reflect the degree of ductility and required reliability of the member under the load effects being considered; and (4) to reflect the importance of the member in the structure.[C.2,C.3] For example, a lower ϕ is used for columns than for beams because columns generally have less ductility, are more sensitive to variations in concrete strength, and generally support larger loaded areas than beams. Furthermore, spiral columns are assigned a higher ϕ than tied columns because they have greater ductility or toughness.

CODE

C.9.3.2 — Strength reduction factor ϕ shall be as follows:

C.9.3.2.1 — Tension-controlled sections, as defined in 10.3.4 (see also C.9.3.2.6) 0.90

C.9.3.2.2 — Compression-controlled sections, as defined in 10.3.3:

(a) Members with spiral reinforcement conforming to 10.9.3 .... 0.75

(b) Other reinforced members 0.70

For sections in which the net tensile strain in the extreme tension steel at nominal strength is between the limits for compression-controlled and tension-controlled sections, ϕ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as the net tensile strain in the extreme tension steel at nominal strength increases from the compression-controlled strain limit to 0.005.

Alternatively, when Appendix B is used, for members in which f_y does not exceed 60,000 psi, with symmetric reinforcement, and with $(h - d' - d_s)/h$ not less than 0.70, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ to zero. For other reinforced members, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ or ϕP_b, whichever is smaller, to zero.

COMMENTARY

RC.9.3.2.1 — In applying C.9.3.2.1 and C.9.3.2.2, the axial tensions and compressions to be considered are those caused by external forces. Effects of prestressing forces are not included.

RC.9.3.2.2 — Previously, the code gave the magnitude of the ϕ-factor for cases of axial load or flexure, or both, in terms of the type of loading. For these cases, the ϕ-factor is now determined by the strain conditions at a cross section, at nominal strength.

A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections. Members with spiral reinforcement are assigned a higher ϕ than tied columns because they have greater ductility or toughness.

For sections subjected to axial load with flexure, design strengths are determined by multiplying both P_n and M_n by the appropriate single value of ϕ. Compression-controlled and tension-controlled sections are defined in 10.3.3 and 10.3.4 as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than 0.005, respectively. For sections with net tensile strain ε_t in the extreme tension steel at nominal strength between the above limits, the value of ϕ may be determined by linear interpolation, as shown in Fig. RC.9.3.2. The concept of net tensile strain ε_t is discussed in R10.3.3.

Because the compressive strain in the concrete at nominal strength is assumed in 10.2.3 to be 0.003, the net tensile strain limits for compression-controlled members may also be stated in terms of the ratio c/d_t, where c is the depth of

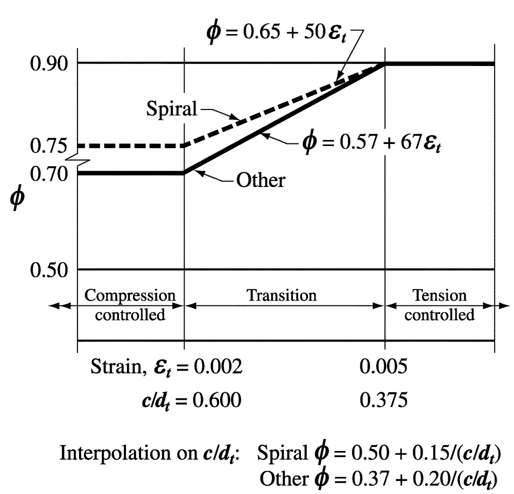


Fig. RC.9.3.2—Variation of ϕ with net tensile strain ε_t and c/d_t for Grade 60 reinforcement and for prestressing steel.

the neutral axis at nominal strength, and d_t is the distance from the extreme compression fiber to the extreme tension steel. The c/d_t limits for compression-controlled and tension-controlled sections are 0.6 and 0.375, respectively. The 0.6 limit applies to sections reinforced with Grade 60 steel and to prestressed sections. Figure RC.9.3.2 also gives equations for ϕ as a function of c/d_t.

The net tensile strain limit for tension-controlled sections may also be stated in terms of the ρ/ρ_b as defined in previous editions of the code. The net tensile strain limit of 0.005 corresponds to a ρ/ρ_b ratio of 0.63 for rectangular sections with Grade 60 reinforcement.

C.9.3.2.3 — Shear and torsion 0.85

C.9.3.2.4 — Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models) 0.70

C.9.3.2.5 — Post-tensioned anchorage zones ... 0.85

RC.9.3.2.5 — The ϕ-factor of 0.85 reflects the wide scatter of results of experimental anchorage zone studies. Because 18.13.4.2 limits the nominal compressive strength of unconfined concrete in the general zone to $0.7\lambda f'_{ci}$, the effective design strength for unconfined concrete is $0.85 \times 0.7\lambda f'_{ci} \approx 0.6\lambda f'_{ci}$.

C.9.3.2.6 — Flexure sections without axial load in pre-tensioned members where strand embedment is less than the development length as provided in 12.9.1.1...0.85

RC.9.3.2.6 — If a critical section occurs in a region where strand is not fully developed, failure may be by bond slip. Such a failure resembles a brittle shear failure; hence, the requirement for a reduced ϕ.

C.9.3.3 — Development lengths specified in Chapter 12 do not require a ϕ-factor.

C.9.3.4 — In structures that rely on special moment resisting frames or special reinforced concrete structural walls to resist earthquake effects, the strength reduction factors ϕ shall be modified as given in (a) through (c):

(a) The strength reduction factor for shear shall be 0.60 for any structural member that is designed to resist earthquake effects if its nominal shear strength is less than the shear corresponding to the development of the nominal flexural strength of the member. The nominal flexural strength shall be determined considering the most critical factored axial loads and including earthquake effects;

(b) The strength reduction factor for shear in diaphragms shall not exceed the minimum strength reduction factor for shear used for the vertical components of the primary lateral-force-resisting system;

(c) The strength reduction factor for shear in joints and diagonally reinforced coupling beams shall be 0.85.

RC.9.3.4 — Strength-reduction factors in C.9.3.4 are intended to compensate for uncertainties in estimation of strength of structural members in buildings. They are based primarily on experience with constant or steadily increasing applied load. For construction in regions of high seismic risk, some of the strength reduction factors have been modified in C.9.3.4 to account for the effects of displacements on strength into the nonlinear range of response on strength.

Section C.9.3.4(a) refers to brittle members, such as low-rise walls or portions of walls between openings, or diaphragms that are impractical to reinforce to raise their nominal shear strength above nominal flexural strength for the pertinent loading conditions.

Short structural walls were the primary vertical elements of the lateral-force-resisting system in many of the parking structures that sustained damage during the 1994 Northridge earthquake. Section C.9.3.4(b) requires the shear strength reduction factor for diaphragms to be 0.60 if the shear strength reduction factor for the walls is 0.60.

CODE

C.10.6 — Distribution of flexural reinforcement in beams and one-way slabs

C.10.6.1 — This section prescribes rules for distribution of flexural reinforcement to control flexural cracking in beams and in one-way slabs (slabs reinforced to resist flexural stresses in only one direction).

C.10.6.2 — Distribution of flexural reinforcement in two-way slabs shall be as required by 13.3.

C.10.6.3 — Flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross section as required by C.10.6.4.

C.10.6.4 — When design yield strength f_y for tension reinforcement exceeds 40,000 psi, cross sections of maximum positive and negative moment shall be so proportioned that the quantity z given by

$$z = f_s \sqrt[3]{d_c A} \qquad \text{(C-9)}$$

(for reinforcement located in one layer) does not exceed 115 kips per in. for normal environmental exposure and 95 kips per inch for severe environmental exposure. Normal environmental exposure is defined as liquid retention, exposure to liquids more alkaline than pH of 5, or exposure to sulfate solutions of less than 1000 ppm. Severe environmental exposures are conditions in which the limits defining normal environmental exposure are exceeded. Calculated

COMMENTARY

RC.10.6 — Distribution of flexural reinforcement in beams and one-way slabs

RC.10.6.1 — Many structures designed by working stress methods and with low steel stress served their intended functions with very limited flexural cracking. When high-strength reinforcing steels are used at high service load stresses, however, visible cracks must be expected, and steps must be taken in detailing of the reinforcement to control cracking. Environmental engineering concrete structures have traditionally performed well by using quality concrete as defined in this standard, using adequate compaction, limiting maximum bar stresses, and equally distributing more smaller bars rather than few larger bars on tension faces.

Control of cracking is particularly important when reinforcement with a yield strength in excess of 40,000 psi is used. Current good detailing practices will usually lead to adequate crack control even when reinforcement of 60,000 psi yield is used.

Extensive laboratory work[C.4-C.6] involving modern deformed bars has confirmed that crack width at service loads is proportional to steel stress. The significant variables reflecting steel detailing, however, were found to be thickness of concrete cover and the area of concrete in the zone of maximum tension surrounding each individual reinforcing bar.

Crack width is inherently subject to wide scatter, even in careful laboratory work, and is influenced by shrinkage and other time-dependent effects. The best crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

RC.10.6.3 — Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

RC.10.6.4 — Equation (C-9) will provide a distribution that will reasonably control flexural cracking. The equation is written in a form emphasizing reinforcing details rather than crack width. It is based on the Gergely-Lutz expression

$$w = 0.076\beta f_s \sqrt[3]{d_c A}$$

in which w is in units of 0.001 in. To simplify practical design, an approximate value of 1.2 is used for β (ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement). Laboratory tests[C.7] have shown that the Gergely-Lutz expression applies reasonably to one way slabs. The average ratio β_i about 1.35 for floor slabs, rather than the value 1.2 used for beams. Accordingly, it would be consistent to reduce the maximum values for z by the factor 1.2/1.35.

CODE

flexural stress in reinforcement at service load f_s (kips per in.^{2}) shall be computed as the moment divided by the product of steel area and internal moment arm. In place of such computations, it is permitted to take f_s as 45 percent of specified yield strength f_y. Where clear concrete cover exceeds 2 in., d_c is permitted to be based on 2 in. of clear concrete cover.

C.10.6.5 — Where flanges of T-beam construction are in tension, part of the flexural tension reinforcement shall be distributed over an effective flange width as defined in 8.10, or a width equal to 1/10 the span, whichever is smaller. If the effective flange width exceeds 1/10 the span, some longitudinal reinforcement shall be provided in the outer portions of the flange.

COMMENTARY

The numerical limitations of z = 115 and 95 kips per in. for normal environmental exposure and severe environmental exposure, respectively, correspond to limiting crack widths of 0.010 and 0.009 in. These z values were established for cover equal to or less than 2 in. and should be based on this value even when cover exceeds 2 in. Additional cover may be regarded as added protection.

The effective tension area of concrete surrounding the principal reinforcement is defined as having the same centroid as the reinforcement. Moreover, this area is to be bounded by the surfaces of the cross section and a straight line parallel to the neutral axis. Computation of effective area per bar $\boldsymbol{A}$ is as illustrated in Fig. RC.10.6.4.

For normal environmental exposure, deformed bars or wire should be spaced so that z does not exceed 115 kips per in. The spacing of the bars should not exceed 12 in. Bar size preferably should not exceed No. 11. For severe environmental exposure, deformed bars should be spaced so that z does not exceed 95 kips per in., and surface or other protection or barrier should be provided, suitable for the particular conditions of exposure. Provisions of C.10.6.4 in this Code are intended to provide liquid-tight environmental engineering concrete structures within the scope of this Code.

RC.10.6.5 — In major T-beams, distribution of the negative reinforcement for control of cracking must take into account two considerations: (1) wide spacing of the reinforcement across the full effective width of flange may cause some wide cracks to form in the slab near the web; and (2) close spacing near the web leaves the outer regions of the flange unprotected. The 1/10 limitation is to guard against too wide a spacing, with some additional reinforcement required to protect the outer portions of the flange.

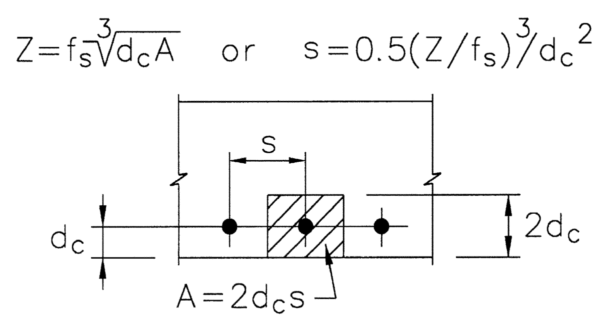


Fig. RC.10.6.4—Effective tension area of concrete.

CODE

C.10.6.6 — If the effective depth ***d*** of a beam or joist exceeds 36 in., longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance ***d*/2** nearest the flexural tension reinforcement. The area of skin reinforcement A_{sk} per foot of height on each side face shall be ≥**0.012(*d* – 30)**.The maximum spacing of the skin reinforcement shall not exceed the lesser of ***d*/6** and 12 in. It shall be permitted to include such reinforcement in strength computations if a strain compatibility analysis is made to determine stress in the individual bars or wires. The total area of longitudinal skin reinforcement in both faces need not exceed one-half of the required flexural tensile reinforcement.

COMMENTARY

RC.10.6.6 — For relatively deep flexural members, some reinforcement should be placed near the vertical faces in the tension zone to control cracking in the web. Without such auxiliary steel, the width of the cracks in the web may greatly exceed the crack widths at the level of the flexural tension reinforcement.

The requirements for skin reinforcement were modified in the ACI 318-89 code, as the previous requirements were found to be inadequate in some cases. For lightly reinforced members, these requirements may be reduced to one-half of the main flexural reinforcement. Where the provisions for deep beams, walls, or precast panels require more steel, those provisions (along with their spacing requirements) will govern.

APPENDIX D — ANCHORING TO CONCRETE

CODE

D.0 — Notation

A_{brg} = bearing area of the head of stud or anchor bolt, in.^{2}

A_{No} = projected concrete failure area of one anchor, for calculation of strength in tension when not limited by edge distance or spacing, in.^{2} (see D 5.2.1)

A_N = projected concrete failure area of an anchor or group of anchors, for calculation of strength in tension, as defined in, in.^{2} (see D.5.2.1) A_N shall not be taken greater than nA_{No}

A_{se} = effective cross-sectional area of anchor, in.^{2}

A_{sl} = effective cross-sectional area of expansion or undercut anchor sleeve, if sleeve is within shear plane, in.^{2}

A_{Vo} = projected concrete failure area of one anchor, for calculation of strength in shear, when not limited by corner influences, spacing, or member thickness, in.^{2} (see D.6.2.1)

A_V = projected concrete failure area of an anchor or group of anchors, for calculation of strength in shear, in.^{2} (see D.6.2.1) A_V shall not be taken greater than nA_{Vo}

c = distance from center of an anchor shaft to the edge of concrete, in.

c_1 = distance from the center of an anchor shaft to the edge of concrete in one direction, in.; where shear force is applied to anchor, c_1 is in the direction of the shear force.

c_2 = distance from center of an anchor shaft to the edge of concrete in the direction orthogonal to c_1, in.

c_{max} = the largest edge distance, in.

c_{min} = the smallest edge distance, in.

d_o = outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, in. (see also D.8.4)

COMMENTARY

RD.0 — Notation

A_{No}: See Fig. RD.5.2.1(a).

A_N: See Fig. RD.5.2.1(b).

A_{se} = The effective cross-sectional area of an anchor should be provided by the manufacturer of expansion anchors with reduced cross-sectional area for the expansion mechanism. For threaded bolts, ANSI/ASME B1.1[D.1] defines A_{se} as

$$A_{se} - \frac{\pi}{4}\left(d_o - \frac{0.9743}{n_t}\right)^2$$

where n_t is the number of threads per inch.

A_{Vo}: See Fig. RD.6.2.1(a).

A_V: See Fig. RD.6.2.1(b).

c_1: See Fig. RD.6.2.1(a).

CODE

d'_o = value substituted for d_o when an oversized anchor is used, in. (see D.8.4)

e_h = distance from the inner surface of the shaft of a J- or L-bolt to the outer tip of the J- or L-bolt, in.

e'_N = eccentricity of normal force on a group of anchors; the distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension, in.; e'_N is always positive

e'_V = eccentricity of shear force on a group of anchors; the distance between the point of shear force application and the centroid of the group of anchors resisting shear in the direction of the applied shear, in.

f'_c = specified compressive strength of concrete, psi

f_{ct} = specified tensile strength of concrete, psi

f_r = modulus of rupture of concrete, psi (see 9.5.2.3)

f_t = calculated concrete tensile stress in a region of a member, psi

f_y = specified yield strength of anchor steel, psi

f_{ut} = specified tensile strength of anchor steel, psi

f_{utsl} = specified tensile strength of anchor sleeve, psi

h = thickness of member in which an anchor is anchored, measured parallel to anchor axis, in.

h_{ef} = effective anchor embedment depth, in. (see D.8.5)

k = coefficient for basic concrete breakout strength in tension

k_{cp} = coefficient for pryout strength

ℓ = load bearing length of anchor for shear, not to exceed $8d_o$, in.

= h_{ef} for anchors with a constant stiffness over the full length of the embedded section, such as headed studs or post-installed anchors with one tubular shell over the full length of the embedment depth

= $2d_o$ for torque-controlled expansion anchors with a distance sleeve separated from the expansion sleeve

n = number of anchors in a group

N_b = basic concrete breakout strength in tension of a single anchor in cracked concrete, lb (see D 5.2.2)

N_{cb} = nominal concrete breakout strength in tension of a single anchor, lb (see D.5.2.1)

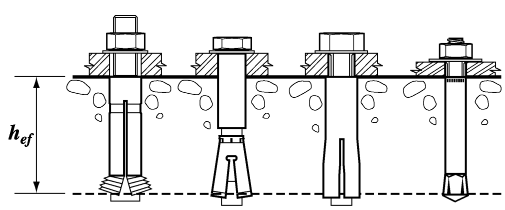
N_{cbg} = nominal concrete breakout strength in tension of a group of anchors, lb (see D.5.2.1)

N_n = nominal strength in tension, lb

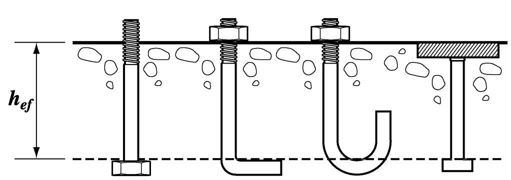
COMMENTARY

e_N = Actual eccentricity of a normal force on an attachment (See Fig. RD.5.2.4.)

h_{ef} = Effective embedment depths for a variety of anchor types are shown in Fig. RD.0



(a) Post-installed anchors



(b) Cast-in-place anchors

Fig. RD.0—Types of anchors.

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COMMENTARY

N_p = pullout strength in tension of a single anchor in cracked concrete, lb (see D.5.3.4 or D.5.3.5)

N_{pn} = nominal pullout strength in tension of a single anchor, lb (see D.5.3.1)

N_{sb} = side-face blowout strength of a single anchor, lb

N_{sbg} = side-face blowout strength of a group of anchors, lb

N_s = nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, lb (see D.5.1.1 or D.5.1.2)

N_u = factored tensile load, lb

s = anchor center-to-center spacing, in.

s_o = spacing of the outer anchors along the edge in a group, in.

t = thickness of washer or plate, in.

V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete, lb (see D.6.2.2 or D.6.2.3)

V_{cb} = nominal concrete breakout strength in shear of a single anchor, lb (see D.6.2.1)

V_{cbg} = nominal concrete breakout strength in shear of a group of anchors, lb (see D.6.2.1)

V_{cp} = nominal concrete pryout strength, lb (see D.6.3)

V_n = nominal shear strength, lb

V_s = nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, lb (see D.6.1.1 or D.6.1.2)

V_u = factored shear load, lb

ϕ = strength reduction factor (see D.4.4 and D.4.5)

ψ_1 = modification factor, for strength in tension, to account for anchor groups loaded eccentrically (see D.5.2.4)

ψ_2 = modification factor, for strength in tension, to account for edge distances smaller than $1.5h_{ef}$ (see D.5.2.5)

ψ_3 = modification factor, for strength in tension, to account for cracking (see D.5.2.6 and D.5.2.7)

ψ_4 = modification factor, for pullout strength, to account for cracking (see D.5.3.1 and D.5.3.6)

ψ_5 = modification factor, for strength in shear, to account for anchor groups loaded eccentrically (see D.6.2.5)

ψ_6 = modification factor, for strength in shear, to account for edge distances smaller than $1.5c_1$ (see D.6.2.6)

ψ_7 = modification factor, for strength in shear, to account for cracking (see D.6.2.7)

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D.1 — Definitions

Anchor — A steel element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads, including headed bolts, hooked bolts (J- or L-bolt), headed studs, expansion anchors, or undercut anchors.

Anchor group — A number of anchors of approximately equal effective embedment depth with each anchor spaced at less than three times its embedment depth from one or more adjacent anchors.

Anchor pullout strength — The strength corresponding to the anchoring device or a major component of the device sliding out from the concrete without breaking out a substantial portion of the surrounding concrete.

Attachment — The structural assembly, external to the surface of the concrete, that transmits loads to or receives loads from the anchor.

Brittle steel element — An element with a tensile test elongation of less than 14 percent, or reduction in area of less than 30 percent, or both.

Cast-in anchor — A headed bolt, headed stud, or hooked bolt installed before placing concrete.

Concrete breakout strength — The strength corresponding to a volume of concrete surrounding the anchor or group of anchors separating from the member.

Concrete pryout strength — The strength corresponding to formation of a concrete spall behind a short, stiff anchor with an embedded base that is displaced in the direction opposite to the applied shear force.

Distance sleeve — A sleeve that encases the center part of an undercut anchor, a torque-controlled expansion anchor, or a displacement-controlled expansion anchor, but does not expand.

Ductile steel element — An element with a tensile test elongation of at least 14 percent and reduction in area of at least 30 percent. A steel element meeting the requirements of ASTM A 307 shall be considered ductile.

Edge distance — The distance from the edge of the concrete surface to the center of the nearest anchor.

Effective embedment depth — The overall depth through which the anchor transfers force to or from the surrounding concrete. The effective embedment depth will normally be the depth of the concrete failure surface in tension applications. For cast-in headed anchor bolts and headed studs, the effective embedment depth is measured from the bearing contact surface of the head. (See Fig. RD.0.)

Expansion anchor — A post-installed anchor,

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RD.1 — Definitions

Brittle steel element and ductile steel element — The 14 percent elongation should be measured over the gage length specified in the appropriate ASTM standard for the steel.

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inserted into hardened concrete that transfers loads to or from the concrete by direct bearing or friction or both. Expansion anchors may be torque-controlled, where the expansion is achieved by a torque acting on the screw or bolt; or displacement-controlled, where the expansion is achieved by impact forces acting on a sleeve or plug and the expansion is controlled by the length of travel of the sleeve or plug.

Expansion sleeve — The outer part of an expansion anchor that is forced outward by the center part, either by applied torque or impact, to bear against the sides of the predrilled hole.

Five percent fractile — A statistical term meaning 90 percent confidence that there is 95 percent probability of the actual strength exceeding the nominal strength.

Hooked bolt — A cast-in anchor anchored mainly by mechanical interlock from the 90-deg bend (L-bolt) or 180-deg bend (J-bolt) at its lower end, having a minimum e_h of $3d_o$.

Headed stud — A steel anchor conforming to the requirements of AWS D1.1 and affixed to a plate or similar steel attachment by the stud arc welding process before casting.

Post-installed anchor — An anchor installed in hardened concrete. Expansion anchors and undercut anchors are examples of post-installed anchors.

Projected area — The area on the free surface of the concrete member that is used to represent the larger base of the assumed rectilinear failure surface.

Side-face blowout strength — The strength of anchors with deeper embedment but thinner side cover corresponding to concrete spalling on the side face around the embedded head while no major breakout occurs at the top concrete surface.

Specialty insert — Predesigned and prefabricated cast-in anchors specifically designed for attachment of bolted or slotted connections. Specialty inserts are often used for handling, transportation, and erection, but are also used for anchoring structural elements. Specialty inserts are not within the scope of this appendix.

Supplementary reinforcement — Reinforcement proportioned to tie a potential concrete failure prism to the structural member.

Undercut anchor — A post-installed anchor that develops its tensile strength from the mechanical interlock provided by undercutting of the concrete at the embedded end of the anchor. The undercutting is

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Five percent fractile — The determination of the coefficient k associated with the 5 percent fractile, $\bar{x} - K\sigma$, depends on the number of tests n used to compute $\bar{x}$ and σ. Values of k range, for example, from 1.645 for $n = \infty$, to 2.010 for n = 40, and 2.568 for n = 10. With this definition of the 5 percent fractile, the nominal strength in D.4.2 is the same as the characteristic strength in ACI 355.2.

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achieved with a special drill before installing the anchor or alternatively by the anchor itself during its installation.

D.2 — Scope

D.2.1 — This appendix provides design requirements for anchors in concrete used to transmit structural loads by means of tension, shear, or a combination of tension and shear between: (a) connected structural elements; or (b) safety-related attachments and structural elements. Safety levels specified are intended for in-service conditions, rather than for short-term handling and construction conditions.

D.2.2 — This appendix applies to both cast-in anchors and post-installed anchors. Specialty inserts, through bolts, multiple anchors connected to a single steel plate at the embedded end of the anchors, adhesive or grouted anchors, and direct anchors such as powder or pneumatic actuated nails or bolts, are not included. Reinforcement used as part of the embedment shall be designed in accordance with other parts of the code.

D.2.3 — Headed studs and headed bolts having a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal or exceeding $1.4N_p$ (where N_p is given by Eq. (D-13)) are included. Hooked bolts that have a geometry that has been demonstrated to result in a pullout strength without the benefit of friction in uncracked concrete equal or exceeding $1.4N_p$ (where N_p is given by Eq. (D-14)) are included. Post-installed anchors that meet the assessment requirements of ACI 355.2 are included. The suitability of the post-installed anchor for use in concrete shall have been demonstrated by the ACI 355.2 prequalification tests.

D.2.4 — Load applications that are predominantly high cycle fatigue or impact loads are not covered by this appendix.

D.3 — General requirements

D.3.1 — Anchors and anchor groups shall be designed for critical effects of factored loads as determined by elastic analysis. Plastic analysis approaches are permitted where nominal strength is controlled by

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RD.2 — Scope

RD.2.1 — Appendix D is restricted in scope to structural anchors that transmit structural loads related to strength, stability, or life safety. Two types of applications are envisioned. The first is connections between structural elements where the failure of an anchor or an anchor group could result in loss of equilibrium or stability of any portion of the structure. The second is where safety-related attachments that are not part of the structure (such as sprinkler systems, heavy suspended pipes, or barrier rails) are attached to structural elements. The levels of safety defined by the combinations of load factors and ϕ-factors are appropriate for structural applications. Other standards may require more stringent safety levels during temporary handling.

RD.2.2 — The wide variety of shapes and configurations of specialty inserts makes it difficult to prescribe generalized tests and design equations for many insert types. Hence, they have been excluded from the scope of Appendix D. Adhesive anchors are widely used, and can perform adequately. At this time, however, such anchors are outside the scope of this appendix. The mechanical post-installed anchors included in this appendix are not recommended in locations to be submerged. To prevent penetration of liquid into the concrete, only cast-in-place or adhesive anchors are recommended.

RD.2.3 — Typical cast-in headed studs and headed bolts with geometries consistent with ANSI/ASME B1.1,[D.1] B18.2.1,[D.2] and B18.2.6[D.3] have been tested and proven to behave predictably, so calculated pullout values are acceptable. Post-installed anchors do not have predictable pullout capacities, and therefore are required to be tested. For a post-installed anchor to be used in conjunction with the requirements of this appendix, the results of the ACI 355.2 tests have to indicate that pullout failures exhibit an acceptable load-displacement characteristic or that pullout failures are precluded by another failure mode.

RD.2.4 — The exclusion from the scope of load applications producing high cycle fatigue or extremely short duration impact (such as blast or shock wave) are not meant to exclude seismic load effects. D.3.3 presents additional requirements for design when seismic loads are included.

RD.3 — General requirements

RD.3.1 — When the strength of an anchor group is governed by breakage of the concrete, the behavior is brittle, and there is limited redistribution of the forces between the highly stressed and less stressed anchors. In this case, the

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ductile steel elements, provided that deformational compatibility is taken into account.

D.3.2 — The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in 9.2.

D.3.3 — When anchor design includes seismic loads, the additional requirements of D.3.3.1 through D.3.3.5 shall apply.

D.3.3.1 — The provisions of Appendix D do not apply to the design of anchors in plastic hinge zones of concrete structures under seismic loads.

D.3.3.2 — In regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories, post-installed structural anchors for use under D.2.3 shall have passed the Simulated Seismic Tests of ACI 355.2.

D.3.3.3 — In regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories, the design strength of anchors shall be taken as $0.75\phi N_n$ and

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theory of elasticity is required to be used assuming the attachment that distributes loads to the anchors is sufficiently stiff. The forces in the anchors are considered to be proportional to the external load and its distance from the neutral axis of the anchor group.

If anchor strength is governed by ductile yielding of the anchor steel, significant redistribution of anchor forces can occur. In this case, an analysis based on the theory of elasticity will be conservative. References D.4 to D.6 discuss non-linear analysis, using theory of plasticity, for the determination of the capacities of ductile anchor groups.

RD.3.3 — Post-installed structural anchors are required to be qualified for moderate or high seismic risk zone usage by demonstrating the capacity to undergo large displacements through several cycles as specified in the seismic simulation tests of ACI 355.2. Because ACI 355.2 excludes plastic hinge zones, Appendix D is not applicable to the design of anchors in plastic hinge zones under seismic loads. In addition, the design of anchors in zones of moderate or high seismic risk is based on a more conservative approach by the introduction of 0.75 factor on the design strength ϕN_n and ϕV_n, and by requiring the system to have adequate ductility. Anchorage capacity should be governed by ductile yielding of a steel element. If the anchor cannot meet these ductility requirements, then the attachment is required to be designed so as to yield at a load well below the anchor capacity. In designing attachments for adequate ductility, the ratio of yield to ultimate load capacity should be considered. A connection element could yield only to result in a secondary failure as one or more elements strain harden and fail if the ultimate load capacity is excessive when compared to the yield capacity.

Under seismic conditions, the direction of shear loading may not be predictable. The full shear load should be assumed in any direction for a safe design.

RD.3.3.1 — Section 3.1 of ACI 355.2 specifically states that the seismic test procedures do no simulate the behavior of anchors in plastic hinge zones. The possible higher level of cracking and spalling in plastic hinge zones are beyond the damage states for which Appendix D is applicable.

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$0.75\phi V_n$, where ϕ is given in D.4.4 or D.4.5 and N_n and V_n are determined in accordance with D.4.1.

D.3.3.4 — In regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories, anchors shall be designed to be governed by tensile or shear strength of a ductile steel element, unless D.3.3.5 is satisfied.

D.3.3.5 — Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

D.3.4 — All provisions for anchor axial tension and shear strength apply to normalweight concrete. When lightweight aggregate concrete is used, provisions for N_n and V_n shall be modified by multiplying all values of $\sqrt{f_c'}$ affecting N_n and V_n, by 0.75 for all-lightweight concrete and 0.85 for sand-lightweight concrete. Linear interpolation shall be permitted when partial sand replacement is used.

D.3.5 — The values of f_c' used for calculation purposes in this appendix shall not exceed 10,000 psi for cast-in anchors, and 8000 psi for post-installed anchors. Testing is required for post-installed anchors when used in concrete f_c' with greater than 8000 psi.

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RD.3.5 — A limited number of tests of cast-in-place and post-installed anchors in high-strength concrete[D.7] indicate that the design procedures contained in this appendix become unconservative, particularly for cast-in anchors, at f_c' = 11,000 to 12,000 psi. Until further tests are available, an upper limit of f_c' = 10,000 psi has been imposed in the design of cast-in-place anchors. This is consistent with Chapters 11 and 12. The companion ACI 355.2 does not require testing of post-installed anchors in concrete with f_c' greater than 8000 psi because some post-installed anchors may have difficulty expanding in very high-strength concretes. Because of this, f_c' is limited to 8000 psi in the design of post-installed anchors unless testing is performed.

D.4 — General requirements for strength of anchors

D.4.1 — Strength design of anchors shall be based either on computation using design models that satisfy the requirements of D.4.2, or on test evaluation using the 5 percent fractile of test results for the following:

(a) Steel strength of anchor in tension (D.5.1);

(b) Steel strength of anchor in shear (D.6.1);

(c) Concrete breakout strength of anchor in tension (D.5.2);

(d) Concrete breakout strength of anchor in shear (D.6.2);

(e) Pullout strength of anchor in tension (D.5.3);

RD.4 — General requirements for strength of anchors

RD.4.1 — This section provides requirements for establishing the strength of anchors to concrete. The various types of steel and concrete failure modes for anchors are shown in Fig. RD.4.1(a) and RD.4.1(b). Comprehensive discussions of anchor failure modes are included in References D.8 to D.10. Any model that complies with the requirements of D.4.2 and D.4.3 can be used to establish the concrete-related strengths. For anchors such as headed bolts, headed studs, and post-installed anchors, the concrete breakout design methods of D.5.2 and D.6.2 are acceptable. The anchor strength is also dependent on the pullout strength of D.5.3, the side-face blowout strength of D.5.4, and the minimum spacings and edge distances of D.8. The design of anchors for tension recognizes that the strength of

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(f) Concrete side-face blowout strength of anchor in tension (D.5.4); and

(g) Concrete pryout strength of anchor in shear (D.6.3).

In addition, anchors shall satisfy the required edge distances, spacings, and thicknesses to preclude splitting failure, as required in D.8.

D.4.1.1 — For the design of anchors, except as required in D.3.3

$$\phi N_n \geq N_u \tag{D-1}$$

$$\phi V_n \geq V_u \tag{D-2}$$

D.4.1.2 — In Eq. (D-1) and (D-2), ϕN_n and ϕV_n are the lowest design strengths determined from all appropriate failure modes. ϕN_n is the lowest design strength in tension of an anchor or group of anchors as determined from consideration of ϕN_s, $\phi n N_{pn}$, either ϕN_{sb} or ϕN_{sbg}, and either ϕN_{cb} or ϕN_{cbg}. ϕV_n is the lowest design strength in shear of an anchor or a group of anchors as determined from consideration of ϕV_s, either ϕV_{cb} or ϕV_{cbg}, and ϕV_{cp}.

D.4.1.3 — When both N_u and V_u are present, interaction effects shall be considered in accordance with D.4.3.

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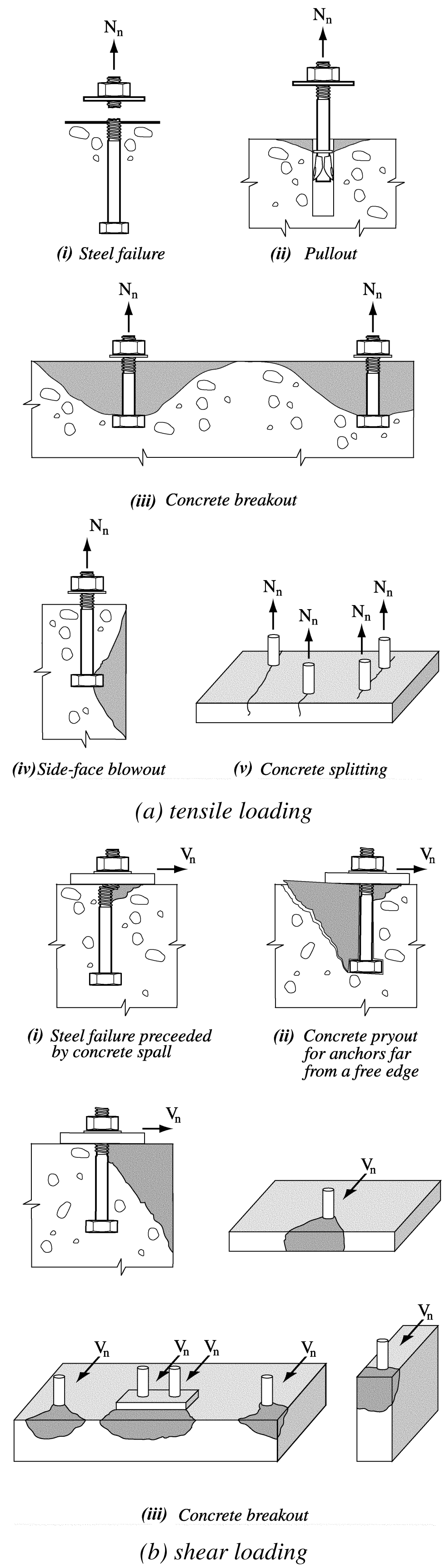

Fig. RD4.1—Failure modes for anchors.

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anchors is sensitive to appropriate installation; installation requirements are included in D.9. Some post-installed anchors are less sensitive to installation errors and tolerances. This is reflected in varied ϕ-factors based on the assessment criteria of ACI 355.2.

Test procedures can also be used to determine the single-anchor breakout strength in tension and in shear. The test results, however, are required to be evaluated on a basis statistically equivalent to that used to select the values for the concrete breakout method "considered to satisfy" provisions of D.4.2. The basic strength cannot be taken greater than the 5 percent fractile. The number of tests has to be sufficient for statistical validity and should be considered in the determination of the 5 percent fractile.

D.4.2 — The nominal strength for any anchor or group of anchors shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests. The materials used in the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5 percent fractile of the basic individual anchor strength. For nominal strengths related to concrete strength, modifications for size effects, the number of anchors, the effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loadings of anchor groups, and presence or absence of cracking shall be taken into account. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

RD.4.2 and RD.4.3 — D.4.2 and D.4.3 establish the performance factors for which anchor design models are required to be verified. Many possible design approaches exist, and the user is always permitted to "design by test" using D.4.2 as long as sufficient data are available to verify the model.

D.4.2.1 — The effect of supplementary reinforcement provided to confine or restrain the concrete breakout, or both, shall be permitted to be included in the design models used to satisfy D.4.2.

RD.4.2.1 — The addition of supplementary reinforcement in the direction of the load, confining reinforcement, or both, can greatly enhance the strength and ductility of the anchor connection. Such enhancement is practical with cast-in anchors such as those used in precast sections.

The shear strength of headed anchors located near the edge of a member can be significantly increased with appropriate supplementary reinforcement. References D.8, D.11, and D.12 provide substantial information on design of such reinforcement. The effect of such supplementary reinforcement is not included in the ACI 355.2 anchor acceptance tests or in the concrete breakout calculation method of D.5.2 and D.6.2. The designer has to rely on other test data and design theories in order to include the effects of supplementary reinforcement.

For anchors exceeding the limitations of D.4.2.2, or for situations where geometric restrictions limit breakout capacity, or both, reinforcement oriented in the direction of load and proportioned to resist the total load within the breakout prism, and fully anchored on both sides of the breakout planes, may be provided instead of calculating breakout capacity.

The breakout strength of an unreinforced connection can be

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D.4.2.2 — For anchors with diameters not exceeding 2 in., and tensile embedments not exceeding 25 in. in depth, the concrete breakout strength requirements shall be considered satisfied by the design procedure of D.5.2 and D.6.2.

D.4.3 — Resistance to combined tensile and shear loads shall be considered in design using an interaction expression that results in computation of strength in substantial agreement with results of comprehensive tests. This requirement shall be considered satisfied by D.7.

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taken as an indication of the load at which significant cracking will occur. Such cracking can represent a serviceability problem if not controlled. (See RD.6.2.1.)

RD.4.2.2 — The method for concrete breakout design included as "considered to satisfy" D.4.2 was developed from the concrete capacity design (CCD) method,[D.9,D.10] which was an adaptation of the κ method,[D.13,D.14] and is considered to be accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD method predicts the load capacity of an anchor or group of anchors by using a basic equation for tension, or for shear for a single anchor in cracked concrete, and multiplied by factors that account for the number of anchors, edge distance, spacing, eccentricity, and absence of cracking. The limitations on anchor size and embedment length are based on the current range of test data.

The breakout strength calculations are based on a model suggested in the κ method. It is consistent with a breakout prism angle of approximately 35 degrees [Fig. RD.4.2.2(a) and (b)].

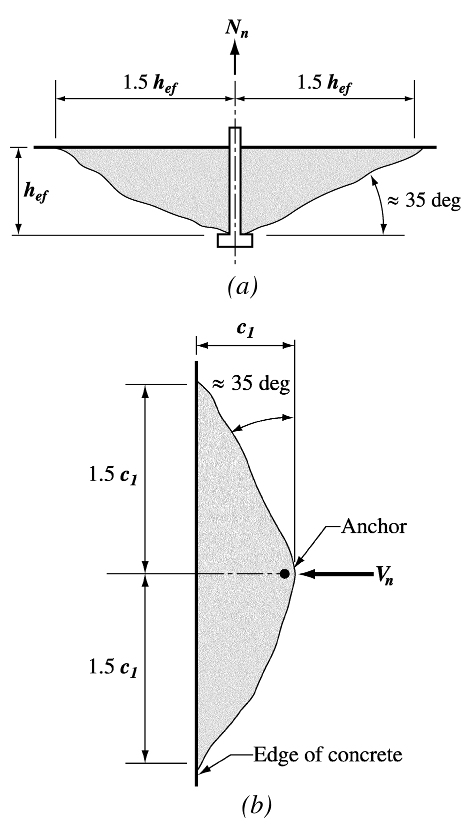

Fig. RD.4.2.2—(a) Breakout cone for tension; and (b) breakout cone for shear.

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D.4.4 — Strength reduction factor ϕ for anchors in concrete shall be as follows when the load combinations of 9.2 are used:

(a) Anchor governed by strength of a ductile steel element

i) Tension loads 0.75

ii) Shear loads 0.65

(b) Anchor governed by strength of a brittle steel element

i) Tension loads 0.65

ii) Shear loads 0.60

(c) Anchor governed by concrete breakout, side-face blowout, pullout, or pryout strength

	Condition A	Condition B
i) Shear loads	0.75	0.70
ii) Tension loads		
Cast-in headed studs, headed bolts, or hooked bolts	0.75	0.70
Post-installed anchors with category as determined from ACI 355.2		
Category 1 (Low sensitivity to installation and high reliability)	0.75	0.65
Category 2 (Medium sensitivity to installation and medium reliability)	0.65	0.55
Category 3 (High sensitivity to installation and lower reliability)	0.55	0.45

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.

D.4.5 — Strength reduction factor ϕ for anchors in concrete shall be as follows when the load combinations referenced in Appendix C are used:

(a) Anchor governed by strength of a ductile steel element

i) Tension loads 0.80

ii) Shear loads 0.75

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RD.4.4 — The ϕ factors for steel strength are based on using f_{ut} to determine the nominal strength of the anchor (see D.5.1 and D.6.1) rather than f_y as used in the design of reinforced concrete members. Although the ϕ factors for use with f_{ut} appear low, they result in a level of safety consistent with the use of higher ϕ factors applied to f_y. The smaller ϕ factors for shear than for tension do not reflect basic material differences, but rather account for the possibility of a non-uniform distribution of shear in connections with multiple anchors. It is acceptable to have a ductile failure of a steel element in the attachment if the attachment is designed so that it will undergo ductile yielding at a load level no greater than 75 percent of the minimum design strength of an anchor (see D.3.3.4). For anchors governed by the more brittle concrete breakout or blowout failure, two conditions are recognized. If supplementary reinforcement is provided to tie the failure prism into the structural member (Condition A), more ductility is present than in the case where such supplementary reinforcement is not present (Condition B). Design of supplementary reinforcement is discussed in RD.4.2.1 and References D.8, D.11, D.12, and D.15. Further discussion of strength reduction factors is presented in RD.4.5.

The ACI 355.2 tests for sensitivity to installation procedures determine the category appropriate for a particular anchoring device. In the ACI 355.2 tests, the effects of variability in anchor torque during installation, tolerance on drilled hole size, energy level used in setting anchors, and for anchors approved for use in cracked concrete, increased crack widths are considered. The three categories of acceptable post-installed anchors are:

Category 1 — low sensitivity to installation and high reliability;

Category 2 — medium sensitivity to installation and medium reliability; and

Category 3 — high sensitivity to installation and lower reliability.

The capacities of anchors under shear loads are not as sensitive to installation errors and tolerances. Therefore, for shear calculations of all anchors, ϕ = 0.75 for Condition A and ϕ = 0.70 for Condition B.

RD.4.5 — As noted in R9.1, the 2002 code incorporated the load factors of ASCE 7-98 and the corresponding strength reduction factors provided in the 1999 Appendix C into 9.2 and 9.3, except that the factor for flexure has been increased. Developmental studies for the ϕ-factors to be used for Appendix D were based on the 1999 9.2 and 9.3 load and strength reduction factors. The resulting ϕ-factors are presented in D.4.5 for use with the load factors of the 2002 Appendix C. The ϕ-factors for use with the load

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(b) Anchor governed by strength of a brittle steel element

i) Tension loads 0.70

ii) Shear loads 0.65

(c) Anchor governed by concrete breakout, side-face blowout, pullout, or pryout strength

	Condition A	Condition B
i) Shear loads	0.85	0.75
ii) Tension loads		
Cast-in headed studs, headed bolts, or hooked bolts	0.85	0.75
Post-installed anchors with category as determined from ACI 355.2		
Category 1 (Low sensitivity to installation and high reliability)	0.85	0.75
Category 2 (Medium sensitivity to installation and medium reliability)	0.75	0.65
Category 3 (High sensitivity to installation and lower reliability)	0.65	0.55

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.

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factors of the 1999 Appendix C were determined in a manner consistent with the other ϕ-factors of the 1999 Appendix C. These ϕ-factors are presented in D.4.4 for use with the load factors of 2002 9.2. Because developmental studies for ϕ-factors to be used with Appendix D, for brittle concrete failure modes, were performed for the load and strength reduction factors now given in Appendix C, the discussion of the selection of these ϕ-factors appears in this section.

Even though the ϕ-factor for plain concrete in Appendix C uses a value of 0.65, the basic factor for brittle concrete failures (ϕ = 0.75) was chosen based on results of probabilistic studies[D.16] that indicated the use of ϕ = 0.65 with mean values of concrete-controlled failures produced adequate safety levels. Because the nominal resistance expressions used in this appendix and in the test requirements are based on the 5 percent fractiles, the ϕ = 0.65 value would be overly conservative. Comparison with other design procedures and probabilistic studies[D.16] indicated that the choice of ϕ = 0.75 was justified. For applications with supplementary reinforcement and more ductile failures (Condition A), the ϕ factors are increased. The value of ϕ = 0.85 is compatible with the level of safety for shear failures in concrete beams, and has been recommended in the PCI Design Handbook[D.17] and by ACI 349.[D.15]

D.5 — Design requirements for tensile loading

D.5.1 — Steel strength of anchor in tension

D.5.1.1 — The nominal strength N_s of an anchor in tension as governed by the steel shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

D.5.1.2 — The nominal strength N_s of an anchor or group of anchors in tension shall not exceed

$$N_s = nA_{se}f_{ut} \tag{D-3}$$

where f_{ut} shall not be taken greater than **1.9**f_y or 125,000 psi.

RD.5 — Design requirements for tensile loading

RD.5.1 — Steel strength of anchor in tension

RD.5.1.2 — The nominal tension strength of anchors is best represented by $A_{se}f_{ut}$ rather than $A_{se}f_y$ because the large majority of anchor materials do not exhibit a well-defined yield point. The American Institute of Steel Construction (AISC) has based tension strength of anchors on $A_{se}\,f_{ut}$ since the 1986 edition of their specifications. The use of Eq. (D-3) with 9.2 load factors and the ϕ-factors of D.4.4 give design strengths consistent with the AISC Load and Resistance Factor Design Specifications.[D.18]

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COMMENTARY

The limitation of $\mathbf{1.9}f_y$ on f_{ut} is to ensure that under service load conditions the anchor does not exceed f_y. The limit on f_{ut} of $\mathbf{1.9}f_y$ was determined by converting the LRFD provisions to corresponding service level conditions. For Section 9.2, the average load factor of 1.4 (from $\mathbf{1.2}D$ + $\mathbf{1.6}L$) divided by the highest ϕ-factor (0.75 for tension) results in a limit of f_{ut}/f_y of 1.4/0.75 = 1.87. For Appendix C, the average load factor of 1.55 (from $\mathbf{1.4}D$ + $\mathbf{1.7}L$), divided by the highest ϕ-factor (0.80 for tension), results in a limit of f_{ut}/f_y of 1.55/0.8 = 1.94. For consistent results, the serviceability limitation of f_{ut} was taken as $\mathbf{1.9}f_y$. If the ratio of f_{ut} to f_y exceeds this value, the anchoring may be subjected to service loads above f_y under service loads. Although not a concern for standard structural steel anchors (maximum value of f_{ut}/f_y is 1.6 for ASTM A 307), the limitation is applicable to some stainless steels.

D.5.2 — Concrete breakout strength of anchor in tension

D.5.2.1 — The nominal concrete breakout strength N_{cb} or N_{cbg} of an anchor or group of anchors in tension shall not exceed

For a single anchor

$$N_{cb} = \frac{A_N}{A_{No}}\psi_2\psi_3N_b \qquad \text{(D-4)}$$

For a group of anchors

$$N_{cbg} = \frac{A_N}{A_{No}}\psi_1\psi_2\psi_3N_b \qquad \text{(D-5)}$$

A_N is the projected area of the failure surface for the anchor or group of anchors that shall be approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward $1.5h_{ef}$ from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors. A_N shall not exceed nA_{No}, where n is the number of tensioned anchors in the group. A_{No} is the projected area of the failure surface of a single anchor remote from edges

$$A_{No} = 9h_{ef}^2 \qquad \text{(D-6)}$$

D.5.2.2 — The basic concrete breakout strength N_b of a single anchor in tension in cracked concrete shall not exceed

$$N_b = k\sqrt{f_c'}\,h_{ef}^{1.5} \qquad \text{(D-7)}$$

where

k = 24 for cast-in anchors; and

k = 17 for post-installed anchors.

Alternatively, for cast-in headed studs and headed

RD.5.2 — Concrete breakout strength of anchor in tension

RD.5.2.1 — The effects of multiple anchors, spacing of anchors, and edge distance on the nominal concrete breakout strength in tension are included by applying the modification factors A_N/A_{No} and ψ_2 in Eq. (D-4) and (D-5).

Figure RD.5.2.1(a) shows A_{No} and the development of Eq. (D-6). A_{No} is the maximum projected area for a single anchor. Figure RD.5.2.1(b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. Because A_N is the total projected area for a group of anchors, and A_{No} is the area for a single anchor, there is no need to include n, the number of anchors, in Eq. (D-4) or (D-5). If anchor groups are positioned in such a way that their projected areas overlap, the value of A_N is required to be reduced accordingly.

RD.5.2.2 — The basic equation for anchor capacity was derived[D.9-D.11,D.14] assuming a concrete failure prism with an angle of about 35 degrees, considering fracture mechanics concepts.

The values of k in Eq. (D-7) were determined from a large database of test results in uncracked concrete[D.9] at the 5 percent fractile. The values were adjusted to corresponding k values for cracked concrete.[D.10,D.19] Higher k values for post-installed anchors may be permitted, provided they have been determined from product approval testing in accor-

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bolts with 11 in. ≤ h_{ef} ≤ 25 in., the basic concrete breakout strength of a single anchor in tension in cracked concrete shall not exceed

$$N_b = 16\sqrt{f_c'}h_{ef}^{5/3} \quad \text{(D-8)}$$

COMMENTARY

dance with ACI 355.2. When using k values from ACI 355.2 product approval reports, ψ_3 shall be taken as 1.0 because the published test results of the ACI 355.2 product approval tests provide specific k values for cracked or uncracked concrete. For anchors with a deep embedment (h_{ef} > 11 in.), some test evidence indicates the use of $h_{ef}^{1.5}$ can be overly conservative for some cases. Often, such tests have been with selected aggregates for special applications. An alternative expression (Eq. (D-8)) is provided using $h_{ef}^{5/3}$ for evaluation of cast-in anchors with 11 in. ≤ h_{ef} ≤ 25 in. The limit of 25 in. corresponds to the upper range of test data. This expression can also be appropriate for some undercut post-installed anchors. D.4.2, however, should be used with test results to justify such applications.

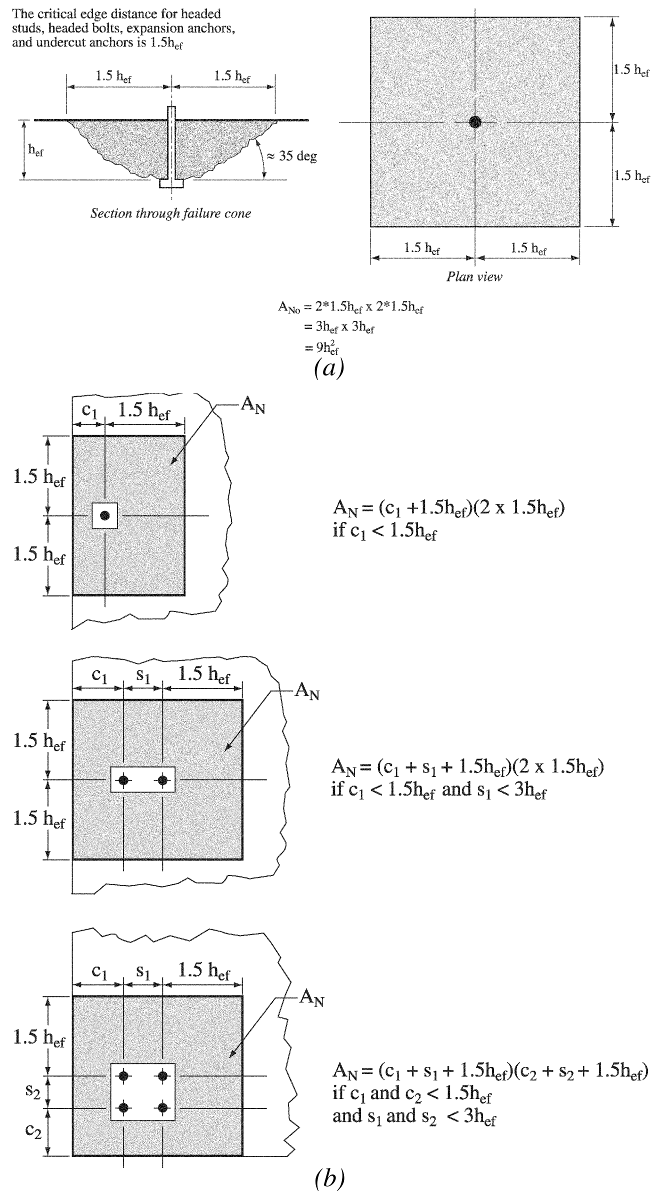

Fig. RD.5.2.1—(a) Calculation of A_{No}; and (b) projected areas for single anchors and groups of anchors and calculation of A_N.

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D.5.2.3 — For the special case of anchors in an application with three or four edges along with the largest edge distance $c_{max} \leq 1.5h_{ef}$, the embedment depth h_{ef} used in Eq. (D-6) through (D-11) shall be limited to $c_{max}/1.5$.

COMMENTARY

RD.5.2.3 — For anchors influenced by three or more edges where any edge distance is less than $1.5h_{ef}$, the tensile breakout strength computed by the ordinary CCD method, which is the basis for Eq. (D-7) and (D-8), gives misleading results. This occurs because the ordinary definitions of A_N/A_{No} do not correctly reflect the edge effects. If the value of h_{ef} is limited to $c_{max}/1.5$, however, where c_{max} is the largest of the influencing edge distances that are less than or equal to the actual $1.5h_{ef}$, this problem is corrected. As shown by Lutz,[D.20] this limiting value of h_{ef} is to be used in Eq. (D-6) to (D-11). This approach is best understood when applied to an actual case. Figure RD.5.2.3 shows how the failure surface has the same area for any embedment beyond the proposed limit on h_{ef} (taken as h'_{ef} in the figure). In this example, the proposed limit on the value of h_{ef} to be used in the computations where $h_{ef} = c_{max}/1.5$ results in $h_{ef} = h'_{ef}$ = 4 in./1.5 = 2.67 in. For this example, this would be the proper value to be used for h_{ef} in computing the resistance even if the actual embedment depth is larger.

D.5.2.4 — The modification factor for eccentrically loaded anchor groups is

$$\psi_1 = \frac{1}{\left(1 + \dfrac{2e'_N}{3h_{ef}}\right)} \leq 1 \qquad \text{(D-9)}$$

Equation (D-9) is valid for $e'_N \leq s/2$.

If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension shall be considered when determining the eccentricity e'_N for use in Eq. (D-9).

In the case where eccentric loading exists about two axes, the modification factor ψ_1 shall be computed for each axis individually and the product of these factors used as ψ_1 in Eq. (D-5).

RD.5.2.4 — Figure RD.5.2.4(a) shows dimension $e'_N = e_N$ for a group of anchors that are all in tension but that have a resultant force eccentric with respect to the centroid of the anchor group. Groups of anchors can be loaded in such a way that only some of the anchors are in tension (Fig. RD.5.2.4(b)).

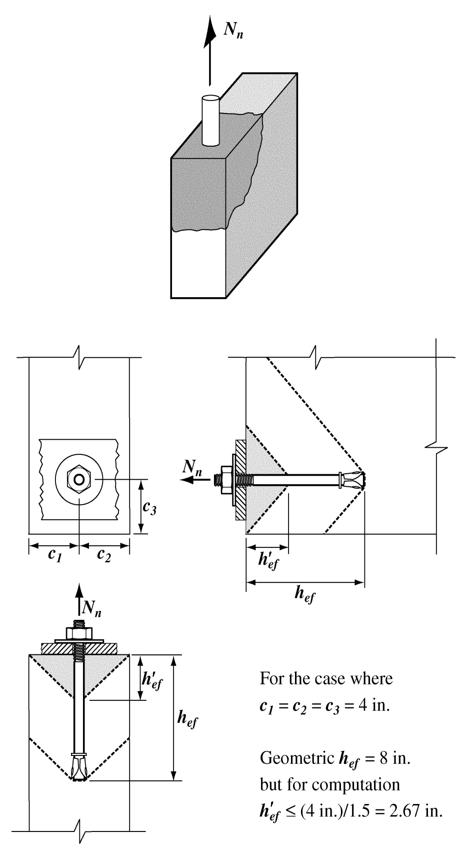

Fig. RD.5.2.3—Failure surfaces in narrow members for different embedment depths.

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D.5.2.5 — The modification factor for edge effects is

$$\psi_2 = 1 \text{ if } c_{min} \geq 1.5h_{ef} \qquad \text{(D-10)}$$

$$\psi_2 = 0.7 + 0.3\frac{c_{min}}{1.5h_{ef}} \text{ if } c_{min} < 1.5h_{ef} \qquad \text{(D-11)}$$

COMMENTARY

In this case, only the anchors in tension are to be considered in the determination of e'_N. The anchor loading has to be determined as the resultant anchor tension at an eccentricity with respect to the center of gravity of the anchors in tension. Equation (D-9) is limited to cases where $e'_N \leq s/2$ to alert the designer that all anchors may not be in tension.

RD.5.2.5 — If anchors are located close to an edge so that there is not enough space for a complete breakout prism to develop, the load-bearing capacity of the anchor is further reduced beyond that reflected in A_N/A_{No}. If the smallest side cover distance is greater than $1.5h_{ef}$, a complete prism can form and there is no reduction ($\psi_2 = 1$). If the side cover is less than $1.5h_{ef}$, the factor ψ_2 is required to adjust for the edge effect.[D.9]

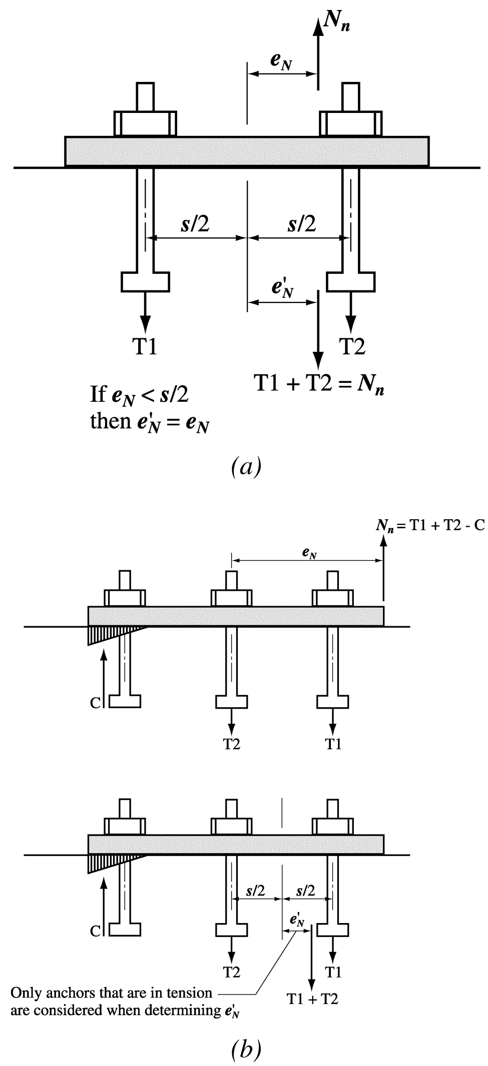

Fig. RD.5.2.4—Definition of dimension e'_N for group anchors: (a) when all anchors in group are in tension; and (b) when only some anchors in group are in tension.

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D.5.2.6 — When an anchor is located in a region of a concrete member where analysis indicates no cracking ($f_t < f_r$) at service load levels, the following modification factor shall be permitted:

ψ_3 = 1.25 for cast-in anchors

ψ_3 = 1.4 for post-installed anchors.

When analysis indicates cracking at service load levels, ψ_3 shall be taken as 1.0 for both cast-in anchors and post-installed anchors. Post-installed anchors shall be qualified for use in cracked concrete in accordance with ACI 355.2. The cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with 10.6.4, or equivalent crack control shall be provided by confining reinforcement.

D.5.2.7 — When an additional plate or washer is added at the head of the anchor, it shall be permitted to calculate the projected area of the failure surface by projecting the failure surface outward $1.5h_{ef}$ from the effective perimeter of the plate or washer. The effective perimeter shall not exceed the value at a section projected outward more than t from the outer edge of the head of the anchor, where t is the thickness of the washer or plate.

D.5.3 — Pullout strength of anchor in tension

D.5.3.1 — The nominal pullout strength N_{pn} of an anchor in tension shall not exceed

$$N_{pn} = \psi_4 N_p \qquad \text{(D-12)}$$

D.5.3.2 — For post-installed expansion and undercut anchors, the values of N_p shall be based on the 5 percent fractile of results of tests performed and evaluated according to ACI 355.2. It is not permissible to calculate the pullout strength in tension for such anchors.

D.5.3.3 — For single cast-in headed studs and headed bolts, it shall be permitted to evaluate the pullout strength in tension using D.5.3.4. For single J- or L-bolts, it shall be permitted to evaluate the pullout strength in tension using D.5.3.5. Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

D.5.3.4 — The pullout strength in tension of a single headed stud or headed bolt N_p for use in Eq. (D-12), shall not exceed

$$N_p = A_{brg} 8 f_c' \qquad \text{(D-13)}$$

COMMENTARY

RD.5.2.6 — Post-installed and cast-in anchors that have not met the requirements for use in cracked concrete according to ACI 355.2 should be used in uncracked regions only. The analysis for the determination of crack formation should include the effects of restrained shrinkage (see 7.12.1.2). The anchor qualification tests of ACI 355.2 require that anchors in cracked concrete zones perform well in a crack that is 0.012 in. wide. If wider cracks are expected, confining reinforcement to control the crack width to about 0.012 in. should be provided.

The concrete breakout strengths given by Eq. (D-7) assume cracked concrete (that is, ψ_3 = 1.0) with $\psi_3 k$ = 24 for cast-in-place, and 17 for post-installed (cast-in 40 percent higher). When the uncracked concrete ψ_3 factors are applied (1.25 for cast-in, and 1.4 for post-installed), the results are $\psi_3 k$ factors of 30 for cast-in and 24 for post-installed (25 percent higher for cast-in). This agrees with field observations and tests that show cast-in anchor strength exceeds that of post-installed for both cracked and uncracked concrete.

RD.5.3 — Pullout strength of anchor in tension

RD.5.3.2 — The pullout strength equations given in D.5.3.4 and D.5.3.5 are only applicable to cast-in headed and hooked anchors;[D.8,D.21] they are not applicable to expansion and undercut anchors that use various mechanisms for end anchorage unless the validity of the pullout strength equations are verified by tests.

RD.5.3.3 — The pullout strength in tension of headed studs or headed bolts can be increased by providing confining reinforcement, such as closely spaced spirals, throughout the head region. This increase can be demonstrated by tests.

RD.5.3.4 — Equation (D-13) corresponds to the load at which the concrete under the anchor head begins to crush.[D.8.D.15] It is not the load required to pull the anchor completely out of the concrete, so the equation contains no term relating to embedment depth. The designer should be aware that local crushing under the head will greatly reduce the stiffness of the connection, and generally will be the beginning of a pullout failure.

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D.5.3.5 — The pullout strength in tension of a single hooked bolt N_p for use in Eq. (D-12) shall not exceed

$$N_p = 0.9 f_c' e_h d_o \quad \text{(D-14)}$$

where $3d_o \leq e_h \leq 4.5d_o$.

D.5.3.6 — For an anchor located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ at service load levels, the following modification factor shall be permitted

$$\psi_4 = 1.4$$

Otherwise, ψ_4 shall be taken as 1.0.

D.5.4 — Concrete side-face blowout strength of a headed anchor in tension

D.5.4.1 — For a single headed anchor with deep embedment close to an edge $(c < 0.4h_{ef})$, the nominal side-face blowout strength N_{sb} shall not exceed

$$N_{sb} = 160c\sqrt{A_{brg}}\sqrt{f_c'} \quad \text{(D-15)}$$

If the single headed anchor is located at a perpendicular distance c_2 less than $3c$ from an edge, the value of N_{sb} shall be multiplied by the factor $(1 + c_2/c)/4$ where $1 \leq c_2/c \leq 3$.

D.5.4.2 — For multiple headed anchors with deep embedment close to an edge $(c < 0.4h_{ef})$ and spacing between anchors less than $6c$, the nominal strength of the group of anchors for a side-face blowout failure N_{sbg} shall not exceed

$$N_{sbg} = \left(1 + \frac{s_o}{6c}\right) N_{sb} \quad \text{(D-16)}$$

where s_o = spacing of the outer anchors along the edge in the group; and N_{sb} is obtained from Eq. (D-15) without modification for a perpendicular edge distance.

D.6 — Design requirements for shear loading

D.6.1 — Steel strength of anchor in shear

D.6.1.1 — The nominal strength of an anchor in shear as governed by steel V_s shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

D.6.1.2 — The nominal strength V_s of an anchor or group of anchors in shear shall not exceed (a) through (c):

(a) For cast-in headed stud anchors

$$V_s = nA_{se} f_{ut} \quad \text{(D-17)}$$

COMMENTARY

RD.5.3.5 — Equation (D-14) for hooked bolts was developed by Lutz based on the results of Reference D.21. Reliance is placed on the bearing component only, neglecting any frictional component because crushing inside the hook will greatly reduce the stiffness of the connection, and generally will be the beginning of pullout failure. The limits on e_h are based on the range of variables used in the three tests programs reported in Reference D.21.

RD.5.4 — Concrete side-face blowout strength of a headed anchor in tension

The design requirements for side-face blowout are based on the recommendations of Reference D.22. These requirements are applicable to headed anchors that usually are cast-in anchors. Splitting during installation rather than side-face blowout generally governs post-installed anchors, and is evaluated by the ACI 355.2 requirements.

RD.6 — Design requirements for shear loading

RD.6.1 — Steel strength of anchor in shear

RD.6.1.2 — The nominal shear strength of anchors is best represented by $A_{se} f_{ut}$ for headed stud anchors and $0.6A_{se} f_{ut}$ for other anchors rather than a function of $A_{se} f_y$ because typical anchor materials do not exhibit a well-defined yield point. The use of Eq. (D-17) and (D-18) with

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where f_{ut} shall not be taken greater than $1.9f_y$ or 125,000 psi;

(b) For cast-in headed bolt and hooked bolt anchors

$$V_s = n0.6A_{se}f_{ut} \qquad \text{(D-18)}$$

where f_{ut} shall not be taken greater than $1.9f_y$ or 125,000 psi;

(c) For post-installed anchors

$$V_s = n(0.6A_{se}f_{ut} + 0.4_{sl}f_{utsl}) \qquad \text{(D-19)}$$

where f_{ut} shall not be taken greater than $1.9f_y$ or 125,000 psi.

D.6.1.3 — Where anchors are used with built-up grout pads, the nominal strengths of D.6.1.2 shall be multiplied by a 0.80 factor.

D.6.2 — Concrete breakout strength of anchor in shear

D.6.2.1 — The nominal concrete breakout strength, V_{cb} or V_{cbg}, in shear of an anchor or group of anchors shall not exceed:

(a) For shear force perpendicular to the edge on a single anchor

$$V_{cb} = \frac{A_V}{A_{Vo}}\psi_6\psi_7V_b \qquad \text{(D-20)}$$

(b) For shear force perpendicular to the edge on a group of anchors

$$V_{cbg} = \frac{A_V}{A_{Vo}}\psi_5\psi_6\psi_7V_b \qquad \text{(D-21)}$$

(c) For shear force parallel to an edge, V_{cb} or V_{cbg} shall be permitted to be twice the value for shear force determined from Eq. (D-20) or (D-21), respectively, with ψ_6 taken equal to 1;

(d) For anchors located at a corner, the limiting nominal concrete breakout strength shall be determined for each edge, and the minimum value shall be used.

V_b is the basic concrete breakout strength value for a single anchor. A_V is the projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. It shall be permitted to evaluate this area as the base of a truncated half pyramid projected on the side face of the member where the top of the half pyramid is given by the axis of the anchor row selected as critical. The value of c_1 shall be taken as the distance from the edge to this axis. A_V shall not exceed nA_{Vo}, where n is the number of anchors in the group.

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9.2 load factors and the φ–factors of D.4.4 give design strengths consistent with the AISC load and resistance factor design specifications.[D.18]

The limitation of $1.9f_y$ on f_{ut} is to ensure that under service load conditions the anchor stress does not exceed f_y. The limit on f_{ut} of $1.9f_y$ was determined by converting the LRFD provisions to corresponding service level conditions as discussed in RD.5.1.2.

RD.6.2 — Concrete breakout strength of anchor in shear

RD.6.2.1 — The shear strength equations were developed from the CCD method. They assume a breakout cone angle of approximately 35 degrees (see Fig. RD.4.2.2(b)), and consider fracture mechanics theory. The effects of multiple anchors, spacing of anchors, edge distance, and thickness of the concrete member on nominal concrete breakout strength in shear are included by applying the reduction factor A_V/A_{Vo} in Eq. (D-20) and (D-21), and ψ_5 in Eq. (D-21). For anchors far from the edge, D.6.2 usually will not govern. For these cases, D.6.1 and D.6.3 often govern.

Figure RD.6.2.1(a) shows A_{Vo} and the development of Eq. (D-22). A_{Vo} is the maximum projected area for a single anchor that approximates the surface area of the full breakout prism or cone for an anchor unaffected by edge distance, spacing, or depth of member. Figure RD.6.2.1(b) shows examples of the projected areas for various single anchor and multiple anchor arrangements. A_V approximates the full surface area of the breakout cone for the particular arrangement of anchors. Because A_V is the total projected area for a group of anchors, and A_{Vo} is the area for a single anchor, there is no need to include the number of anchors in the equation.

The assumption shown in the upper right example of Fig. RD.6.2.1(b), with the case for two anchors perpendicular to the edge, is a conservative interpretation of the distribution of the shear force on an elastic basis. If the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load would be transferred to the stiffer and stronger rear anchor. For cases where nominal strength is not controlled by ductile steel elements, D.3.1 requires that load effects be determined by elastic analysis. The PCI Design Handbook approach[D.17] suggests in Section 6.5.2.2 that the increased capacity of the anchors away from the edge be considered. Because this is a reasonable approach, assuming that the

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A_{vo} is the projected area for a single anchor in a deep member and remote from edges in the direction perpendicular to the shear force. It shall be permitted to evaluate this area as the base of a half pyramid with a side length parallel to the edge of $3c_1$ and a depth of $1.5c_1$

$$A_{vo} = 4.5(c_1)^2 \qquad \text{(D-22)}$$

Where anchors are located at varying distances from the edge and the anchors are welded to the attachment so as to distribute the force to all anchors, it shall be permitted to evaluate the strength based on the distance to the farthest row of anchors from the edge. In this case, it shall be permitted to base the value of c_1 on the distance from the edge to the axis of the farthest anchor row that is selected as critical, and all of the shear shall be assumed to be carried by this critical anchor row alone.

D.6.2.2 — The basic concrete breakout strength V_b in shear of a single anchor in cracked concrete shall not exceed

$$V_b = 7\left(\frac{\ell}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}(c_1)^{1.5} \qquad \text{(D-23)}$$

D.6.2.3 — For cast-in headed studs, headed bolts, or hooked bolts that are continuously welded to steel attachments having a minimum thickness equal to the greater of 3/8 in. or half of the anchor diameter, the basic concrete breakout strength V_b in shear of a single anchor in cracked concrete shall not exceed

$$V_b = 8\left(\frac{\ell}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}(c_1)^{1.5} \qquad \text{(D-24)}$$

COMMENTARY

anchors are spaced far enough apart so that the shear failure surfaces do not intersect,[D.11] D.6.2 allows such a procedure. If the failure surfaces do not intersect, as would generally occur if the anchor spacing s is equal to or greater than $1.5c_1$, then after formation of the near-edge failure surface, the higher capacity of the farther anchor would resist most of the load. As shown in the bottom right example in Fig. RD.6.2.1(b), it would be appropriate to consider the full shear capacity to be provided by this anchor with its much larger resisting failure surface. No contribution of the anchor near the edge is then considered. Checking the near-edge anchor condition is advisable to preclude undesirable cracking at service load conditions. Further discussion of design for multiple anchors is given in Reference D.8.

For the case of anchors near a corner subjected to a shear force with components normal to each edge, a satisfactory solution is to check independently the connection for each component of the shear force. Other specialized cases, such as the shear resistance of anchor groups where all anchors do not have the same edge distance, are treated in Reference D.11.

The detailed provisions of D.6.2.1(a) apply to the case of shear force directed towards an edge. When the shear force is directed away from the edge, the strength will usually be governed by D.6.1 or D.6.3.

The case of shear force parallel to an edge is shown in Fig. RD.6.2.1(c). A special case can arise with shear force parallel to the edge near a corner. In the example of a single anchor near a corner (see Fig. RD.6.2.1(d)), where the edge distance to the side c_2 is 40 percent or more of the distance c_1 in the direction of the load, the shear strength parallel to that edge can be computed directly from Eq. (D-20) and (D-21) using c_1 in the direction of the load.

RD.6.2.2 — Like the concrete breakout tensile capacity, the concrete breakout shear capacity does not increase with the failure surface, which is proportional to $(c_1)^2$. Instead the capacity increases proportionally to $(c_1)^{1.5}$ due to size effect. The capacity is also influenced by the anchor stiffness and the anchor diameter.[D.9-D.11,D.14]

The constant, 7, in the shear strength equation was determined from test data reported in Reference D.9 at the 5 percent fractile adjusted for cracking.

RD.6.2.3 — For the special case of cast-in headed bolts continuously welded to an attachment, test data[D.23,D.24] show that somewhat higher shear capacity exists, possibly due to the stiff welding connection clamping the bolt more effectively than an attachment with an anchor gap. Because of this, the basic shear value for such anchors is increased. Limits are imposed to ensure sufficient rigidity. The design of supplementary reinforcement is discussed in References D.8, D.11, and D.12.

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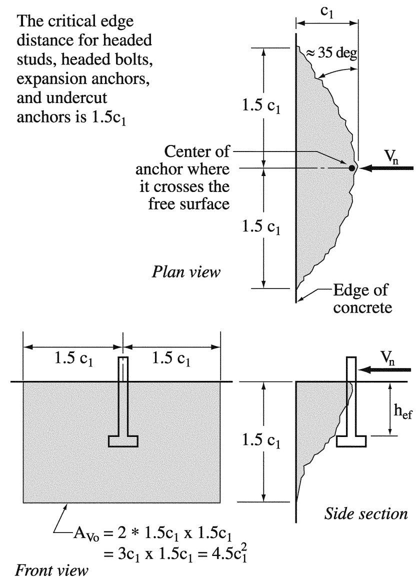

Fig. RD.6.2.1(a)—Calculation of A_{Vo}.

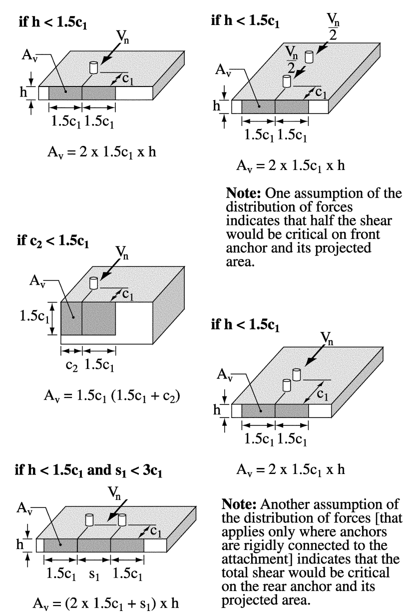

Fig. RD.6.2.1(b)—Projected area for single anchors and groups of anchors and calculation of A_V.

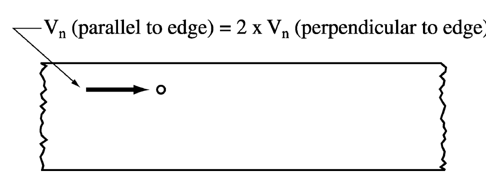

Fig. RD.6.2.1(c)—Shear force parallel to an edge.

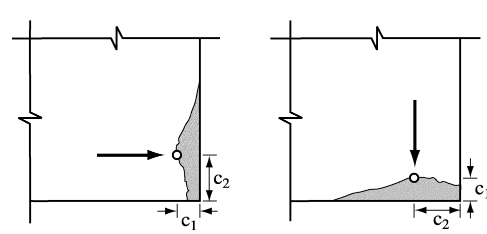

Fig. RD.6.2.1(d)—Shear force near a corner.

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provided that:

(a) For groups of anchors, the strength is determined based on the strength of the row of anchors farthest from the edge;

(b) The center-to-center spacing of the anchors is not less than 2.5 in.; and

(c) Supplementary reinforcement is provided at the corners if $c_2 \leq 1.5e_f$.

D.6.2.4 — For the special case of anchors influenced by three or more edges, the edge distance c_1 used in Eq. (D-22), (D-23), (D-24), (D-25), (D-26), and (D-27) shall be limited to $h/1.5$.

D.6.2.5 — The modification factor for eccentrically loaded anchor groups is

$$\psi_5 = \frac{1}{1 + \dfrac{2e_V'}{3c_1}} \leq 1 \qquad \text{(D-25)}$$

Equation (D-25) is valid for $e_V' \leq s/2$.

D.6.2.6 — The modification factor for edge effect is

$$\psi_6 = 1.0 \text{ if } c_2 \geq 1.5c_1 \qquad \text{(D-26)}$$

$$\psi_6 = 0.7 + 0.3(c_2/1.5c_1) \text{ if } c_2 < 1.5c_1 \qquad \text{(D-27)}$$

D.6.2.7 — For anchors located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ at service loads, the following modification factor shall be permitted

$$\psi_7 = 1.4$$

For anchors located in a region of a concrete member where analysis indicates cracking at service load levels, the following modification factors shall be permitted:

ψ_7 **= 1.0** for anchors in cracked concrete with no supplementary reinforcement or edge reinforcement smaller than a No. 4 bar;

ψ_7 **= 1.2** for anchors in cracked concrete with supplementary reinforcement of a No. 4 bar or greater between the anchor and the edge; and

COMMENTARY

RD.6.2.4 — For anchors influenced by three or more edges where any edge distance is less than $1.5c_1$, the shear breakout strength computed by the basic CCD Method, which is the basis for Eq. (D-23) and (D-24), gives safe but misleading results. These special cases were studied for the κ method[D.14] and the problem was pointed out by Lutz.[D.20] Similar to the approach used for tensile breakouts in D.5.2.3, a correct evaluation of the capacity is determined if the value of c_1 to be used in Eq. (D-22) to (D-27) is limited to $h/1.5$.

RD.6.2.5 — This section provides a modification factor for an eccentric shear force towards an edge on a group of anchors. If the shear load originates above the plane of the concrete surface, the shear should first be resolved as a shear in the plane of the concrete surface, with a moment that may or may not also cause tension in the anchors, depending on the normal force. Figure RD.6.2.5 defines the term e'_v for calculating the ψ_5 modification factor that accounts for the fact that more shear is applied on one anchor than the other, tending to split the concrete near an edge. If $e'_v \leq s/2$, the CCD procedure is not applicable.

RD.6.2.7 — Torque-controlled and displacement-controlled expansion anchors are permitted in cracked concrete under pure shear loadings.

CODE

ψ_7 **= 1.4** for anchors in cracked concrete with supplementary reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the supplementary reinforcement enclosed within stirrups spaced at not more than 4 in.

D.6.3 — Concrete pryout strength of anchor in shear

D.6.3.1 — The nominal pryout strength V_{cp} shall not exceed

$$V_{cp} = k_{cp}N_{cb} \tag{D-28}$$

where

k_{cp} = 1.0 for h_{ef} < 2.5 in.;

k_{cp} = 2.0 for $h_{ef} \geq$ 2.5 in.; and

N_{cb} shall be determined from Eq. (D-4), lb.

D.7 — Interaction of tensile and shear forces

Unless determined in accordance with D.4.3, anchors or groups of anchors that are subjected to both shear and axial loads shall be designed to satisfy the requirements of D.7.1 through D.7.3. The value of ϕN_n shall be as required in D.4.1.2. The value of ϕV_n shall be as defined in D.4.1.2.

D.7.1 — If $V_u \leq 0.2\phi V_n$, then full strength in tension shall be permitted: $\phi N_n \geq N_u$.

D.7.2 — If $N_u \leq 0.2\phi N_n$, then full strength in shear shall be permitted: $\phi V_n \geq V_u$.

D.7.3 — If $V_u > 0.2\phi V_n$ and $N_u > 0.2\phi N_n$, then

$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \leq 1.2 \tag{D-29}$$

COMMENTARY

RD.6.3 — Concrete pryout strength of anchor in shear

Reference D.9 indicates that the pryout shear resistance can be approximated as one to two times the anchor tensile resistance with the lower value appropriate for h_{ef} less than 2.5 in.

RD.7 — Interaction of tensile and shear forces

The shear-tension interaction expression has traditionally been expressed as

$$\left(\frac{N_n}{N_n}\right)^{\alpha} + \left(\frac{V_u}{V_n}\right)^{\alpha} \leq 1.0$$

where α varies from 1 to 2. The current trilinear recommendation is a simplification of the expression where α = 5/3 (Fig. RD.7). The limits were chosen to eliminate the requirement for computation of interaction effects where very small values of the second force are present. Any other interaction expression that is verified by test data, however, can be used to satisfy D.4.3.

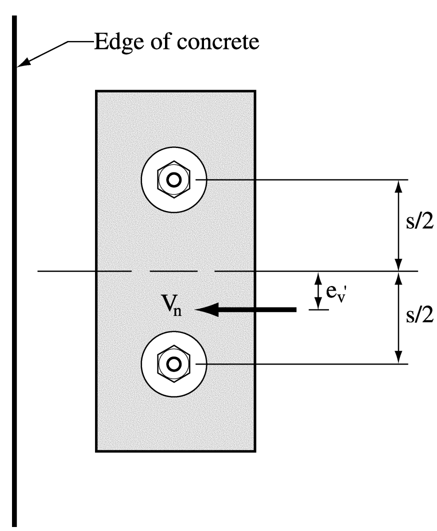

Fig. RD.6.2.5—Definition of dimensions e_V'.

CODE

D.8 — Required edge distances, spacings, and thicknesses to preclude splitting failure

Minimum spacings and edge distances for anchors and minimum thicknesses of members shall conform to D.8.1 through D.8.5, unless supplementary reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with ACI 355.2 shall be permitted.

D.8.1 — Unless determined in accordance with D.8.4, minimum center-to-center spacing of anchors shall be $4d_o$ for untorqued cast-in anchors, and $6d_o$ for torqued cast-in anchors and post-installed anchors.

D.8.2 — Unless determined in accordance with D.8.4, minimum edge distances for cast-in headed anchors that will not be torqued shall be based on minimum cover requirements for reinforcement in 7.7. For cast-in headed anchors that will be torqued, the minimum edge distances shall be $6d_o$.

D.8.3 — Unless determined in accordance with D.8.4, minimum edge distances for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement in 7.7, or the minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2, and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific ACI 355.2 test information, the minimum edge distance shall be taken as not less than:

Undercut anchors .... $6d_o$

Torque-controlled anchors .... $8d_o$

Displacement-controlled anchors .... $10d_o$

D.8.4 — For anchors where installation does not produce a splitting force and that will remain untorqued, if the edge distance or spacing is less than those specified in D.8.1 to D.8.3, calculations shall be performed by substituting for d_o a smaller value d'_o that meets the requirements of D.8.1 to D.8.3. Calculated forces applied to the anchor shall be limited to the values corresponding to an anchor having a diameter of d'_o.

COMMENTARY

RD.8 — Required edge distances, spacings, and thicknesses to preclude splitting failure

The minimum spacings, edge distances, and thicknesses are very dependent on the anchor characteristics. Installation forces and torques in post-installed anchors can cause splitting of the surrounding concrete. Such splitting also can be produced in subsequent torquing during connection of attachments to anchors including cast-in anchors. The primary source of values for minimum spacings, edge distances, and thicknesses of post-installed anchors should be the product-specific tests of ACI 355.2. In some cases, however, specific products are not known in the design stage. Approximate values are provided for use in design.

RD.8.2 — Because the edge cover over a deep embedment close to the edge can have a significant effect on the side-face blowout strength of D.5.4, in addition to the normal concrete cover requirements, the designer may wish to use larger cover to increase the side-face blowout strength.

RD.8.3 — Drilling holes for post-installed anchors can cause microcracking. The requirement for a minimum edge distance twice the maximum aggregate size is to minimize the effects of such microcracking.

RD.8.4 — In some cases, it may be desirable to use a larger-diameter anchor than the requirements on D.8.1 to D.8.3 permit. In these cases, it is permissible to use a larger-diameter anchor provided the design strength of the anchor is based on a smaller assumed anchor diameter d'_o.

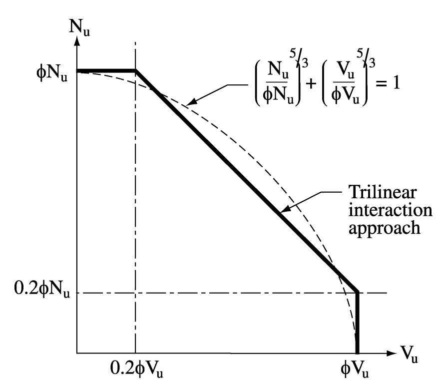

Fig. RD.7—Shear and tensile load interaction equation.

CODE

D.8.5 — The value of h_{ef} for an expansion or undercut post-installed anchor shall not exceed the greater of either 2/3 of the member thickness or the member thickness less 4 in.

D.8.6 — Project drawings and project specifications shall specify use of anchors with a minimum edge distance as assumed in design.

D.9 — Installation of anchors

D.9.1 — Anchors shall be installed in accordance with the project drawings and project specifications.

COMMENTARY

RD.8.5 — This minimum thickness requirement is not applicable to through-bolts because they are outside the scope of Appendix D. In addition, splitting failures are caused by the load transfer between the bolt and the concrete. Because through-bolts transfer their load differently than cast-in or expansion and undercut anchors, they would not be subject to the same member thickness requirements. Post-installed anchors should not be embedded deeper than 2/3 of the member thickness.

RD.9 — Installation of anchors

Many anchor performance characteristics depend on proper installation of the anchor. Anchor capacity and deformations can be assessed by acceptance testing under ACI 355.2. These tests are carried out assuming that the manufacturer's installation directions will be followed. Certain types of anchors can be sensitive to variations in hole diameter, cleaning conditions, orientation of the axis, magnitude of the installation torque, crack width, and other variables. Some of this sensitivity is indirectly reflected in the assigned ϕ values for the different anchor categories, which depend in part on the results of the installation safety tests. Gross deviations from the ACI 355.2 acceptance testing results could occur if anchor components are incorrectly exchanged, or if anchor installation criteria and procedures vary from those recommended. Project specifications should require that anchors be installed according to the manufacturer's recommendations.

APPENDIX E — NOTATION

Items set in Times Roman type appear only in the commentary.

a = depth of rectangular stress block. Chapter 9

a = depth of equivalent rectangular tress block as defined in 10.2.7.1, in. Chapters 10 and 12

a = shear span, distance between concentrated load and face of support, in. Chapter 11

A = area of that part of cross section between flexural tension face and center of gravity of gross section, in.^{2} Chapter 18

A_b = area of an individual horizontal bar or wire, in.^{2} Chapter 10

A_b = area of an individual bar, in.^{2} Chapter 12

A_{brg} = bearing area of the head of stud or anchor bolt, in.^{2}, Appendix D

A_c = area of core of spirally reinforced compression member measured to outside diameter of spiral, in.^{2} Chapter 10

A_c = area of concrete section resisting shear transfer, in.^{2} Chapter 11

A_c = area of concrete of assumed critical section for transfer of moment at slab-column connection, in.^{2} See Fig. R11.12.6.2. Chapter 11

A_c = area of contact surface being investigated for horizontal shear, in.^{2} Chapter 17

A_{cf} = larger cross-sectional area of the slab-beam strips of the two orthogonal equivalent frames intersecting at a column of a two-way slab, in^2. Chapter 14

A_{ch} = cross-sectional area of a structural member measured out-to-out of transverse reinforcement, in.^{2} Chapter 21

A_{cp} = area enclosed by outside perimeter of concrete cross section, in.^{2} See 11.6.1. Chapter 11

A_{cp} = area of concrete section, resisting shear, of an individual pier or horizontal wall segment, in.^{2} Chapter 21

A_{cv} = gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, in.^{2} Chapter 21

A_f = area of reinforcement in bracket or corbel resisting factored moment, $[V_u a + N_{uc}(h - d)]$, in.^{2} Chapter 11

A_f = base area of footing, in.^{2} Chapter 15

A_g = gross area of section, in.^{2} Chapters 9, 10, 14, 15, 21, and Appendix I

A_g = gross area of section, in.^{2} For a hollow section A_g is the area of the concrete only and does not include the area of the void(s). Chapter 11

A_g = gross area of column, in.^{2} Chapter 16

A_h = area of shear reinforcement parallel to flexural tension reinforcement, in.^{2} Chapter 11

A_j = effective cross-sectional area within a joint, in.^{2}, see 21.5.3.1, in a plane parallel to plane of reinforcement generating shear in the joint. The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of:

(a) beam width plus the joint depth;

(b) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. See 21.5.3.1. Chapter 21

A_ℓ = total area of longitudinal reinforcement to resist torsion, in.^{2} Chapter 11

A_n = area of reinforcement in bracket or corbel resisting tensile force N_{uc}, in.^{2} Chapter 11

A_N = projected concrete failure area of an anchor or group of anchors, for calculation of strength in tension, as defined in D.5.2.1, in.^{2} A_N shall not be taken greater than nA_{No}. [See Fig. RD.5.2.1 (b)], Appendix D

A_{No} = projected concrete failure area of one anchor, for calculation of strength in tension when not limited by edge distance or spacing, as defined in D.5.2.1, in.^{2} [See Fig. RD.5.2.1 (a)], Appendix D

A_o = gross area enclosed by shear flow path, in.^{2} Chapter 11

A_{oh} = area enclosed by centerline of the outermost closed transverse torsional reinforcement, in.^{2} Chapter 11

A_{ps} = area of prestressed reinforcement in tension zone, in.^{2} Chapters 11 and 18

A_s = area of nonprestressed tension reinforcement, in.^{2} Chapters 8, 10, 11, 12, 18, and 21

A_s = area of tension reinforcement. Chapter 9

A_s = area of steel, in.^{2} Chapter 9

A_s = area of longitudinal tension reinforcement in wall segment, in.^{2} Chapter 14

A'_s = area of compression reinforcement, in.^{2} Chapters 8, 9, and 18

A_{se} = area of effective longitudinal tension reinforcement in wall segment, in.^{2}, as calculated by Eq. (14-8)

A_{se} = effective cross-sectional area of anchor, in.^{2}, Appendix D

A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension h_c, in.^{2} Chapter 21

A_{si} = effective cross-sectional area of expansion or undercut anchor sleeve, if sleeve is within shear plane, in.^{2}, Appendix D

$A_{s,min}$ = minimum amount of flexural reinforcement, in.^{2} See 10.5. Chapter 10

A_{st} = total area of longitudinal reinforcement, (bars or steel shapes), in.^{2} Chapter 10

A_t = area of structural steel shape, pipe, or tubing in a composite section, in.^{2} Chapter 10

A_t = area of one leg of a closed stirrup resisting torsion within a distance s, in.^{2} Chapter 11

A_{tr} = total cross-sectional area of all transverse reinforcement that is within the spacing s and that crosses the potential plane of splitting through the reinforcement being developed, in.^{2} Chapter 12

A_v = area of shear reinforcement within a distance s, or area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s for deep flexural members, in.^{2} Chapter 11

A_v = area of shear reinforcement within a distance s, in.^{2} Chapter 12 and Appendix I

A_v = area of ties within a distance s, in.^{2}, Chapters 17 and 21.

A_v = projected concrete failure area of an anchor or group of anchors, for calculation of strength in shear, as defined in D.6.2.1, in.^{2} A_v shall not be taken greater than nA_{vo}. [See Fig. RD.6.2.1(b)], Appendix D

A_{vd} = total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam, in.^{2} Chapter 21

A_{vf} = area of shear-friction reinforcement, in.^{2} Chapter 11

A_{vh} = area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2, in.^{2} Chapter 11

A_{vo} = projected concrete failure area of one anchor, for calculation of strength in shear, when not limited by corner influences, spacing, or member thickness, as defined in D.6.2.1, in.^{2} [See Fig. RD.6.2.1(a)], Appendix D

A_w = area of an individual wire to be developed or spliced, in.^{2} Chapter 12

A_1 = loaded area, in.^{2} Chapter 10 and Appendix I

A_2 = the area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal, in.^{2} Chapter 10

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} Appendix I

b = width of compression face of member, in. Chapters 8, 9, 10, 11, and 18

b = effective compressive flange width of a structural member, in. Chapter 21

b_o = perimeter of critical section for slabs and footings, in. Chapter 11 and Appendix I

b_o = critical perimeter for shear for pile groups. Chapter 15

b_t = width of that part of cross section containing the closed stirrups resisting torsion, in. Chapter 11

b_v = width of cross section at contact surface being investigated for horizontal shear, in. Chapter 17

b_w = web width, in. Chapter 10

b_w = web width, or diameter of circular section, in. Chapters 11, 12, 21, and Appendix I

b_1 = width of the critical section defined in 11.12.1.2 measured in the direction of the span for which moments are determined, in. Chapters 11 and 13

b_2 = width of the critical section defined in 11.12.1.2 measured in the direction perpendicular to b_1, in. Chapters 11 and 13

B_c = buckling reduction factor for creep, nonlinearity, and cracking of concrete. Appendix G

B_i = buckling reduction factor for geometrical imperfections. Appendix G

c = distance from extreme compression fiber to neutral axis, in. Chapters 9, 10, and 14

c = spacing or cover dimension, in. See 12.2.4. Chapter 12

c = distance from the extreme compression fiber to neutral axis, see 10.2.7, calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth, in. Chapter 21

c = distance from center of an anchor shaft to the edge of concrete, in., Appendix D

c_{AB} = distance from centroidal axis of critical section to perimeter of c_{CD} critical section. See Fig. R11.12.6.2. Chapter 11

c_c = clear cover from the nearest surface in tension to the surface of the flexural tension steel, in. Chapters 10 and 14

c_{max} = the largest edge distance, in., Appendix D

c_{min} = the smallest edge distance, in., Appendix D

c_t = dimension equal to the distance from the interior face of the column to the slab edge measured parallel to c_1, but not exceeding c_1, in.

c_1 = size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, in. Chapters 11, 13, and 21

c_1 = distance from the center of an anchor shaft to the edge of concrete in one direction, in.; where shear force is applied to anchor, c_1 is in the direction of the shear force. [See Fig. RD.6.2.1(a)], Appendix D

c_2 = size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which moments are being determined, in. Chapters 11 and 13

c_2 = distance from center of an anchor shaft to the edge of concrete in the direction orthogonal to c_1, in. Appendix D

C = cross-sectional constant to define torsional properties. The constant C for T- or L-sections shall be permitted to be evaluated by dividing the section into separate rectangular parts and summing the values of C for each part. Chapter 13

$$= \Sigma\left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3}$$

C_m = a factor relating actual moment diagram to an equivalent uniform moment diagram. Chapter 10

d = distance from extreme compression fiber to centroid of tension reinforcement, in. Chapters 7, 8, 9, 10, 12, and Appendix I

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, but need not be less than $0.80h$ for prestressed members, in. (For circular sections, d need not be less than the distance from extreme compression fiber to centroid of tension reinforcement in opposite half of member). Chapter 11

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in. Chapter 14

d = effective depth of footing. Chapter 15

d = distance from extreme compression fiber to centroid of tension reinforcement, in. Chapter 7, 8

d = distance from extreme compression fiber to centroid of tension reinforcement for entire composite section, in. Chapter 17

d = distance from extreme compression fiber to centroid of nonprestressed tension reinforcement, in. Chapter 18

d = effective depth of section, in. Chapter 21

d' = distance from extreme compression fiber to centroid of compression reinforcement, in. Chapters 9 and 18

d_b = nominal diameter of bar, wire, or prestressing strand, in. Chapters 7, 10, and 12

d_b = diameter of flexural reinforcement. Chapter 11

d_b = bar diameter, in. Chapter 21

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto, in. Chapter 10

d_o = outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, in. (See also D.8.4), Appendix D

d_o' = value substituted for d_o when an oversized anchor is used, in. (See D.8.4), Appendix D

d_p = diameter of pile at footing base, in. Chapter 15

d_p = distance from extreme compression fiber to centroid of prestressed reinforcement, in. Chapter 18

d_s = distance from extreme tension fiber to centroid of tension reinforcement, in. Chapter 9

d_t = distance from extreme compression fiber to extreme tension steel, in. Chapter 9

D = dead loads, or related internal moments and forces. Chapters 9, 18, and 20

D = dead loads, or related internal moments and forces. Chapter 21

D = dead load consisting of: (a) weight of the member itself; (b) weight of all materials of construction incorporated into the building to be permanently supported by the member, including built-in partitions; and (c) weight of permanent equipment. Appendix C

D_i = resolution of V_i into diagonal compression force. Chapter 11

D_T = inside diameter of circular tank, ft. Chapter 21

e = eccentricity of load parallel to axis of member measured from centroid of gross section. Chapter 10

e = base of Napierian logarithms. Chapter 18

e_h = distance from the inner surface of the shaft of a J- or L-bolt to the outer tip of the J- or L-bolt, in., Appendix D

e'_N = eccentricity of normal force on a group of anchors; the distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension, in.; e'_N is always positive [See Fig. RD.5.2.4(a) and (b)], Appendix D

e'_V = eccentricity of shear force on a group of anchors; the distance between the point of shear force application and the centroid of the group of anchors resisting shear in the direction of the applied shear, in., Appendix D

E = load effects of earthquake, or related internal moments and forces. Chapters 9 and 21

E = earthquake load. Appendix C

E_c = modulus of elasticity of concrete, psi. See 8.5.1. Chapters 8, 9, 10, 14, 19, and Appendix I

E_c = modulus of elasticity of concrete under short-term load, psi. Appendix G

E_{cb} = modulus of elasticity of beam concrete, psi. Chapter 13

E_{cs} = modulus of elasticity of slab concrete, psi. Chapter 13

EI = relative flexural stiffness of member. Chapter 8

EI = flexural stiffness of compression member. See Eq. (10-12) and (10-13). Chapter 10

E_s = modulus of elasticity of reinforcement, psi. See 8.5.2 or 8.5.3. Chapters 8, 10, and Appendix I

f'_c = specified compressive strength of concrete, psi. Chapters 4, 5, 8, 9, 10, 11, 12, 14, 18, 19, 20, 21, Appendixes I, D, and G

f'_{cr} = required average concrete strength, psi. Chapter 4

f'_{cr} = required average compressive strength of concrete used as the basis for selection of concrete proportions, psi. Chapter 5

$\sqrt{f_c'}$ = square root of specified compressive strength of concrete, psi. Chapters 9, 11, 12, 18, 19, 21, and Appendix I

f'_{ci} = compressive strength of concrete at time of initial prestress, psi. Chapters 7 and 18

$\sqrt{f_{ci}'}$ = square root of compressive strength of concrete at time of initial prestress, psi. Chapter 18

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi. Chapters 9, 11, 12, Appendix I, and Appendix D

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, psi. Chapter 11

f_{dc} = decompression stress. Stress in the prestressing steel when stress is zero in the concrete at the same level as the centroid of the tendons, ksi. Chapter 18

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, psi. (In a composite member, f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone). Chapter 11

f_{pc} = average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses), psi. Chapter 18

f_{pe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads, psi. Chapter 11

f_{ps} = prestressing tendon stress at ultimate at section of maximum moment. Chapter 11

f_{ps} = stress in prestressed reinforcement at nominal strength. See text for units. Chapters 12 and 18

f_{pu} = specified tensile strength of prestressing steel, psi. Chapters 11 and 18

f_{py} = specified yield strength of prestressing tendons, psi. Chapter 18

f_r = modulus of rupture of concrete, psi. Chapters 9, 18, and Appendix D

f_s = calculated stress in reinforcement at service loads, ksi. Chapter 9 and 10

f_s = maximum allowable stress in reinforcement at service loads ksi. Chapter 10

f_s = permissible tensile stress in reinforcement, psi. Appendix I

f_{se} = effective prestressing tendon stress after all prestress losses. Chapter 11

f_{se} = effective stress in prestressed reinforcement (after allowance for all prestress losses). See text for units. Chapters 12 and 18

f_t = extreme fiber stress in tension in the precompressed tensile zone computed using gross section properties, psi. See text for units. Chapter 18

f_t = calculated concrete tensile stress in a region of a member, psi, Appendix D

f_{ut} = specified tensile strength of anchor steel, psi, Appendix D

f_{utsi} = specified tensile strength of anchor sleeve, psi, Appendix D

f_y = specified yield strength of nonprestressed reinforcement, psi. Chapters 3, 7, 8, 9, 10, 11, 12, 14, 17, 18, 19, 21, and Appendix I

f_y = yield strength of tension reinforcement. Chapter 20

f_y = specified yield strength of anchor steel, psi, Appendix D

f_{yh} = specified yield strength of circular tie, hoop, or spiral reinforcement, psi. Chapter 11

f_{yh} = specified yield strength of transverse reinforcement, psi. Chapter 21

$f_{y\ell}$ = yield strength of longitudinal torsional reinforcement, psi. Chapter 11

f_{yt} = specified yield strength of transverse reinforcement, psi. Chapter 12

f_{yv} = yield strength of closed transverse torsional reinforcement, psi. Chapter 11

F = loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces. Chapter 9

F = loads due to fluids with well-defined pressures and maximum heights. Appendix C

GJ = relative torsional stiffness of member. Chapter 8

h = overall thickness of member, in. Chapters 9, 10, 11, 12, 13, 14, 18, 20, and 21

h = overall thickness of composite member, in. Chapter 11 and 17

h = thickness of shell or folded plate, in. Chapter 19

h = thickness of member in which an anchor is anchored, measured parallel to anchor axis, in. Appendix D

h_c = cross-sectional dimension of column core measured center-to-center of confining reinforcement, in. Chapter 21

h_d = dome shell thickness, in. Appendix G

h_{ef} = effective anchor embedment depth, in. (See D.8.5 and Fig. RD.0). Appendix D

h_v = total depth of shearhead cross section, in. Chapter 11

h_w = total height of wall from base to top, in. Chapter 11

h_w = height of entire wall (diaphragm) or of the segment of wall (diaphragm) considered, in. Chapter 21

h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the column, in. Chapter 21

H = loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces. Chapter 9

H = loads due to the weight and lateral pressure of soil and water in soil. Appendix C

H_L = depth of stored liquid, ft. Chapter 21

I = moment of inertia of section resisting externally applied factored loads, in.^{4} Chapter 11

I_b = moment of inertia about centroidal axis of gross section of beam as defined in 13.2.4, in.^{4} Chapter 13

I_{cr} = moment of inertia of cracked section transformed to concrete, in.^{4} Chapter 9 and 14

I_e = effective moment of inertia for computation of deflection, in.^{4} Chapter 9 and 14

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.^{4} Chapters 9 and 10

I_s = moment of inertia about centroidal axis of gross section of slab, in.^{4} Chapter 13

= $h^3/12$ times width of slab defined in notations α and β_t

I_{se} = moment of inertia of reinforcement about centroidal axis of member cross section, in.^{4} Chapter 10

I_t = moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross section, in.^{4} Chapter 10

jd = moment arm at a section, in. Chapter 12

J_c = property of assumed critical section analogous to polar moment of inertia. See Fig. 11.12.6.2. Chapter 11

k = effective length factor for compression members. Chapter 10

k = effective length factor. Chapter 14

k = coefficient for basic concrete breakout strength in tension, Appendix D

k_{cp} = coefficient for pryout strength, Appendix D

K = wobble friction coefficient per foot of tendon. Chapter 18

K_t = torsional stiffness of torsional member; moment per unit rotation. See R13.7.5. Chapter 13

K_{tr} = transverse reinforcement index. Chapter 12

= $\dfrac{A_{tr} f_{yt}}{1500sn}$ (constant 1500 carries the unit lb/in.^{2})

K_1 = factor to determine portion of shear strength provided by concrete at a section. Chapter 11

ℓ = span length of beam or one-way slab, as defined in 8.7; clear projection of cantilever, in. Chapter 9

ℓ = span length of flexural member measured center-to-center of joints. Chapter 10

ℓ = span length of member. Chapter 11

ℓ = clear span, in. Chapter 16

ℓ = load bearing length of anchor for shear, not to exceed $8d_o$, in., Appendix D

= h_{ef} for anchors with a constant stiffness over the full length of the embedded section, such as headed studs or post-installed anchors with one tubular shell over the full length of the embedment depth, Appendix D

= $2d_o$ for torque-controlled expansion anchors with a distance sleeve separated from the expansion sleeve, Appendix D

ℓ_a = additional embedment length at support or at point of inflection, in. Chapter 12

ℓ_c = length of a compression member in a frame, measured from center to center of the joints in the frame. Chapter 10

ℓ_c = vertical distance between supports, in. Chapter 14

ℓ_d = development length, in. Chapters 7 and 19

ℓ_d = development length of deformed bars or deformed wire in tension, in. Chapter 12

= ℓ_{db} × applicable modification factors

ℓ_d = development length for a straight bar. Chapter 21

ℓ_d = required development length for straight bar embedded in confined concrete (Section 21.5.4.3). Chapter 21

ℓ_{dc} = development length of deformed bars or deformed wire in compression, in. Chapter 12

ℓ_{dc} = length of bar embedded in confined concrete. Chapter 21

ℓ_{dh} = development length of standard hook in tension, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus radius of bend and one bar diameter), in. Chapter 12

= ℓ_{hb} × applicable modification factors

ℓ_{dh} = development length for a bar with a standard hook a defined in Eq. (21-5), in. Chapter 21

ℓ_{dm} = required development length if bar is not entirely embedded in confined concrete. Chapter 21

ℓ_{hb} = basic development length of standard hook in tension, in. Chapter 12

ℓ_n = clear span for positive moment or shear and average of adjacent clear spans for negative moment, in. Chapter 8

ℓ_n = clear span measured face-to-face of supports, in. Chapter 11 and 21

ℓ_n = length of clear span in long direction of two-way construction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases, in. Chapter 9

ℓ_n = length of clear span in direction that moments are being determined, measured face-to-face of supports, in. Chapter 13

ℓ_n = beam clear span. Chapter 21

ℓ_o = minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, in. Chapter 21

ℓ_t = span of member under load test, in. (The shorter span for two-way slab systems.) Span is the smaller of (a) distance between centers of supports, and (b) clear distance between supports plus thickness, h, of member. In Eq. (20-1), span for a cantilever shall be taken as twice the distance from support to cantilever end, in. Chapter 20

ℓ_u = unsupported length of compression member, in. Chapter 10

ℓ_v = length of shearhead arm from centroid of concentrated load or reaction, in. Chapter 11

ℓ_w = horizontal length of wall, in. Chapter 11 and 14

ℓ_w = length of entire wall or a segment of wall considered in direction of shear force, in. Chapter 21

ℓ_x = length of prestressing steel element from jacking end to any point x ft. See Eq. (18-1) and (18-2). Chapter 18

ℓ_1 = length of span in long direction that moments are being determined, measured center-to-center of supports, in. Chapter 13

ℓ_2 = length of span transverse to ℓ_1, measured center-to-center of supports, in. See also 13.6.2.3 and 13.6.2.4. Chapter 13

L = live loads or related internal moments and forces. Chapters 9, 18, and 20

L = live loads, or related internal moments and forces. Chapter 21

L = live loads due to intended use and occupancy, including loads due to movable objects and movable partitions and loads temporarily supported by the structure during maintenance. L includes any permissible reduction. If resistance to impact loads is taken into account in design, such effects shall be included with the live load L. Appendix C

L_r = roof live load, or related internal moments and forces. Chapter 9

L_r = roof live loads. Appendix C

L_T = length of a rectangular tank (inside dimension) parallel to the design earthquake direction being evaluated, ft. Chapter 21

M = maximum unfactored moment due to service loads, including P∆ effects, in.-lb. Chapter 14

M = design moment, in.-lb. Appendix I

M_a = maximum moment in member at stage deflection is computed, in,-lb. Chapter 9 and 14

M_c = factored moment to be used for design of compression member, in.-lb. Chapter 10

M_c = moment at the face of the joint, corresponding to the nominal flexural strength of the column framing into that joint, calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength, in.-lb. See 21.4.2.2. Chapter 21

M_{cr} = cracking moment, in.-lb. See 9.5.2.3. Chapter 9

M_{cr} = moment causing flexural cracking at section due to externally applied loads, in.-lb. See 11.4.2.1. Chapter 11

M_{cr} = moment causing flexural cracking due to applied lateral and vertical loads, in.-lb. Chapter 14

M_{ct} = total moment including dead load to cause cracking at extreme fiber in tension, in.-lb. Chapter 11

M_d = service dead load moment, in.-lb. Chapter 9

M_g = moment at the face of the joint, corresponding to the nominal flexural strength of the girder including slab where in tension, framing into that joint, in.-lb. See 21.4.2.2. Chapter 21

M_ℓ = service live load moment. Chapter 9

M_m = modified moment, in.-lb. Chapter 11

M_{max} = maximum factored moment at section due to externally applied loads, in.-lb. Chapter 11

M_n = nominal moment strength at section, in.-lb. Chapter 14 and 21

M_n = nominal moment strength. Chapters 9, 11, and 18

M_n = nominal moment strength at section, in.-lb. Chapter 12

= $A_s f_y(d - a/2)$

$M_{n\ell}$ = nominal beam moment, left. Chapter 21

M_{nr} = nominal beam moment, right. Chapter 21

M_{ns} = unmagnified nonsway moment at each end of each column. Chapter 10

M_o = total factored static moment, in.-lb. Chapter 13

M_p = required plastic moment strength of shearhead cross section, in.-lb. Chapter 11

M_{pr} = probable flexural moment strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least $1.25f_y$ and a strength reduction factor ϕ of 1.0, in.-lb. Chapter 21

M_s = moment due to loads causing appreciable way, in.-lb. Chapter 10

M_s = portion of lab moment balanced by support moment, in.-lb. Chapter 21

M_{sa} = maximum unfactored applied moment due to service loads, not including P∆ effects, in.-lb. Chapter 14

M_u = required moment strength. Chapter 9

M_u = factored moment at section, in.-lb. Chapters 10, 11, 13, and 21

M_u = factored moment at section including P∆ effects, in.-lb. Chapter 14

M_{ua} = moment at the midheight section of the wall due to factored lateral and eccentric vertical loads, in.-lb. Chapter 14

M_v = moment resistance contributed by shearhead reinforcement, in.-lb. Chapter 11

M_1 = smaller factored end moment on a compression member, positive if member is bent in single curvature, negative if bent in double curvature, in.-lb. Chapter 10

M_{1ns} = factored end moment on a compression member at the end at which M_1 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, in.-lb. Chapter 10

M_{1s} = factored end moment on compression member at the end at which M_1 acts, due to loads that cause appreciable sideway, calculated using a first-order elastic frame analysis, in.-lb. Chapter 10

M_2 = larger factored end moment on compression member, always positive, in.-lb. Chapter 10

$M_{2,min}$ = minimum value of M_2, in.-lb. Chapter 10

M_{2ns} = factored end moment on compression member at the end at which M_2 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, in.-lb. Chapter 10

M_{2s} = factored end moment on compression member at the end at which M_2 acts, due to loads that cause appreciable sideway, calculated using a first-order elastic frame analysis, in.-lb. Chapter 10

N = number of consecutive strength tests. Chapter 5

n = number of bars or wires being spliced or developed along the plane of splitting. Chapter 12

n = modular ratio of elasticity, but not less than 6. Chapter 14

= E_s/E_c

n = number of monostrand anchorage devices in a group. Chapter 18

n = modular ratio of elasticity. Appendix I

= E_s/E_c

n = number of anchors in a group, Appendix D

n_1, n_2 = number of tests in each test record respectively. Chapter 5

N = design axial load normal to cross section occurring simultaneously with V; to be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage. Appendix I

N_b = basic concrete breakout strength in tension of a single anchor in cracked concrete, as defined in D.5.2.2, lb, Appendix D

N_c = tensile force in concrete due to unfactored dead load plus live load (D + L), lb. Chapter 18

N_{cb} = nominal concrete breakout strength in tension of a single anchor, as defined in D.5.2.1, lb, Appendix D

N_{cbg} = nominal concrete breakout strength in tension of a group of anchors, as defined in D.5.2.1, lb, Appendix D

N_i = resolution of V_i into axial tension force. Chapter 11

N_n = nominal strength in tension, lb, Appendix D

N_p = pullout strength in tension of a single anchor in cracked concrete, as defined in D.5.3.4 or D.5.3.5, lb, Appendix D

N_{pn} = nominal pullout strength in tension of a single anchor, as defined in D.5.3.1, lb, Appendix D

N_s = nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, as defined in D.5.1.1 or D.5.1.2, lb, Appendix D

N_{sb} = side-face blowout strength of a single anchor, lb, Appendix D

N_{sbg} = side-face blowout strength of a group of anchors, lb, Appendix D

N_u = factored axial load normal to cross section occurring simultaneously with V_u or T_u; to be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage, lb. Chapter 11

N_u = factored tensile load, lb, Appendix D

N_{uc} = factored tensile force applied at top of bracket or corbel acting simultaneously with V_u, to be taken as positive for tension, lb. Chapter 11

p_{cp} = outside perimeter of the concrete cross section, in. See 11.6.1. Chapter 11

p_h = perimeter of centerline of outermost closed transverse torsional reinforcement, in. Chapter 11

P = design axial loads. Appendix I

P = loads, forces, and effects due to ponding. Appendix C

P_c = critical load, lb. See Eq. (10-13). Chapter 10

P_n = nominal axial load strength at given eccentricity, lb. Chapters 9 and 10

P_n = nominal axial load strength. Appendix I

P_{ni} = nominal axial load strength at given eccentricity along both axes. Chapter 10

P_{nw} = nominal axial load strength of wall designed by the empirical method, lb. (see 14.5) Chapter 14

P_{nx} = nominal axial load strength at given eccentricity along x-axis. Chapter 10

P_{ny} = nominal axial load strength at given eccentricity along y-axis. Chapter 10

P_o = nominal axial load strength at zero eccentricity, lb. Chapters 10 and 21

P_s = unfactored axial load at the design (midheight) section including effects of self-weight, lb. Chapter 14

P_s = prestressing tendon force at jacking end, lb. Chapter 18

P_{su} = factored prestressing force at the anchorage device, lb. Chapter 18

P_u = required axial load strength. Chapters 9 and 14

P_u = factored axial load at given eccentricity ϕP_n, lb. Chapter 10

P_u = factored axial load, lb. Chapter 14

P_u = factored design axial load. Chapter 21

P_u = factored unit design pressure on dome shell, lb/ft^2. Appendix G

P_x = prestressing tendon force at any point x, lb. Chapter 18

q = shear flow. Chapter 11

q_s = soil reaction due to factored loading. Chapter 15

Q = stability index for a story. See 10.11.4. Chapter 10

r = radius of gyration of cross section of a compression member, in. Chapter 10

r_d = inside radius of dome, ft. Appendix G

r_i = averaged maximum radius of curvature over a dome imperfection area with a diameter of $2.5\sqrt{r_d h_d}$, ft. Appendix G

R = rain load, or related internal moments and forces. Chapter 9

R = rain loads, except ponding. Appendix C

R_w = numerical coefficient representing the combined effect of the structure's ductility, energy-dissipating capacity, and structural. Chapter 21

s = standard deviation, psi. Chapter 5

s = statistical average standard deviation where two test records are used to estimate the standard deviation. Chapter 5

s = center-to-center spacing of the deformed bars, in. Chapter 10

s = spacing of shear or torsion reinforcement measured in a direction parallel to longitudinal reinforcement, in. Chapter 11

s = maximum center-to-center spacing of transverse reinforcement within ℓ_d center-to-center, in. Chapter 12

s = spacing of ties measured along the longitudinal axis of the member, in. Chapter 17

s = center-to-center spacing of flexural tension steel near the extreme tension face, in. Where there is only one bar or tendon near the extreme tension face, s is the width of extreme tension face. Chapter 18

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, in. Chapter 21

s = spacing of shear reinforcement in direction parallel to longitudinal reinforcement, in. Appendix I

s = anchor center-to-center spacing, in., Appendix D

s_o = maximum spacing of transverse reinforcement, in. Chapter 21

s_o = spacing of the outer anchors along the edge in a group, in., Appendix D

s_w = spacing of wire to be developed or spliced, in. Chapter 12

s_x = longitudinal spacing of transverse reinforcement within the length ℓ_o, in. Chapter 21

s_1, s_2 = standard deviations calculated from two test records, 1 and 2, respectively. Chapter 5

s_1 = spacing of vertical reinforcement in wall, in. Chapter 11

s_2 = spacing of shear or torsion reinforcement measured in a direction perpendicular to longitudinal reinforcement—or spacing of horizontal reinforcement in wall, in. Chapter 11

S_d = environmental durability factor. Chapters 9 and 21

S = snow load, or related internal moments and forces. Chapters 9 and 21

S = snow loads. Appendix C

S_e = moment, shear, or axial force at connection corresponding with development of probable strength at intended yield locations, based on the governing mechanism of inelastic lateral deformation, considering both gravity and earthquake load effects. Chapter 21

S_n = nominal, flexural, shear, or axial strength of the connection. Chapter 21

S_y = yield strength of connection, based on f_y, for moment, shear, or axial force. Chapter 21

t = thickness of a wall of a hollow section, in. Chapter 11

t = thickness of washer or plate, in., Appendix D

T = cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete. Chapter 9

T = torsional moment on a member. Chapter 11

T = self-straining forces and effects arising from contraction or expansion resulting from temperature changes, shrinkage, moisture changes, creep in component materials, movement due to differential settlement, or combinations thereof. Appendix C

T_{cr} = torque or torsion on a member causing first crack. Chapter 11

T_n = nominal torsional moment strength, in.-lb. Chapter 11

T_u = factored torsional moment at section, in.-lb. Chapter 11

U = service load bond stress, psi. Chapter 12

U = required strength to resist factored loads or related internal moments and forces. Chapters 9 and 21

U = factored concentric load on footing. Chapter 15

v = design shear stress. Appendix I

v_c = shear stress provided by the concrete at a section, psi. Chapter 11

v_c = permissible shear stress carried by concrete, psi. Appendix I

v_h = permissible horizontal shear stress, psi. Appendix I

v_n = nominal shear stress, psi. See 11.12.6.2, Chapters 11 and 21

v_u = factored shear stress. Chapter 11

V = shear required to cause a flexural crack at the section in question. Chapter 11

V = service load shear. Chapter 12

V = design shear force at section. Appendix I

V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete, as defined in D.6.2.2 or D.6.2.3, lb, Appendix D

V_c = nominal shear strength provided by concrete. Chapters 8, 9, 11, and 21

V_c = nominal shear strength provided by concrete. See 11.12.2.1. Chapter 13

V_{cb} = nominal concrete breakout strength in shear of a single anchor, as defined in D.6.2.1, lb, Appendix D

V_{cbg} = nominal concrete breakout strength in shear of a group of anchors, as defined in D.6.2.1, lb, Appendix D

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, lb. Chapter 11

V_{cp} = nominal concrete pryout strength, as defined in D.6.3, lb, Appendix D

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web, lb. Chapter 11

V_d = shear force at section due to unfactored dead load, lb. Chapter 11

V_e = design shear force determined from 21.3.4.1 or 21.4.5.1, lb. Chapter 21

V_i = one of the shear forces V_1 to V_4. Chapter 11

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}, lb. Chapter 11

V_n = nominal shear strength. Chapters 9 and 11

V_n = nominal shear strength, lb. Chapters 11, 12, 21, and Appendix D

V_{nh} = nominal horizontal shear strength, lb. Chapter 17

V_p = vertical component of effective prestress force at section, lb. Chapter 11

V_s = nominal shear strength provided by shear reinforcement, lb. Chapter 9 and 11

V_s = nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, as defined in D.6.1.1 or D.6.1.2, lb, Appendix D

V_u = required shear strength. Chapter 9

V_u = factored horizontal shear in a story, lb. Chapter 10

V_u = factored shear force at section, lb. Chapters 9, 11, 12, 13, 17, and 21

V_u = factored shear load, lb, Appendix D

V_1, V_2, V_3, V_4 = resolution of shear flow into shear forces on sides of tube or space truss. Chapter 11

w = crack width, in. Chapter 10

w = service load per unit length or per unit area. Appendix I

w_c = weight of concrete, lb/ft.^{3} Chapters 8 and 9

w_d = factored dead load per unit area. Chapter 13

w_D = dead load per unit length or per unit area. Chapter 21 and Appendix I

w_ℓ = factored live load per unit area. Chapter 13

w_L = live load per unit length or per unit area. Chapter 21 and Appendix I

w_u = factored load per unit length of beam or per unit area of slab. Chapter 8

w_u = factored load per unit area. Chapter 13

w_u = factored load per unit length or per unit area. Appendix I

W = wind load, or related internal moments and forces. Chapter 9

W = wind load. Appendix C

x = distance from section being investigated to the support. Chapter 11

x = distance between adjacent spliced bars. Chapter 12

x = shorter overall dimension of rectangular part of cross section, in. Chapter 13

X_i = individual strength tests as defined in 5.5.1.4. Chapter 5

$\overline{X}$ = average of n strength test results. Chapter 5

y = longer overall dimension of rectangular part of cross section, in. Chapter 13

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, in. Chapters 9 and 11

α (alpha) = ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of the beam. Chapters 9 and 13

$= \dfrac{E_{cb}I_b}{E_{cs}I_s}$

α = angle between inclined stirrups and longitudinal axis of member. Chapter 11 and Appendix I

α = reinforcement location factor. See 12.2.4. Chapter 12

α = total angular change of prestressing tendon profile in radians from tendon jacking end to any point x. Chapter 18

α = angle between the diagonal reinforcement and the longitudinal axis of a diagonally reinforced coupling beam. Chapter 21

α_c = coefficient defining the relative contribution of concrete strength to wall strength. See Eq. (21-7). Chapter 21

α_f = angle between shear-friction reinforcement and shear plane. Chapter 11

α_m = average value of α for all beams on edges of a panel. Chapter 9

α_s = constant used to compute V_c in slabs and footings. Chapter 11

α_v = ratio of flexural stiffness of shearhead arm to that of surrounding composite slab section. See 11.12.4.5. Chapter 11

α_1 = α in direction of ℓ_1. Chapter 13

α_2 = α in direction of ℓ_2. Chapter 13

β **(beta)** = ratio of clear spans in long to short direction of two-way slabs. Chapter 9

β = ratio of distances to neutral axis from the extreme tension fiber and from the centroid of the main reinforcement. Chapter 10

β = coating factor. See 12.2.4. Chapter 12

β = ratio of long side to short side of footing. Chapter 15

β_b = ratio of area of reinforcement cut off to total area of tension reinforcement at section. Chapter 12

β_c = ratio of long side to short side of concentrated load or reaction area. Chapter 11 and Appendix I

β_d = (a) for non-sway frames, β_d is the ratio of the maximum factored axial dead load to the total factored axial load;

(b) for sway frames, except as required in (c), β_d is the ratio of the maximum factored sustained shear within a story to the total factored shear in that story; and

(c) for stability checks of sway frames carried out in accordance with 10.13.6, β_d is the ratio of the maximum factored sustained axial load to the total factored axial load. Chapter 10

β_p = constant used to compute V_c in prestressed slabs. Chapter 11

β_t = ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of beam, center-to-center of supports. Chapter 13

$= \dfrac{E_{cb}C}{2E_{cs}I_s}$

β_1 = factor defined in 10.2.7.3. Chapters 8, 10, and 18

γ **(gamma)** = reinforcement size factor. See 12.2.4. Chapter 12

γ = Combined Load Factor, See 9.2.6

γ_f = fraction of unbalanced moment transferred by flexure at slab-column connections. See 13.5.3.2. Chapters 11 and 13

γ_f = fraction of M_s assigned to slab effective width. Chapter 21

γ_p = factor for type of prestressing tendon. Chapter 18

= 0.55 for f_{py}/f_{pu} not less than 0.80

= 0.40 for f_{py}/f_{pu} not less than 0.85

= 0.28 for f_{py}/f_{pu} not less than 0.90

γ_v = fraction of unbalanced moment transferred by eccentricity of shear at slab-column connections. See 11.12.6.1. Chapters 11 and 13

$= 1 - \gamma_f$

δ_b = moment magnification factor for frames braced against side-sway. Chapter 10

δ_{ns} = moment magnification factor for frames braced against sidesway, to reflect effects of member curvature between ends of compression member. Chapter 10

δ_s = moment magnification factor for frames not braced against sidesway, to reflect lateral drift resulting from lateral and gravity loads. Chapter 10

δ_u = design displacement, in. Chapter 21

Δf_p **(delta)** = difference between f_{ps} and prestressing tendon stress at ultimate at section being considered. Chapter 11

Δf_{ps} = stress in prestressing steel at service loads less decompression stress, ksi. Chapter 18.

Δ_{max} = measured maximum deflection in. See Eq. (20-1). Chapter 20

Δ_{rmax} = measured residual deflection, in. See Eq. (20-2) and (20-3). Chapter 20

Δ_{fmax} = maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test, in. See Eq. (20-3). Chapter 20

Δ_o = relative lateral deflection between the top and bottom of a story due to V_u computed using a first-order elastic frame analysis and stiffness values satisfying 10.11.1, in. Chapter 10

Δ_s = maximum deflection at or near midheight due to service loads, in. Chapter 14

Δ_u = deflection at midheight of wall due to factored loads, in. Chapter 14

ε_s **(epsilon)** = strain in reinforcement corresponding to calculated stress. Chapter 10

ε_t = net tensile strain in extreme tension steel at nominal strength. Chapters 9 and 10

ε_y = yield strain of reinforcement. Chapter 10

η **(eta)** = number of identical arms of shearhead. Chapter 11

θ **(theta)** = angle of compression diagonals in truss analogy for torsion. Chapter 11

λ **(lambda)** = multiplier for additional long-term deflection as defined in 9.5.2.5. Chapter 9

λ = correction factor related to unit weight of concrete. Chapters 11, 17, and 18

λ = lightweight aggregate concrete factor. See 12.2.4. Chapter 12

μ **(mu)** = coefficient of friction. See 11.7.4.3. Chapter 11

μ = curvature friction coefficient. Chapter 18

ξ **(xi)** = time-dependent factor for sustained load. See 9.5.2.5. Chapter 9

ρ **(rho)** = ratio of nonprestressed tension reinforcement. Chapters 8, 9, 10, 11, 13, 18, and 21

= A_s/bd

ρ = $A_s/(\ell_w d)$ ratio of tension reinforcement. Chapter 14

ρ = ratio of tension reinforcement. Chapter 20

ρ' = ratio of nonprestressed compression reinforcement. Chapter 8

= A_s'/bd

ρ' = reinforcement ratio for nonprestressed compression reinforcement, A_s'/bd. Chapter 9

ρ' = ratio of compression reinforcement. Chapter 18

= A_s'/bd.

ρ_b = reinforcement ratio producing balanced strain conditions. See 10.3.2. Chapters 8, 10, and 13

ρ_b = reinforcement ratio producing balanced strain conditions. See B10.3.3. Chapter 9 and 14

ρ_g = ratio of total reinforcement area to cross-sectional area of column. Chapter 21

ρ_h = ratio of horizontal shear reinforcement area to gross concrete area of vertical section. Chapter 11

ρ_n = ratio of vertical shear reinforcement area to gross concrete area of horizontal section. Chapter 11

ρ_n = ratio of area of distributed reinforcement parallel to the plane of A_{cv} to gross concrete area perpendicular to that reinforcement. Chapter 21

ρ_p = ratio of prestressed reinforcement. Chapter 18

= A_{ps}/bd_p

ρ_s = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member. Chapter 10

ρ_s = ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out). Chapter 21

ρ_v = ratio of the tie reinforcement area to area of contact surface. Chapter 17

= $A_v/b_v s$

ρ_v = ratio of area of distributed reinforcement perpendicular to the plane of A_{cv} to gross concrete area A_{cv}. Chapter 21

ρ_w = $A_s/b_w d$. Chapter 11

ρ_w = ratio of tension reinforcement. Appendix I

= $A_s/b_w d$

Σ_o **(sigma)** = perimeter of bar, in. Chapter 12

τ **(tau)** = shear stress. Chapter 11

ϕ **(phi)** = strength reduction factor. See 9.3. Chapters 8, 9, 10, 11, 13, 14, 17, 18, 19, 21, and Appendixes D and G

ϕ = strength reduction factor. See I.2.1. Appendix I

ϕ_K = stiffness reduction factor. See R10.12.3. Chapter 10

ψ **(psi)** = ratio of sum of stiffnesses of compression members to sum of stiffnesses of flexural members at one end of a compression member. Chapter 10

ψ_m = average of ψ-values at two ends of a compression member. Chapter 10

ψ_{min} = smaller of ψ-values at two ends of a compression member. Chapter 10

ψ_1 = modification factor, for strength in tension, to account for anchor groups loaded eccentrically, as defined in D.5.2.4, Appendix D

ψ_2 = modification factor, for strength in tension, to account for edge distances smaller than $1.5h_{ef}$, as defined in D.5.2.5, Appendix D

ψ_3 = modification factor, for strength in tension, to account for cracking, as defined in D.5.2.6 and D.5.2.7, Appendix D

ψ_4 = modification factor, for pullout strength, to account for cracking, as defined in D.5.3.1 and D.5.3.6, Appendix D

ψ_5 = modification factor, for strength in shear, to account for anchor groups loaded eccentrically, as defined in D.6.2.5, Appendix D

ψ_6 = modification factor, for strength in shear, to account for edge distances smaller than $1.5c_1$, as defined in D.6.2.6, Appendix D

ψ_7 = modification factor, for strength in shear, to account for cracking, as defined in D.6.2.7, Appendix D

ω (omega) = $\rho f_y / f'_c$. Chapter 18

ω′ = $\rho' f_y / f'_c$. Chapter 18

ω_p = $\rho_p f_{ps} / f'_c$. Chapter 18

ω_w, ω_{pw}, ω'_w = reinforcement indices for flanged sections computed as for ω, ω_p, and ω′ except that b shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only. Chapter 18

Notes

APPENDIX F — METAL REINFORCEMENT INFORMATION

As an aid to users of the ACI 350 Code, information on sizes, areas, and weights of various steel reinforcement is presented.

ASTM STANDARD REINFORCING BARS

Bar size, no.	Nominal diameter, in.	Nominal area, in.^{2}	Nominal weight, lb/ft
3	0.375	0.11	0.376
4	0.500	0.20	0.668
5	0.625	0.31	1.043
6	0.750	0.44	1.502
7	0.875	0.60	2.044
8	1.000	0.79	2.670
9	1.128	1.00	3.400
10	1.270	1.27	4.303
11	1.410	1.56	5.313
14	1.693	2.25	7.650
18	2.257	4.00	13.600

ASTM STANDARD PRESTRESSING TENDONS

Type*	Nominal diameter, in.	Nominal area, in.^{2}	Nominal weight, lb/ft
Seven-wire strand (Grade 250)	1/4 (0.250)	0.036	0.122
	5/16 (0.313)	0.058	0.197
	3/8 (0.375)	0.080	0.272
	7/16 (0.438)	0.108	0.367
	1/2 (0.500)	0.144	0.490
	(0.600)	0.216	0.737
Seven-wire strand (Grade 270)	3/8 (0.375)	0.085	0.290
	7/16 (0.438)	0.115	0.390
	1/2 (0.500)	0.153	0.520
	(0.600)	0.217	0.740
Prestressing wire	0.192	0.029	0.098
	0.196	0.030	0.100
	0.250	0.049	0.170
	0.276	0.060	0.200
	0.162	0.021	0.070
	0.177	0.025	0.084
	0.207	0.034	0.110
	0.225	0.040	0.140
	0.235	0.043	0.150
Prestressing bars (plain)	3/4	0.44	1.50
	7/8	0.60	2.04
	1	0.78	2.67
	1-1/8	0.99	3.38
	1-1/4	1.23	4.17
	1-3/8	1.48	5.05
Prestressing bars (deformed)	5/8	0.28	0.98
	3/4	0.42	1.49
	1	0.85	3.01
	1-1/4	1.25	4.39
	1-3/8	1.58	5.56

* Availability of some tendon sizes should be investigated in advance.

ASTM STANDARD WIRE REINFORCEMENT

W & D size		Nominal diameter, in.	Nominal area, $in.^2$	Nominal weight, lb/ft	Area, $in.^2$/ft of width for various spacings						
					Center-to-center spacing, in.						
Plain	Deformed				2	3	4	6	8	10	12
W31	D31	0.628	0.310	1.054	1.86	1.24	0.93	0.62	0.465	0.372	0.31
W30	D30	0.618	0.300	1.020	1.80	1.20	0.90	0.60	0.45	0.366	0.30
W28	D28	0.597	0.280	0.952	1.68	1.12	0.84	0.56	0.42	0.336	0.28
W26	D26	0.575	0.260	0.934	1.56	1.04	0.78	0.52	0.39	0.312	0.26
W24	D24	0.553	0.240	0.816	1.44	0.96	0.72	0.48	0.36	0.288	0.24
W22	D22	0.529	0.220	0.748	1.32	0.88	0.66	0.44	0.33	0.264	0.22
W20	D20	0.504	0.200	0.680	1.20	0.80	0.60	0.40	0.30	0.24	0.20
W18	D18	0.478	0.180	0.612	1.08	0.72	0.54	0.36	0.27	0.216	0.18
W16	D16	0.451	0.160	0.544	0.96	0.64	0.48	0.32	0.24	0.192	0.16
W14	D14	0.422	0.140	0.476	0.84	0.56	0.42	0.28	0.21	0.168	0.14
W12	D12	0.390	0.120	0.408	0.72	0.48	0.36	0.24	0.18	0.144	0.12
W11	D11	0.374	0.110	0.374	0.66	0.44	0.33	0.22	0.165	0.132	0.11
W10.5		0.366	0.105	0.357	0.63	0.42	0.315	0.21	0.157	0.126	0.105
W10	D10	0.356	0.100	0.340	0.60	0.40	0.30	0.20	0.15	0.12	0.10
W9.5		0.348	0.095	0.323	0.57	0.38	0.285	0.19	0.142	0.114	0.095
W9	D9	0.338	0.090	0.306	0.54	0.36	0.27	0.18	0.135	0.108	0.09
W8.5		0.329	0.085	0.289	0.51	0.34	0.255	0.17	0.127	0.102	0.085
W8	D8	0.319	0.080	0.272	0.48	0.32	0.24	0.16	0.12	0.096	0.08
W7.5		0.309	0.075	0.255	0.45	0.30	0.225	0.15	0.112	0.09	0.075
W7	D7	0.298	0.070	0.238	0.42	0.28	0.21	0.14	0.105	0.084	0.07
W6.5		0.288	0.065	0.221	0.39	0.26	0.195	0.13	0.097	0.078	0.065
W6	D6	0.276	0.060	0.204	0.36	0.24	0.18	0.12	0.09	0.072	0.06
W5.5		0.264	0.055	0.187	0.33	0.22	0.165	0.11	0.082	0.066	0.055
W5	D5	0.252	0.050	0.170	0.30	0.20	0.15	0.10	0.075	0.06	0.05
W4.5		0.240	0.045	0.153	0.27	0.18	0.135	0.09	0.067	0.054	0.045
W4	D4	0.225	0.040	0.136	0.24	0.16	0.12	0.08	0.06	0.048	0.04
W3.5		0.211	0.035	0.119	0.21	0.14	0.105	0.07	0.052	0.042	0.035
W3		0.195	0.030	0.102	0.18	0.12	0.09	0.06	0.045	0.036	0.03
W2.9		0.192	0.029	0.098	0.174	0.116	0.087	0.058	0.043	0.035	0.029
W2.5		0.178	0.025	0.085	0.15	0.10	0.075	0.05	0.037	0.03	0.025
W2		0.159	0.020	0.068	0.12	0.08	0.06	0.04	0.03	0.024	0.02
W1.4		0.135	0.014	0.049	0.084	0.056	0.042	0.028	0.021	0.017	0.014

APPENDIX G — CIRCULAR WIRE AND STRAND WRAPPED PRESTRESSED CONCRETE ENVIRONMENTAL STRUCTURES

CODE

COMMENTARY

G.0 — Notation

E_c = modulus of elasticity of concrete under short-term load, psi

f'_c = specified compressive strength of concrete, psi

h_d = dome shell thickness, in.

L = live load

P_u = factored unit design pressure on dome shell, lb/ft^2

r_d = inside radius of dome, ft

r_I = averaged maximum radius of curvature over a dome imperfection area with a diameter of $2.5\sqrt{r_d h_d}$, ft

B_c = buckling reduction factor for creep, nonlinearity, and cracking of concrete

B_i = buckling reduction factor for geometrical imperfections

ϕ = strength reduction factor, see 9.3

G.1 — Scope

The requirements in this appendix are intended to supplement the general requirements for reinforced concrete and prestressed concrete design and construction given in ACI 318, ACI 350, and ACI 301. Design and construction requirements address these elements or components of circular wrapped prestressed concrete tanks:

(a) Floors

- Reinforced concrete

(b) Floor-wall connections

- Hinged
- Fixed
- Partially fixed
- Unrestrained
- Changing restraint

(c) Walls

- Cast-in-place concrete walls
- Shotcrete walls with steel diaphragms

RG.1 — Scope

This appendix gives guidance to individuals charged with design and construction of circular prestressed concrete structures by requiring practices that have resulted in successful structures.

Pretensioned high-strength strands are sometimes used vertically in precast panels.

CODE

• Precast concrete wall systems with steel diaphragms

(d) Wall-roof connections

• Hinged

• Fixed

• Partially fixed

• Unrestrained

(e) Roofs

• Concrete dome roofs with prestressed dome ring including cast-in-place concrete, shotcrete, and precast concrete.

• Flat concrete roofs

(f) Wall and dome ring prestressing systems

• Circumferential using wrapped wire or strand systems

• Vertical using single or multiple high-strength strands, bars, or wires

Internal tendon tanks and structures are covered in Chapter 18.

G.2 — Design

G.2.1 — Strength and serviceability

G.2.1.1 — Strength

Structures and structural members shall be proportioned to have strengths at all sections equal to or exceeding the minimum required strengths calculated for the factored loads and forces in such combinations as required in Chapters 9 and 18. The environmental durability factor is not applicable to design of prestressed reinforcement.

G.2.1.2 — Corrosion protection of prestressed reinforcement

G.2.1.2.1 — Circumferential prestressed wire or strand placed on the exterior surface of a core wall shall be protected by at least 1 in. of shotcrete cover. Each wire or strand shall be fully encased in shotcrete.

G.2.1.2.2 — Vertical prestressed reinforcement in wrapped tanks shall be post-tensioned in ducts and protected by portland cement or epoxy grout.

G.2.2 — Wall design

G.2.2.1 — Design methods

The design of the wall shall be based on linear cylindrical shell analysis. The design shall consider the effects of prestressing, internal loads, backfill, and other external loads. The design shall also provide for

COMMENTARY

RG.2 — Design

RG.2.1 — Strength and serviceability

RG.2.1.1 — Strength

With prestressed design for environmental engineering concrete structures, durability is provided by maintaining a minimum compression at the element centroid and by limiting maximum tensile stresses. The durability factor in Chapter 9 is used to limit stresses in conventional reinforcement and is not applicable to design of prestressed reinforcement.

RG.2.2 — Wall design

RG.2.2.1 — Design methods

Coefficients, formulas, and other aids for determining bending moments, and circumferential and shear forces based on linear shell analyses are given in References G.1 through G.6.

CODE

(a) and (b):

(a) The effects of shrinkage, elastic shortening, and creep of the concrete and shotcrete, relaxation of prestressed reinforcement, and temperature and moisture gradients;

(b) The joint movements and forces resulting from restraint of deflections, rotations, and deformations that are induced by prestressing forces, design loads, and volume changes.

G.2.2.2 — Wall types

G.2.2.2.1 — Wall types covered in this chapter are described in (a) through (d):

(a) Cast-in-place concrete, prestressed circumferentially with either high-strength steel wire or strand, wound on the external surface of the core wall, and prestressed vertically with grouted steel tendons;

(b) Shotcrete with full height, vertically-fluted steel diaphragm, prestressed circumferentially by wrapping with high-strength steel wire or strand;

(c) Precast concrete vertical panels curved to tank radius having a full-height, vertically-fluted steel diaphragm on the exterior face, and vertical nonprestressed steel reinforcement near the interior face;

(d) Cast-in-place concrete prestressed circumferentially with either high-strength wire or strand, wound on the external surface of the core wall and having a full height, vertically-fluted steel diaphragm near the exterior face, and vertical nonprestressed steel reinforcement near the interior face.

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RG.2.2.2 — Wall types

RG.2.2.2.1 — The vast majority of wire- and strand-wrapped tanks use one of these four basic wall types. Additional details of each type include:

(a) Cast-in-place concrete wall tanks have vertical nonprestressed steel reinforcement near each face for strength and crack control. Circumferential prestressing is covered with shotcrete to provide bond, mechanical protection, and corrosion protection;

(b) Shotcrete walls use a steel diaphragm near one face of the wall, and nonprestressed steel reinforcement is provided near the other face as vertical reinforcement. If needed, additional nonprestressed steel can be provided near the face with the diaphragm. Adjacent sections of the diaphragm are joined and sealed to form an impervious membrane. The diaphragm is coated first with shotcrete, after which the composite wall is prestressed circumferentially by wrapping with high-strength steel wire or strand. Circumferential prestressing is covered with shotcrete to provide bond, mechanical protection, and corrosion protection;

(c) In precast walls, the panels are connected to each other by sheet steel, with the joints between the panels being filled with high-strength mortar or shotcrete. Adjacent sections of diaphragm, both within the panels and between panels, are joined and sealed to form an impervious membrane. The diaphragm is coated first with shotcrete, after which the composite wall is prestressed circumferentially by winding with high-strength steel wire or strand. Circumferential prestressing is covered with shotcrete to provide bond, mechanical protection, and corrosion protection;

(d) In cast-in-place concrete walls using a steel diaphragm, adjacent sections of the diaphragm are joined and sealed to form an impervious membrane. The exterior surface of the diaphragm exposed after the wall is cast is coated first with shotcrete, after which the composite wall is prestressed circumferentially by wrapping with high-strength steel wire or strand. Circumferential prestressing is covered with shotcrete to provide bond, mechanical protection, and corrosion protection.

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G.2.2.2.2 — Liquid-tightness and crack control for liquid-containment structures

Liquid-tightness and crack control shall be provided by circumferentially prestressing a shotcrete, precast, or cast-in-place concrete core wall containing a liquid-tight steel diaphragm, or by a cast-in-place wall with both circumferential and vertical prestressed reinforcement.

Circumferential (horizontal) construction joints shall not be permitted in the core wall between the base and the top; the wall base joint, top joint, and vertical joints shall be the only construction joints permitted.

All vertical construction joints in cast-in-place concrete core walls without a metal diaphragm shall contain waterstops and dowels to prevent leakage and radial displacement of adjacent wall sections.

G.2.2.3 — Wall proportions

Minimum core wall thickness shall be (a), (b), and (c):

(a) 3-1/2 in. for shotcrete walls with a steel diaphragm;

(b) 4 in. for precast concrete walls with a steel diaphragm;

(c) 7 in. for cast-in-place concrete walls.

G.2.2.4 — Circumferential prestressing

G.2.2.4.1 — The total long-term prestressing losses caused by shrinkage, creep, and relaxation in the prestressed reinforcement of liquid-containing walls shall not be assumed less than 25,000 psi.

G.2.2.4.2 — Spacing of prestressed reinforcement: minimum clear spacing between wires or strands shall be 1.5 times the wire or strand diameter, or 1/4 in. for wires and 3/8 in. for strands, whichever is greater. Maximum center-to-center spacing shall be 2 in. for wires and 6 in. for strands, except as provided for wall openings in G.2.2.7.

G.2.2.5 — Wall edge restraints and other secondary bending

Wall edge restraints, discontinuities in application of prestressing, and environmental conditions result in vertical and circumferential bending that affect performance. Consideration shall be given to conditions (a) through (e):

(a) Edge restraint of elastic deformations due to applied loads at the wall-floor joint and at the wall-roof joint;

(b) Restraint of shrinkage and creep of concrete;

(c) Sequence of application of circumferential prestressing;

COMMENTARY

RG.2.2.2.2 — Liquid tightness and crack control for liquid-containment structures

Considerations with respect to leakage are:

(a) The provision of a full-height, vertically-fluted steel diaphragm having sealed edge joints that extend throughout the area of the wall is a positive means of achieving liquid-tightness;

(b) The use of vertical prestressing in cast-in-place core walls without a diaphragm is a positive means of controlling horizontal cracking, thus providing liquid-tightness;

(c) A dense, well-compacted concrete, free of honeycombing and cold joints, is essential for providing a durable, impermeable concrete.

RG.2.2.3 — Wall proportions

Experience in wrapped prestressed tank design and construction has demonstrated that these are practical production limits.

RG.2.2.4 — Circumferential prestressing

RG.2.2.4.1 — Losses may be larger than 25,000 psi in tanks that are not intended for water storage, or that are expected to remain empty for long periods of time. In such cases, it is recommended to calculate prestress loss due to elastic shortening, creep, shrinkage, and steel relaxation by considering properties of the materials and systems used, the service environment, the load durations, and the stress levels in the concrete and prestressing steel. Refer to References G.5, G.7 through G.13, and ACI 209R[G.14] for guidance in calculating prestress losses.

RG.2.2.5 — Wall edge restraints and other secondary bending

Various joint details have been designed to minimize restraint stresses at the locations mentioned in (a). These include joints that use neoprene rubber pads and other elastomeric materials combined with flexible waterstops to minimize restraint of joint translation and rotation. For large-diameter tanks or those located in areas with high seismic risk, it is generally preferable to use an unrestrained wall joint with a flexible seismic connection.

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(d) Banding of prestressing for penetrations as described in G.2.2.7;

(e) Temperature and moisture differences between tank components or through the wall thickness.

G.2.2.6 — Design of vertical reinforcement

G.2.2.6.1 — Walls in liquid-containing tanks having a steel diaphragm shall be vertically reinforced with non-prestressed reinforcement. The area of non-prestressed reinforcement shall be computed using either (a) or (b).

(a) Reinforcement shall be proportioned to resist the full flexural tensile stress resulting from bending due to edge restraint of deformation from loads, primary prestressing forces, and other effects listed in G.2.2.1 and G.2.2.5. The stress levels in the reinforcement and bar spacing for crack control shall be determined based on the provisions of this code, except that the maximum allowable tensile stress under service loads in the reinforcement shall be limited to 18,000 psi. The area of the steel diaphragm shall be permitted to be part of the required vertical reinforcement when a development length of at least 12 in. is provided;

(b) The bending effects due to thermal and shrinkage differences between floor and wall or roof, and the effects of wall thermal and moisture gradients, shall be permitted to be taken into account in walls with a steel diaphragm by providing a minimum area of vertical reinforcement equal to 0.005 times the wall cross section, with one half of the required area placed near each the inner and outer faces of the wall.

G.2.2.6.2 — Walls in liquid-containing tanks not containing a steel diaphragm shall be vertically prestressed to provide for the bending moments caused by wall edge restraints and secondary bending.

(a) Vertical prestressed reinforcement shall be designed to limit the maximum flexural tensile stress after all prestress losses in walls to $6\sqrt{f_c'}$ under the governing combination of design load, wall edge restraint, secondary bending, and circumferential prestress force. Wall sections shall have bonded reinforcement near the tension face. In all locations subject to tensile stresses, the area of bonded reinforcement shall be at least equal to the total flexural tensile force based upon an uncracked concrete section divided by a maximum stress under service loads in the reinforcement of 18,000 psi;

(b) The minimum average effective final vertical prestress applied to the wall shall be 200 psi;

(c) Spacing of vertical prestressing elements shall not exceed 50 in.

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RG.2.2.6 — Design of vertical reinforcement

RG.2.2.6.1 — Alternative methods for determining the effects of thermal and moisture gradients mentioned in Paragraph (b) based on analytical procedures are given in References G.1, G.4, G.6, G.12, G.15, and ACI 349.[G.16] An analytical method should be used where operating conditions or extremely arid regions produce unusually large thermal or moisture gradients.

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G.2.2.7 — Wall penetrations

For penetrations having a height of 2 ft or less, the band of prestressed wires or strands normally required over the height of a penetration shall be displaced into circumferential bands immediately above and below the penetration. The total prestressing force shall not be reduced as the result of a penetration.

(a) Each band shall provide approximately one-half of the displaced prestressing force;

(b) The wires or strands shall not be located closer than 2 in. to wall penetrations;

(c) The wall thickness shall be adequate to support the increased circumferential compressive force adjacent to the penetration;

(d) Vertical bending resulting from the banding of prestressed reinforcement shall be taken into account in the wall design;

(e) Penetrations greater than 2 ft in height shall be designed to provide adequate local reinforcement for nonuniform prestressing distributions on the wall.

G.2.3 — Roof design

G.2.3.1 — Dome roofs

G.2.3.1.1 — Design method: concrete or shotcrete dome roofs shall be designed on the basis of elastic shell analysis. A circumferentially-prestressed dome ring shall be provided at the base of the dome shell to resist the horizontal component of the dome thrust.

G.2.3.1.2 — Thickness: dome shell thickness is governed either by buckling resistance, by minimum thickness for practical construction, or by corrosion protection of reinforcement.

(a) Minimum dome thickness shall be

$$h_d = 1000 r_d \sqrt{\frac{1.5 P_u}{\phi B_i B_c E_c}} \qquad \text{(G-1)}$$

(b) Values for the terms in Eq. (G-1) for such domes are:

(1) P_u is obtained using minimum load factors given in Chapter 9 for dead and live load

(2) $$\phi = 0.7 \qquad \text{(G-2)}$$

(3) $$B_i = (r_d/r_i)^2 \qquad \text{(G-3)}$$

In the absence of other criteria, the maximum value of r_i shall be permitted to be taken as $1.4r_d$, in which case

$$B_i = 0.5 \qquad \text{(G-4)}$$

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RG.2.2.7 — Wall penetrations

Penetrations may be provided in walls for manholes, piping, openings, construction access, or other requirements. Concrete compressive strength may be augmented by compression reinforcement adequately confined by ties in accordance with this document or by steel edge members around the opening. The wall thickness may be increased locally, adjacent to the penetration, provided the thickness is changed gradually.

RG.2.3 — Roof design

RG.2.3.1 — Dome roofs

RG.2.3.1.1 — See References G.1 to G.4 and G.8 for design aids relating to elastic shell analysis. Reference G.17 provides methods for designing thin concrete domes against buckling.

RG.2.3.1.2 — A method for determining the minimum thickness of a monolithic concrete spherical dome shell to provide adequate buckling resistance is given in Reference G.17. This method is based on the linear theory of dome shell stability with consideration of the effects of creep, imperfections, and experience with existing tank domes having large radius-to-thickness ratios.

The conditions that determine the factors B_i and B_c are discussed in Reference G.16. The values for these factors given in Paragraphs (3) and (4) are recommended for use in Eq. (G-1) when domes are designed for conditions where the live load is 12 lb/ft^2 or more, liquid is stored inside the tank, dome thickness is 3 in. or more, f'_c is 4000 psi or more, normalweight aggregates are used, and dead load is applied (shores removed) not earlier than 7 days after concrete placement, with curing as required in ACI 301.

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(4) $B_c = 0.44 + 0.003\ (L)$

$$\text{for } 12 \text{ lb/ft}^2 < L \leq 30 \text{ lb/ft}^2 \qquad \text{(G-5)}$$

$$B_c = 0.53$$

$$\text{for } L \geq 30 \text{ lb/ft}^2$$

(c) Thickness of precast concrete panel dome shells shall not be less than the thickness obtained using Eq. (G-1) when joints between the panels are equivalent in strength and stiffness to a monolithic shell;

(d) Precast concrete panel domes with joints between panels having lower strength or stiffness than the joint characteristics given in Paragraph (c) shall be permitted to be used if the minimum thickness of the panel is increased above the value given in Eq. (G-1) in accordance with an analysis of the stability of a dome with a reduced stiffness as a result of joint details;

(e) Other dome configurations, such as cast-in-place or precast domes with ribs cast monolithically with a thin shell, shall be permitted to be used if their design is substantiated by analysis. This analysis shall show that they have adequate strength and buckling resistance to support the design live and dead loads with at least the same minimum load factors established in Eq. (G-1);

(f) Stresses and deformations resulting from handling and erection shall be taken into account in the design of precast concrete panel domes. Panels shall be cambered whenever their maximum dead-load deflection prior to their final incorporation as a part of the complete dome is greater than 10 percent of their thickness;

(g) The thickness of domes shall not be less than 3 in. for monolithic concrete and shotcrete, 4 in. for precast concrete, and 2 in. for the outer shell of a ribbed dome.

G.2.3.1.3 — Reinforcement area

For monolithic domes, the minimum ratio of nonprestressed reinforcement area to concrete area shall be 0.0025 in both the circumferential and meridianal directions. In domes with a thickness of 6 in. or less, the reinforcement shall be placed approximately at the mid-depth of the shell, except in edge regions. In edge regions of such domes, and throughout domes more than 6 in. thick, reinforcement shall be placed in two layers, one near each face.

G.2.3.1.4 — Dome ring

(a) The minimum ratio of nonprestressed reinforcement area to concrete area in the dome ring shall be 0.0025 for cast-in-place dome rings;

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RG.2.3.1.3 — Reinforcement area

Minimum reinforcement may have to be increased for unusual temperature or moisture conditions. The edge region of the dome is subject to bending stress due to the prestressing of the dome ring and dome live load. Bending moments should be considered in the design.

RG.2.3.1.4 — Dome ring

Circular prestressing of the dome ring is used to eliminate or control the circumferential tension in the dome ring and the dome edge region. The minimum area of nonprestressed

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(b) The dome ring reinforcement shall have sufficient strength to meet the requirements given in G.2.1.1 for dead- and live-load factors and for strength-reduction factors;

(c) An effective prestressing force, after all losses, shall be provided to counteract at least the tension due to dead load, plus a minimum residual circumferential compressive stress equal to the residual compression at the top of the wall. If prestressing for less than the full live load is used, sufficient prestressing steel shall be maintained at reduced stress, or additional nonprestressed reinforcement shall be added, to obtain the required strength;

(d) Maximum initial prestress in wires and strands shall comply with Chapter 18;

(e) Maximum initial compression stress in dome rings shall comply with Chapter 18.

G.3 — Materials

G.3.1 — Shotcrete

G.3.1.1 — General

Unless otherwise indicated herein, shotcrete shall meet the requirements of ACI 506.2.

G.3.1.2 — Allowable chlorides

Maximum water-soluble chloride ions shall not exceed 0.06 percent by weight of the cement in shotcrete as determined by AASHTO T260 to provide corrosion protection in prestressed concrete.

G.3.1.3 — Proportioning

Shotcrete shall be proportioned in accordance with the following requirements of G.3.1.3.1 and G.3.1.3.2:

G.3.1.3.1 — Wire coat shall consist of one part portland cement and not more than three parts fine aggregate by weight.

G.3.1.3.2 — Body coat shall consist of one part portland cement and not more than four parts fine aggregate by weight.

G.3.1.4 — Compressive strength

Minimum 28-day compressive strength of shotcrete shall be 4500 psi.

G.3.2 — Reinforcement

G.3.2.1 — Nonprestressed reinforcement

G.3.2.1.1 — Nonprestressed reinforcement shall meet the requirements of 3.5.

G.3.2.1.2 — Strand for wall-to-footing earthquake cables shall be galvanized and shall meet the

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reinforcement controls shrinkage- and temperature-induced cracking prior to prestressing.

Additional prestress may be provided to counteract some or all of the live load. Generally, a lower initial compression stress than the maximum allowable stress is used in dome rings to limit edge bending moments in regions of the dome and wall adjacent to the dome ring.

RG.3 — Materials

RG.3.1 — Shotcrete

RG.3.1.1 — General

The provisions of ACI 506R[G.18] provide guidance on application of shotcrete.

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requirements of ASTM A 416, Grade 250 or 270, prior to galvanizing, and ASTM A 586, ASTM A 603, or ASTM A 475 after galvanizing. Zinc coating shall meet the requirements of ASTM A 475, Table 4, Class A, or ASTM A 603, Table 2, Class A. Epoxy-coated strand shall meet the requirements of ASTM A 416, Grade 250 or 270, with a fusion-bonded epoxy coating, grit impregnated on the surface, conforming to ASTM A 822.

G.3.2.1.3 — Sheet steel diaphragm for use in the walls of prestressed concrete tanks shall be vertically ribbed with adjacent and opposing channels. The base of the ribs shall be wider than the throat, thus providing a mechanical keyway anchorage between the inner and outer concrete or shotcrete.

(a) Steel diaphragms shall meet the requirements of ASTM A 1008 and shall have a minimum thickness of 0.017 in.;

(b) When galvanized diaphragm is used, hot-dipped galvanized sheet steel shall comply with ASTM A 525. Weight of zinc coating shall be not less than G90 of Table 1 of ASTM A 525.

Steel diaphragm shall be continuous for the full height of the wall. Horizontal splices shall not be permitted.

G.3.2.2 — Circumferential prestressed reinforcement

G.3.2.2.1 — Circumferential prestressed reinforcement shall be wires or strands complying with the following ASTM designations:

(a) Field die-drawn, wire-wound systems: ASTM A 821 or ASTM A 227;

(b) Other wire-wound systems: ASTM A 821; ASTM A 648; Classes I, II, or III; ASTM A 421; and ASTM A 227;

(c) Strand-wound systems: ASTM A 416.

G.3.2.2.2 — When galvanized wire or strand is used for prestressed reinforcement, the wire or strand shall have a zinc coating of at least 0.85 ounces per square foot of uncoated wire surface, except for wire that is stressed by die drawing. If die drawing is used, the minimum required coating shall be permitted to be reduced to 0.50 ounces per square foot of wire surface after stressing. The coating shall meet the requirements for Table 4, Class A coating specified in ASTM A 586.

G.3.2.2.3 — Mechanical splices for prestressed reinforcement shall be ferrous material and shall develop the specified tensile strength of the reinforcement.

COMMENTARY

RG.3.2.1.3 — Some tanks use galvanized steel diaphragms. Consideration should be given to the effect on bond due to hydrogen caused by the reaction of portland cement and zinc.

RG.3.2.2 — Circumferential prestressed reinforcement

RG.3.2.2.2 — A small percentage of wire-wrapped tanks have been constructed with galvanized wire reinforcement. Uncoated steel is generally used for prestressed reinforcement unless galvanized prestressed reinforcement is specified by the Engineer. The large majority of strand-wrapped tanks have used galvanized prestressed strand.

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G.3.3 — Waterstop, bearing pad, and filler materials

G.3.3.1 — Waterstops

Waterstops shall be polyvinyl chloride or other suitable materials. Material proposed for use on the job site shall be certified by the manufacturer based on laboratory tests, or other tests shall be made that will assure compliance with the specification.

G.3.3.2 — Bearing pads

Bearing pads shall consist of neoprene, natural rubber, polyvinyl chloride, or other materials that have demonstrated acceptable performance under conditions and applications similar to those anticipated.

G.3.3.2.1 — Neoprene bearing pads shall have a minimum ultimate tensile strength of 1500 psi, a minimum elongation of 500 percent (ASTM D 412), and a maximum compressive set of 50 percent (ASTM D 395, Method A), with a hardness of 30 to 60 durometers (ASTM D 2240, Type A Durometer). Neoprene bearing pads shall contain only virgin, crystallization-resistant polychloroprene as the raw polymer, and physical properties shall comply with ASTM D 2000, Line Call-Out M2BC4105A14B14.

G.3.3.2.2 — Natural rubber bearing pads shall contain only virgin natural polyisoprene as the raw polymer, and physical properties shall comply with ASTM D 2000, Line Call-Out M4AA41413.

G.3.3.2.3 — Polyvinyl chloride for bearing pads shall meet the requirements of CRD-C-572.

G.3.3.3 — Sponge filler

Sponge filler shall be closed-cell neoprene or rubber meeting the requirements of ASTM D 1056, Grade 2A1 through Grade 2A4. Minimum grade sponge filler used with cast-in-place concrete walls shall be Grade 2A3.

G.3.4 — Sealer for steel diaphragm

Vertical joints between sheets of diaphragm shall be sealed with polysulfide, epoxy, or a mechanical sealer.

G.3.4.1 — Polysulfide sealant

Polysulfide sealant shall be a two-component elastomeric compound meeting the requirements of ASTM C 920 and shall have permanent characteristics of bond to metal surfaces, flexibility, and resistance to extrusion due to hydrostatic pressure. Air-curing sealants shall not be used. Sealant shall be a type that is appropriate for submerged service when used in fluid storage tanks.

COMMENTARY

RG.3.3 — Waterstop, bearing pad, and filler materials

RG.3.3.1 — Waterstops

Plastic waterstops of polyvinyl chloride meeting the requirements of CRD-C-572 are recommended. Splices should be made in accordance with the manufacturer's recommendations.

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G.3.4.2 — Epoxy sealer

Epoxy sealer shall be suitable for bonding to concrete, shotcrete, and steel, and suitable for sealing the vertical joints between sheets of diaphragm. Epoxy sealer shall conform to the requirements of ASTM C 881, Type III, Grade 1, unless other types are approved by the Engineer, and shall be a 100-percent solids, moisture-insensitive, low-modulus epoxy system. When pumped, maximum viscosity of the epoxy shall not exceed 10 poises at 77 °F.

G.3.4.3 — Mechanical seaming

Mechanical seams shall be double-folded and liquid-tight.

G.4 — Construction procedures

G.4.1 — Concrete

G.4.1.1 — Scope

Procedures for concrete construction shall be as specified in ACI 301, except as modified herein.

G.4.1.2 — Precast concrete core walls

G.4.1.2.1 — Concrete for each precast concrete wall panel shall be placed in one continuous operation without cold joints or construction joints.

G.4.1.2.2 — Precast concrete wall panels shall be erected to the correct vertical and circumferential alignment within the tolerances given in G.4.6.

G.4.1.2.3 — When precast wall panels are cast with a steel diaphragm, the edges of the diaphragm of adjoining wall panels shall be joined to form a liquid-tight barrier. Mating edges shall be sealed with an elastomeric or epoxy sealant.

G.4.1.2.4 — Vertical slots between panels shall be free of dirt or foreign substances. Concrete surfaces in the slots shall be clean and damp prior to filling the slots. Slots shall be filled with cast-in-place concrete, cement-sand mortar, or shotcrete compatible with the details of the joint. The strength of the concrete, mortar, or shotcrete shall be not less than that specified for concrete in the wall panels.

G.4.2 — Shotcrete

G.4.2.1 — Construction procedures

Procedures for shotcrete construction shall be as specified in ACI 506.2, except as modified herein.

G.4.2.2 — Shotcrete core walls

Shotcrete core walls shall be built up of individual layers of shotcrete, 2 in. or less in thickness. Thickness control shall be as required in G.4.2.5.

COMMENTARY

RG.4 — Construction procedures

RG.4.1 — Concrete

RG.4.1.1 — Scope

RG.4.1.2 — Precast concrete core walls

RG.4.1.2.3 — Refer to G.3.4.1 and G.3.4.2 for use of elastomeric and epoxy sealers.

RG.4.2 — Shotcrete

RG.4.2.1 — Construction procedures

The provisions of ACI 506R provide guidance on construction techniques appropriate for application of shotcrete.

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G.4.2.3 — Surface preparation of core wall

Prior to application of prestressed reinforcement, dust, efflorescence, oil, and other foreign material shall be removed, and defects in the core wall shall be filled flush with mortar or shotcrete that is bonded to the core wall. Concrete core walls shall be cleaned by abrasive blasting prior to application of prestressed reinforcement and shotcrete. Core walls shall have a bondable exterior surface.

G.4.2.4 — Shotcrete covercoat

G.4.2.4.1 — Externally applied circumferential prestressed reinforcement shall be protected against corrosion and other damage by a shotcrete covercoat.

G.4.2.4.2 — Each layer of circumferential prestressed wire or strand shall be covered first with a wire coat of cement mortar applied by the pneumatic process as soon as practical after prestressing. The shotcrete shall be wet, but not dripping, and provide a minimum cover over the wire of 1/4 in. The nozzle shall be held at a small upward angle not exceeding 5 degrees and shall be constantly moving, without shaking, and always pointing in a radial direction toward the center of the tank. The nozzle distance from the prestressed reinforcement shall be such that shotcrete does not build up over or cover the front faces of the wires or strands until the spaces between them are filled.

G.4.2.4.3 — The wire coat shall be damp-cured by a constant spray or trickling of water down the wall, except that curing shall be permitted to be interrupted during continuous prestressing operations. Curing compounds shall not be used on surfaces that will receive additional shotcrete.

G.4.2.4.4 — Shotcrete material placed incorrectly shall be removed and replaced.

G.4.2.4.5 — A body coat providing a minimum of 1 in. cover over the outside layer of prestressed reinforcement shall be applied over the last layer of wire coat.

(a) If the body coat is not applied as a part of the wire coat, laitence and loose particles shall be removed from the surface of the wire coat prior to the application of the body coat;

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RG.4.2.4 — Shotcrete covercoat

RG.4.2.4.1 — The shotcrete covercoat generally consists of two or more coats: a wire coat placed on the prestressed reinforcement, and a body coat placed on the wire coat. If the covercoat is placed in one coat, the mixture should be the same as the wire coat.

RG.4.2.4.2 — Nozzle distance and wetness of mixture are equally critical to satisfactory encasement of prestressed reinforcement. If the nozzle is held too far back, the shotcrete will deposit on the face of the wire or strand at the same time that it is building up on the core wall, thereby not filling the space behind them. This condition is readily apparent, and should be corrected immediately by adjusting the nozzle distance and, if necessary, the water content.

RG.4.2.4.3 — Curing compounds applied to intermediate layers of shotcrete may interfere with the bonding of subsequent layers, and thus, their use is prohibited.

RG.4.2.4.4 — After the wire coat is in place, visual inspection can immediately determine whether or not proper encasement has been achieved. Where the reinforcement patterns show on the surface as distinct continuous horizontal ridges, the shotcrete has not been driven behind the reinforcement and voids can be expected. If, however, the surface is substantially flat and shows virtually no pattern, a minimum of voids is likely.

RG.4.2.4.5 — Curing should be started as soon as possible without damaging the shotcrete.

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(b) Thickness control shall be as required by G.4.2.5;

(c) The completed shotcrete coating shall be cured for at least 7 days by methods specified by ACI 506.2.

G.4.2.4.6 — After the bodycoat has cured, the surface shall be checked for “hollow sounding” or “drummy” spots by tapping with a light hammer or similar tool. Such spots indicate a lack of bond between coats and shall be repaired. These areas shall be repaired by removal and replacement with properly bonded shotcrete, or by epoxy injection.

G.4.2.5 — Thickness control of shotcrete core walls and covercoats

G.4.2.5.1 — Positive methods shall be used to establish uniform and correct thickness of shotcrete core walls and covercoats.

G.4.2.5.2 — When screed wires are used for thickness control they shall be spaced a maximum of 36 in. apart circumferentially.

G.4.2.6 — Cold-weather shotcreting

Shotcrete placed when the temperature is below 35 °F and rising, or when the temperature is 40 °F and falling, shall be protected in accordance with ACI 506.2. Shotcrete shall not be placed on frozen surfaces. Shotcrete with a strength lower than specified due to cold weather shall be removed and replaced with sound material.

G.4.3 — Forming

G.4.3.1 — Slipforming

Slipforming shall not be used for construction of walls of circular wrapped prestressed concrete structures used to contain liquids.

G.4.3.2 — Steel diaphragms

G.4.3.2.1 — All vertical joints between diaphragms shall be free of voids and sealed for liquid-tightness. The diaphragm form shall be braced and supported in a manner sufficient to eliminate vibrations that would impair the bond between the diaphragm and the concrete or shotcrete.

G.4.3.2.2 — Diaphragms having an oxide coating that is loose or flaky shall not be used.

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RG.4.2.5 — Thickness control of shotcrete core walls and covercoats

RG.4.2.5.1 — Vertical screed wires are the normal method used to establish uniform and correct thickness of shotcrete. Wires should be installed under tension, defining the outside surface of the shotcrete from top to bottom. Wires generally are 18- to 20-gauge high-tensile-strength steel wire. Other methods may be used that will provide positive control of the thickness.

RG.4.3 — Forming

RG.4.3.1 — Slipforming

Slipforming is not recommended for walls of structures used to contain liquids because of the high potential for leakage due to cold joints and honeycombs.

RG.4.3.2.2 — At the time that concrete or shotcrete is placed over the diaphragm, the steel surface may have a light coating of nonflaky oxide (rust).

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G.4.4 — Nonprestressed reinforcement—concrete and shotcrete cover

G.4.4.1 — The minimum concrete and shotcrete cover over the steel diaphragm and nonprestressed reinforcement shall be 1 in.

G.4.4.2 — A minimum cover of 1/2 in. of shotcrete shall be placed over the steel diaphragm before prestressed reinforcement is placed on the core wall.

G.4.5 — Prestressed reinforcement—wire or strand winding

G.4.5.1 — Qualifications

The stressing system used shall be capable of consistently producing the specified stress at every point around the wall within a tolerance of ±7 percent of the specified initial stress in each wire or strand (See G.4.5.5.1).

G.4.5.2 — Anchoring of wire or strand

Each coil of prestressed wire or strand shall be anchored to adjacent wire or strand, or to the wall surface, at sufficiently close intervals to minimize the loss of prestress in case of a break during wrapping.

Anchoring clamps shall be removed wherever cover over the clamp in the completed structure would be less than 1 in.

G.4.5.3 — Splicing of wire or strand

Ends of individual coils shall be joined by splicing devices as specified in G.3.2.2.3.

G.4.5.4 — Concrete or shotcrete strength

Concrete or shotcrete strength at time of stressing shall be at least 1.8 times the maximum initial stress due to prestressing in any wall section.

G.4.5.5 — Stress measurements and wire or strand winding records

G.4.5.5.1 — A calibrated stress-recording device that can be readily recalibrated shall be used to determine stress levels in prestressed reinforcement throughout the wrapping process. At least one stress reading for every coil of wire or strand, or for each 1000 lb thereof, or for every foot of wall per layer, shall be taken after the prestressed reinforcement has been applied on the wall.

G.4.5.5.2 — The total initial prestress force measured on the wall per vertical foot of height shall be not less than the specified initial force in the locations indicated on the design force diagram and not more than 5 percent greater than the specified force.

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RG.4.4 — Nonprestressed reinforcement—Concrete and shotcrete cover

RG.4.4.1 — The shotcrete covercoat may be taken as part of the cover over the diaphragm.

RG.4.5 — Prestressed reinforcement—Wire or strand winding

RG.4.5.1 — Qualifications

Winding should be under the direction of a supervisor having technical knowledge of prestressing principles and experience with the winding system being used.

RG.4.5.5 — Stress measurements and wire or stand winding records

Readings of the force in the prestressed reinforcement in-place on the wall should be made when the wire or strand has reached ambient temperature. All such readings should be made on straight lengths of prestressed reinforcement.

A written record of stress readings, including location and layer, should be maintained. This submission should be reviewed before acceptance of the work.

Continuous electronic recordings taken on the wire or strand in a straight line between the stressing head and the wall may be used in place of the above when the system allows no loss of tension between the reading and final placement on the wall.

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G.4.5.6 — Prestressed reinforcement stress adjustment

If the stress in the installed reinforcement is less than the specified initial prestress, additional wire or strand shall be applied to correct the deficiency. If the stress exceeds 1.07 times the specified initial prestress, the wrapping operation shall be discontinued immediately upon discovery, and adjustments shall be made before wrapping is restarted.

G.4.5.7 — Spacing of prestressed reinforcement

(a) Spacing of wire and strand shall meet the requirements of G.2.2.4.2;

(b) Wire or strand in areas adjacent to openings or inserts shall be uniformly spaced as described in G.2.2.7.

G.4.6 — Tolerances

G.4.6.1 — Tank radius

The maximum permissible deviation from the specified tank radius shall be 0.1 percent of the radius or 60 percent of the core wall thickness, whichever is less.

G.4.6.2 — Localized tank radius

The maximum permissible deviation of the tank radius along any 10 ft of circumference shall be 5 percent of the core wall thickness.

G.4.6.3 — Vertical walls

Walls shall be plumb within 3/8 in. per 10 ft of vertical dimension.

G.4.6.4 — Wall thickness

Wall thickness shall not vary more than minus 1/4 in. nor plus 1/2 in. from the designed thickness.

G.4.6.5 — Precast panels

The centroids of adjoining precast concrete panels shall not vary inwardly or outwardly in a radial direction from one another by more than 3/8 in.

G.4.6.6 — Concrete domes

The average maximum radius of any area on the surface of the dome with a diameter of $2.5\sqrt{r_d h_d}$ shall not be greater than $1.4r_d$.

G.4.7 — Earthquake cables

G.4.7.1 — Separation sleeves

Sleeves of rubber or other compressible material shall surround the strands at the joint to permit radial wall movements. Concrete or grout shall be prevented from entering the sleeve. The separation sleeves shall be

COMMENTARY

RG.4.6 — Tolerances

RG.4.6.6 — Concrete domes

Reference G.16 gives detailed guidance on the size of the average maximum radius and its impact on the required dome thickness.

RG.4.7 — Earthquake cables

Earthquake cables are installed in floor-wall or wall-roof connections to restrain differential tangential motion between the wall and the floor, footing, or roof. The proper design and installation of the seismic cables, including

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designed to provide compressible area around the seismic cables to accommodate radial movement of the wall.

G.4.7.2 — Protection

The cable shall be galvanized or epoxy coated. The portion of the cable not enclosed by sleeves shall bond to the wall concrete or shotcrete and to the floor, footing, or roof concrete.

G.4.8 — Elastomeric bearing pads

G.4.8.1 — Positioning

Bearing pads under the wall or dome shall be attached to the previously cast concrete surface to prevent uplift during subsequent concreting or shotcreting. Nailing of pads shall not be permitted unless pads are specifically designed for such anchorage.

G.4.8.2 — Free-sliding joints

When the wall is designed for a wall-floor joint that is free to translate radially, the joint shall be detailed and constructed to insure freedom from all obstructions to provide for free movement of the base of the wall.

G.4.9 — Sponge rubber fillers

G.4.9.1 — General

Sponge rubber fillers at wall-floor and wall-roof joints shall be of sufficient width and correctly placed to prevent voids between the sponge rubber, bearing pads, and waterstops. Fillers shall be detailed and installed to provide complete separation at the joint as required in the design. The method of securing sponge rubber pads is the same as for elastomeric bearing pads.

G.4.9.2 — Voids

All voids and cavities occurring between butted ends of pads, between pad and waterstops, and between pad and joint filler, shall be filled with sealant compatible with the materials of the pad, filler, waterstop, and the submerged surface. No concrete-to-concrete hard spots that would inhibit free translation of the wall shall be permitted.

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adequately-sized separation sleeves and attention to protection from corrosion, are essential to providing long-term seismic protection. The separation sleeves are designed to provide a compressible area around the seismic cables that accommodates radial movement of the wall.

Cables should be cut to uniform lengths before being placed in the footing forms. Care should be taken during placement of concrete to avoid compression of the bearing pad and restraint of radial wall movement.

RG.4.8 — Elastomeric bearing pads

RG.4.8.1 — Positioning

Bearing pads are normally attached to the concrete with a moisture-insensitive adhesive to prevent uplift during concreting or shotcreting. Pads in cast-in-place concrete walls should also be held in position and protected from damage from nonprestressed reinforcement by inserting small dense concrete blocks on top of the pad under the nonprestressed reinforcement ends.

APPENDIX H — SLABS-ON-SOIL

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H.1 — Scope

There are three types of slabs placed on soil:

1. Structural slabs;
2. Slabs-on-grade; and
3. Membrane slabs.

The requirements for structural slabs on soil are covered in other sections in this code. This appendix addresses the particular requirements for slabs-on-grade and membrane slabs. The design of both of these slab types is based on transmitting loads directly through the slab to the subgrade with no distribution or only wheel load distribution to the subgrade.

H.2 — Subgrade

H.2.1 — The subgrade shall have the necessary strength, stiffness, and stability to support the loads that are to be placed on it.

H.2.2 — Potential settlements of the subgrade shall be provided for in the floor design. Local hard and soft spots, if not avoidable, shall be considered in the slab design. Consideration shall be given to tank slabs founded on more than one type of subgrade, such as part cut and part fill.

H.2.3 — Provisions shall be made to prevent erosion of the base due to water flowing below the slab. Fill gradation shall be selected to permit free drainage without loss of fines or a geotextile fabric provided in the event leakage through a joint or the slab occurs. If the in-situ soils cannot be made acceptable, they shall be removed and replaced with a designed fill.

H.2.4 — The subgrade for slabs shall be of uniform density and compressibility to minimize differential settlement of the slab and footings. Disturbed subgrade or loosely consolidated soil or foundation material shall be removed and replaced with soil compacted as placed. Overexcavation and replacement by compacted imported material shall be required if foundation soils are unsatisfactory for the imposed loadings or do not provide uniform support.

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RH.1 — Scope

Structural slabs distribute applied loads over large areas of the subgrade or from the subgrade to the structure above. They also are used to bridge uniformly applied loadings over assumed soft spots in the subgrade, to distribute wall and column loads to the subgrade, or to transmit hydrostatic uplift forces to the walls and columns above the slab. Mat foundations are an example of structural slabs on soils.

Slabs-on-grade are typically used to provide a watertight barrier under fluid loads, provide a working surface when a tank is empty, and to distribute vehicle wheel loads to the subgrade. Membrane slabs perform the same functions, except that they are not designed to distribute vehicle wheel loads.

In general, membrane floors are primarily used for circular prestressed concrete tanks, but are also used for rectangular tanks and other structures. Their design is based on the theory that a thin slab can accept gradual differential subgrade settlements without affecting watertightness.

RH.2 — Subgrade

RH.2.2 — Soft spots are generally removed and replaced with a lean concrete, flowable fill, or structural fill. A leveling course of select fill should be placed over the entire slab area, especially when the slab is placed in a cut-fill area or where partial or full rock excavation is required for the slab.

RH.2.4 — Compaction should be in accordance with a geotechnical investigation and the recommendation of the geotechnical engineer. Compaction should achieve a density of at least 95 percent of the maximum laboratory density determined by ASTM D 1557 or D 698, according to the type of soil and the geotechnical engineer's recommendations. Field tests for measurement of in-place density should be in accordance with ASTM D 1556.

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H.2.5 — Tolerances for the elevation of the prepared subgrade directly beneath the slab shall be +0 in. and –1 in. over any 100 ft-long section. All transitions in elevation shall be smooth and gradual.

H.3 — Slab thickness

H.3.1 — The minimum thicknesses for slabs-on-grade are:

- 4 in. for slabs with one layer of nonprestressed reinforcement;
- 5 in. for slabs with prestressed reinforcement;
- 6 in. for slabs with top and bottom nonprestressed reinforcement.

H.3.2 — The minimum thicknesses for membrane slabs are:

- 3 in. for slabs with welded wire fabric reinforcement;
- 4 in. for nonprestressed slabs with welded wire fabric reinforcement or bars not greater that #4. Non-prestressed slabs with reinforcement larger than #4 bars shall have a minimum thickness resulting from meeting the concrete cover requirements of Section H.4.4;
- 5 in. for prestressed slabs.

H.3.3 — The maximum thicknesses of membrane slabs are:

- 6 in. for nonprestressed slabs;
- 7 in. for prestressed slabs.

H.3.4 — Tolerances for finished slab surface elevation shall be –0 in. and +3/4 in. with no greater difference than ±1/4 in. in 10 ft.

H.4 — Reinforcement

H.4.1 — Slabs-on-grade shall be reinforced in one or two layers. Membrane slabs shall be reinforced with one layer. The minimum reinforcement shall not be less than the requirements for temperature and shrinkage for structural slabs as described in other sections of this code. Prestressed reinforcement, if used, shall not be less than the amount required to impart a final compression of 200 psi in the slab.

H.4.2 — The minimum ratio of reinforcement area to concrete area shall be 0.005 in each orthogonal direction for nonprestressed membrane slabs. Additional reinforcement shall be provided at floor edges and other discontinuities as required by the design. Welded wire fabric sheets or deformed bar reinforcement shall be used. Maximum wire spacing for welded wire fabric

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RH.3 — Slab thickness

RH.3.2 — Concrete membrane slabs should be as thin as practical with consideration given to construction methods and concrete cover of reinforcement. Slabs requiring two layers of reinforcement would not be considered membrane slabs.

RH.4 — Reinforcement

RH.4.1 — The minimum required reinforcement provides crack control for watertight construction within the allowable limits of criteria for leakage.

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shall be 4 in. Maximum spacing of bar reinforcement shall be the lesser of 12 in. or two times the slab thickness.

H.4.3 — Prestressed membrane slabs shall have reinforcement in each orthogonal direction with a minimum final prestressing of 200 psi. The prestressed reinforcement shall be located within the center one-third of the slab. The tendons shall be tensioned as soon as the concrete compressive strength is adequate to resist the anchorage forces. The minimum ratio of nonprestressed reinforcement area to concrete area shall be 0.0015 in each orthogonal direction. Additional reinforcement shall be provided at slab edges and other discontinuities as required by the design.

H.4.4 — Reinforcement shall have a concrete cover of not less than 1-1/2 in. between the bars or wires and the top surface of the concrete for slabs-on-grade, and 1 in. for membrane slabs. The cover between the bottom of the slab and the reinforcement for both slabs-on-grade and membrane slabs shall not be less than 1-1/2 in. when the subgrade is stabilized so that it will not be displaced by the placement of concrete or when the subgrade is covered with a plastic vapor barrier. The cover shall not be less than 2 in. when the subgrade has not been stabilized or when a vapor barrier is not present. Slabs greater than 8 in. in thickness shall have the same concrete cover requirements as for structural slabs-on-soil. Reinforcement shall be maintained in correct vertical position by support chairs or concrete cubes.

H.5 — Joints

H.5.1 — Waterstops shall be used in all slab joints for liquid-containing structures. Slabs shall be placed continuously in sections as large as practical to avoid construction joints where there is no room for a waterstop. Integral waterstops shall not be used in sections less than 5 in. thick. Integral waterstops shall have at least 2 in. of concrete between the surface of the waterstop and the surface of the concrete. Waterstops designed to be placed at the bottom of the joint shall have no surface of the waterstop within 1 in. of the reinforcing steel.

H.5.2 — The design of slabs-on-soil shall take into account any thickened slab sections or transitions that are provided in the slab.

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RH.4.3 — Stressing of the prestressing steel in two stages is recommended with the first stage within 24 hours of placing concrete.

RH.4.4 — The reduction of cover requirements from other sections of the ACI 350 Code is based on stabilizing the soil to minimize intrusion into the concrete and a high control of subgrade surface plane. Also, the reinforcing steel is not as critical as in structural concrete because the loads are being passed through the concrete rather than being distributed by structural reinforced concrete. Membrane slabs, due to the flexibility of the slabs, will also result in reduced crack widths.

For slabs greater than 8 in. thick, the cover requirements are the same as for structural slabs, as they will tend to have the same action. See ACI 372 for guidance on stabilized base material.

RH.5 — Joints

RH.5.1 — Entire membrane slabs should be cast with no cold joints or construction joints, if practical. Factors to consider include expected experience of the contractor and climate conditions.

RH.5.2 — Thickening of slabs-on-grade at joints may result in the formation of cracks adjacent to the thickened area, as the joint is stronger than the basic slab. Thickening of membrane slabs at joints changes the slab action from a membrane to a stiffened slab. Separate concrete slab sections may be provided below the joints to provide additional room for the waterstop and any additional reinforcement.

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H.6 — Hydrostatic uplift

H.6.1 — Floors subject to hydrostatic uplift shall be provided with under-floor drainage or hydrostatic pressure relief valves or be designed to resist the uplift pressure. When pressure relief valves are used, the floor shall be designed to resist the uplift pressures required to initially open the pressure relief valve.

H.7 — Curing

H.7.1 — Concrete shall be cured in accordance with this code.

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RH.6 — Hydrostatic uplift

RH.6.1 — Slabs that are thickened to counteract uplift pressures may no longer act as a membrane. Pressure relief valves should not be used when contamination of the contents or subgrade may occur. Rock or soil anchors are a means of anchoring the floor against uplift if subgrade conditions permit. Drainage can be directed to a manhole or other drainage structure where the flow can be observed and measured. An alarm system may be used to alert personnel and activate filling the tanks.

RH.7 — Curing

RH.7.1 — Preferred methods of water curing include ponding, soaking, or use of wet coverings.

APPENDIX I — ALTERNATE DESIGN METHOD

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COMMENTARY

I.0 — Notation

Some notation definitions are modified from those in the main body of the code for specific use in the application of Appendix I.

A_g = gross area of section, in.^{2}

A_v = area of shear reinforcement within a distance s, in.^{2}

A_1 = loaded area, in.^{2}

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2}

b_o = perimeter of critical section for slabs and footings, in.

b_w = web width, or diameter of circular section, in.

d = distance from extreme compression fiber to centroid of tension reinforcement, in.

E_c = modulus of elasticity of concrete, psi. See 8.5.1

E_s = modulus of elasticity of reinforcement, psi. See 8.5.2

f_c' = specified compressive strength of concrete, psi. See Chapter 5

$\sqrt{f_c'}$ = square root of specified compressive strength of concrete, psi

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi.

f_s = permissible tensile stress in reinforcement, psi

f_y = specified yield strength of reinforcement, psi. See 3.5.3

M = design moment, in.-lb

n = modular ratio of elasticity

= E_s/E_c

N = design axial load normal to cross section occurring simultaneously with V; to be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage, lb

s = spacing of shear reinforcement in direction parallel to longitudinal reinforcement, in.

v = design shear stress

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v_c = permissible shear stress carried by concrete, psi

v_h = permissible horizontal shear stress, psi

V = design shear force at section

α = angle between inclined stirrups and longitudinal axis of member

β_c = ratio of long side to short side of concentrated load or reaction area

ρ_w = ratio of tension reinforcement

= A_s/b_wd

ϕ = strength reduction factor. See I.2.1.

I.1 — Scope

I.1.1 — Nonprestressed reinforced concrete members shall be permitted to be designed using service loads (without load and environmental-durability factors) and permissible service load stresses in accordance with provisions of Appendix I.

I.1.2 — For design of members not covered by Appendix I, appropriate provisions of this code shall apply.

I.1.3 — All applicable provisions of this code for nonprestressed concrete, except 8.4, shall apply to members designed by the alternate design method.

I.1.4 — Flexural members shall meet requirements for deflection control in 9.5, and requirements of 10.4 through 10.7 of this code.

COMMENTARY

RI.1 — Scope

As an alternate to the strength design method of this code, the design provisions of Appendix I may be used to proportion reinforced concrete members. In the alternate method, a structural member (in flexure) is so designed that the stresses resulting from the action of service loads (without load factors) and computed by the straight-line theory for flexure do not exceed permissible service load stresses. Service load is the load, such as dead, live, and wind that is assumed actually to occur when the structure is in service. The required service loads to be used in design are as prescribed in the general building code. The stresses computed under the action of service loads are limited to values well within the elastic range of the materials so that the straight-line relationship between stress and strain is used (see I.5).

The alternate method is similar to the “working stress design method” of previous ACI Building Codes (for example, ACI 318-63). For members subject to flexure without axial load, the method is identical. Major differences in procedure occur in design of compression members with or without flexure (see I.6) and bond stress and development of reinforcement (see I.4). For shear, the shear strengths provided by concrete for the strength design method are divided by a factor of safety, and the resulting permissible service load stresses is restated in Appendix I (see I.7).

In view of the simplifications permitted, the alternate design method of Appendix I generally will result in designs similar to those designs obtained using the strength design method of this code. Load factors and strength reduction factors of 1.0 are used for both design and analysis. Also, design rules for proportioning by the straight-line theory for flexure have not been updated as thoroughly as the strength design method for proportioning reinforced concrete members.

RI.1.1 — Design by Appendix I does not apply to prestressed members. (Chapter 18 permits linear stress-strain assumptions for computing service load stresses and prestress transfer stresses for investigation of behavior at service conditions.)

CODE

COMMENTARY

RI.1.3 — All other provisions of this code, except those permitting moment redistribution, apply to the alternate design method. These include control of deflections and distribution of flexural reinforcement, as well as all of the provisions related to slenderness effects in compression members in Chapter 10.

RI.1.4 — The general serviceability requirements of this code, such as the requirements for deflection control (see 9.5) and crack control (see 10.6), must be met regardless of whether the strength method or the alternate method is used for design.

I.2 — General

I.2.1 — Load factors and strength reduction factors ϕ shall be taken as unity for members designed by the Alternate Design Method, except as otherwise required by the governing building code.

I.2.2 — Allowable stresses may be increased by one-third where permitted by the governing building code.

I.2.3 — When dead load reduces effects of other loads, members shall be designed for 85 percent of dead load in combination with the other loads.

RI.2 — General

RI.2.1 — Load factors and strength reduction factors for determining safety in the strength design method are not used in the alternate design method. Accordingly, load factors and strength reduction factors ϕ are set equal to 1.0 to eliminate their effect when designing by the alternate method.

When using the moment and shear equations of 8.3.3 and Chapter 13, the factored load w_u must be replaced by the service load w.

RI.2.2 — When lateral loads such as wind or earthquake combined with live and dead load govern the design, members may be proportioned for 75 percent of capacities required in Appendix I. This is similar to the working stress design provisions of previous ACI Building Codes that allowed a one-third increase in stresses for these combinations of loads.

RI.2.3 — The 15 percent reduction for dead load is required for design conditions where dead load reduces the design effects of other loads to allow for the actual dead load being less than the dead load used in design. This provision is analogous to the required strength equation (Eq. (9-6)).

I.3 — Permissible service load stresses

RI.3 — Permissible service load stresses

For convenience, permissible service load stresses are tabulated. Compressive stress in concrete for flexure without axial load is limited to $0.45 f_c'$. Tensile stresses in reinforcement are limited to 20,000 psi for Grade 40 and 50 steel, and 24,000 psi for Grade 60 and higher strength steel. One exception of long standing exists for one-way slabs with clear span lengths 12 ft or less and reinforced with No. 3 bars or welded wire fabric having a diameter not exceeding 3/8 in. For this design condition only, the permissible tensile stress is increased to the lesser of $0.5 f_y$ or 30,000 psi.

Permissible stresses for shear and bearing are percentages of the shear and bearing strengths provided for strength design. The 10 percent increase permitted for joists by 8.11 of the code is already included in the $1.2\sqrt{f_c'}$ value for joists.

Clarification of the use of areas A_1 and A_2 for increased bearing stress is discussed in R10.17.1.

CODE

COMMENTARY

I.3.1 — Stresses in concrete shall not exceed the following:

(a) Flexure
Extreme fiber stress in compression .... $0.45f_c'$

(b) Shear*
Beams and one-way slabs and footings:
Shear carried by concrete, v_c........................ $1.1\sqrt{f_c'}$

Maximum shear carried by concrete plus shear reinforcement .... $v_c + 4.4\sqrt{f_c'}$

Joists:†
Shear carried by concrete, v_c...................... $1.2\sqrt{f_c'}$

Two-way slabs and footings:
Shear carried by concrete, v_c‡ .... $\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'}$

....but not greater than $2\sqrt{f_c'}$

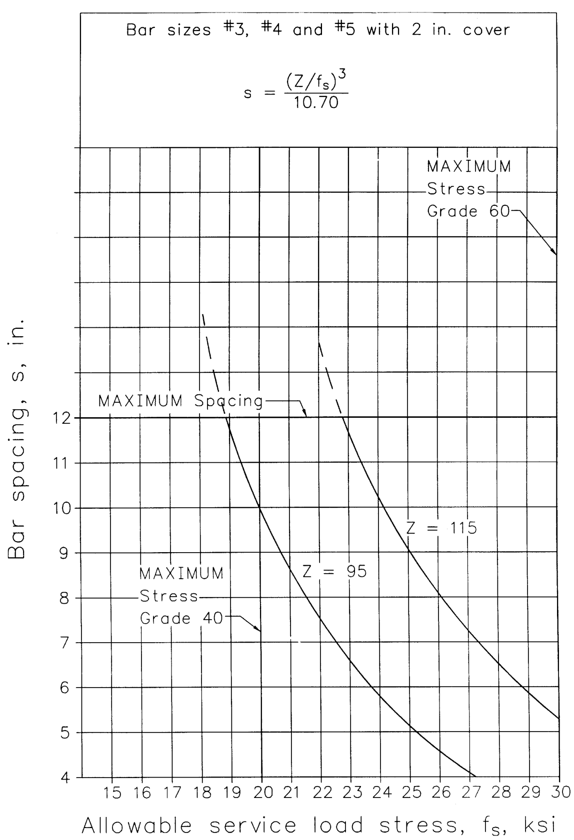

Fig. I.3.3(a) — Bar spacing for flexural crack control (No. 3 through No. 5 bars).

* For more detailed calculation of shear stress carried by concrete v_c and shear values for lightweight aggregate concrete, see I.7.4.
† Designed in accordance with 8.11 of this code.
‡ If shear reinforcement is provided, see I.7.7.4 and I.7.7.5.

CODE

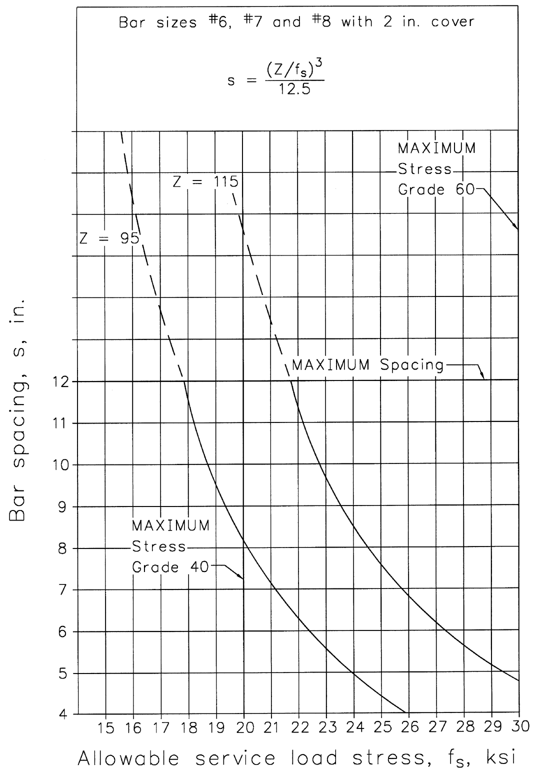

Fig. I.3.3(b) — Bar spacing for flexural crack control (No. 6 through No. 8 bars).

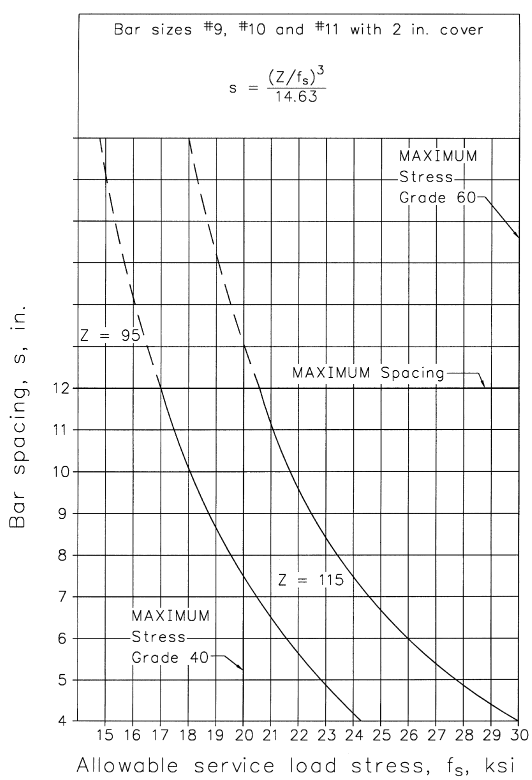

Fig. I.3.3(c) — Bar spacing for flexural crack control (No. 9 through No. 11 bars).

COMMENTARY

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(c) Bearing on loaded area* .... $0.3f_c'$

I.3.2 — Tensile stress in reinforcement f_s shall not exceed the following, unless detailed calculations are performed to address the flexural steel distribution requirements of I.3.3.

(a) Members in axial tension....................20,000 psi

(b) Flexural and shear reinforcement:

		Maximum Stress, psi	
Bar size	Exposure Condition	Grade 40 or 50	Grade 60
#3 - 5	Severe	19,000	19,000
	Normal	20,000	23,000
#6 - 8	Severe	18,000	18,000
	Normal	20,000	22,000
#9 - 11	Severe	17,000	17,000
	Normal	20,000	21,000

I.3.3 — In lieu of the maximum allowable flexural stresses in I.3.2, the allowable flexural stresses and related bar spacings given in Fig. I.3.3(a), (b), and (c), may be used in design. Where allowable flexural stresses exceeding 24 ksi are used in design, strength provisions or ACI 350 must also be satisfied.

I.4 — Development and splices of reinforcement

I.4.1 — Development and splices of reinforcement shall be as required in Chapter 12 of this code.

I.4.2 — In satisfying requirements of 12.11.3, M_n shall be taken as computed moment capacity assuming all positive moment tension reinforcement at the section to be stressed to the permissible tensile stress f_s, and V_u shall be taken as unfactored shear force at the section.

I.5 — Flexure

For investigation of stresses at service loads, straight-line theory (for flexure) shall be used with the following assumptions.

I.5.1 — Strains vary linearly as the distance from the neutral axis, except for deep beams with overall depth-span ratios greater than 2/5 for continuous spans and

*When the supporting surface is wider on all sides than the loaded area, permissible bearing stress on the loaded area shall be permitted to be multiplied by $\sqrt{A_2/A_1}$ but not more than 2. When the supporting surface is sloped or stepped, A_2 shall be permitted to be taken as the area of the lower base of the largest frustum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

COMMENTARY

RI.3.2 — Permissible stresses in reinforcement are established to provide enhanced crack control. Values given for flexural and shear reinforcement are suitable for reinforcing bars spaced at no more than 12 in. centers.

Direct tension forces can produce concrete tensile stresses over an entire cross-sectional area of concrete, resulting in full-depth cracking and exposure of all reinforcement at a section location. Therefore, low permissible stresses are used for direct tension members to limit resulting crack widths. Some designers prefer to use a permissible stress of 14,000 psi for grade 40 reinforcement in hoop tension members, based on their good experiences with designs using the earlier version of the PCA publication, “Circular Concrete Tanks Without Prestressing.” ACI 224.2R provides guidance for estimating crack widths in axial tension members.

RI.3.3 — Closer spacing provides better crack control; hence, higher permissible stresses may be used. Figure I.3.3(a), (b), and (c) may be used for a more detailed analysis of permissible flexural stresses. Other applicable analyses are also permitted. Where flexural stresses in accordance with Figure I.3.3(a), (b), and (c) are used that exceed 24 ksi, strength provisions in accordance with the code should also be satisfied.

RI.4 — Development and splices of reinforcement

In computing development lengths and splice lengths, the provisions of Chapter 12 govern both methods of design equally because, in either case, the development lengths (and splice lengths as multiples of development lengths) are based on the yield strength of the reinforcement. Where M_n and V_u are referenced in Chapter 12, M_n is the service load resisting moment capacity, and V_u is the applied service load shear force (without load factors) at the section.

RI.5 — Flexure

The straight-line theory applies only to design of members in flexure without axial load. Because stresses computed under the action of service loads are well within the elastic range, the straight-line relationship between stress and strain is used with the maximum fiber stress in concrete limited to $0.45f_c'$, and the tensile stress in reinforcement limited to 23,000 psi for Grade 60 steel (see I.3.2).

Straight-line theory may be used for all sectional shapes with or without compression reinforcement when axial load is not present. Because small axial compression loads tend to increase the moment capacity of a section, small axial loads may be disregarded in most cases. When doubt exists as to whether or not the axial compression may be disregarded, the member should be investigated using I.6.

CODE

4/5 for simple spans, a nonlinear distribution of strain shall be considered. See 10.7 of this code.

I.5.2 — Stress-strain relationship of concrete is a straight line under service loads within permissible service load stresses.

I.5.3 — In reinforced concrete members, concrete resists no tension.

I.5.4 — It shall be permitted to take the modular ratio, $n = E_s/E_c$, as the nearest whole number (but not less than 6). Except in calculations for deflections, value of n for lightweight concrete shall be assumed to be the same as for normalweight concrete of the same strength.

I.5.5 — In doubly reinforced flexural members, an effective modular ratio of $2E_s/E_c$ shall be used to transform compression reinforcement for stress computations. Compressive stress in such reinforcement shall not exceed permissible tensile stress.

I.6 — Compression members with or without flexure

I.6.1 — Combined flexure and axial load capacity of compression members shall be taken as 40 percent of that computed in accordance with provisions in Chapter 10 of this code.

I.6.2 — Slenderness effects shall be included according to requirements of 10.10 through 10.13. In Eq. (10-11) and (10-20) the term P_u shall be replaced by 2.5 times the design axial load, and the factor 0.75 shall be taken equal to 1.0.

I.6.3 — Walls shall be designed in accordance with Chapter 14 of this code with flexure and axial load capacities taken as 40 percent of that computed using Chapter 14. In Eq. (14-1), ϕ shall be taken equal to 1.0.

I.7 — Shear and torsion

I.7.1 — Design shear stress v shall be computed by

$$v = \frac{V}{b_w d} \qquad \text{(I-1)}$$

where V is design shear force at section considered.

I.7.2 — When the reaction, in direction of applied shear, introduces compression into the end regions of a member, sections located less than a distance d from face of support shall be permitted to be designed for the same shear v as that computed at a distance d.

COMMENTARY

Deep beams must be designed in accordance with 10.7 of the code.

In transforming compression reinforcement to equivalent concrete for flexural design, $2E_s/E_c$ must be used in locating the neutral axis and calculating moments of inertia. The lesser of twice the calculated stress in the compression reinforcement or the permissible tensile stress is then used to calculate the contribution of the compression reinforcement in computing the resisting moment at service loads.

RI.6 — Compression members with or without flexure

All compression members, with or without flexure, must be proportioned using the strength design method. This departure from the 1963 and previous ACI Building Codes is to provide a more consistent factor of safety for the full range of load-moment interaction. Existing working stress design aids for columns do not satisfy requirements of Appendix I.

The permissible service load capacity is taken as 40 percent of the nominal axial load strength P_n at given eccentricity (ϕ = **1.0**) as computed by the provisions of Chapter 10, subject to appropriate reduction due to effects of slenderness. Use of 40 percent of the nominal strength is equivalent to an overall safety factor U/ϕ of 2.5.

With the alternate design method, P_u/ϕ in Eq. (10-11) and (10-20) is taken as $2.5P$ when gravity loads govern, and as $1.875P$ when lateral loads combined with gravity loads govern the design, where P is the design axial load in the compression member.

RI.7 — Shear and torsion

For convenience, a complete set of design provisions for shear is provided in Appendix I.

The permissible concrete stresses and limiting maximum stresses for shear are 55 percent for beams, joists, walls, and one-way slabs and 50 percent for two-way slabs and footings, respectively, of the shear and torsional moment strengths given in the code for the strength design method.

When gravity load, wind, earthquake, or other lateral forces cause transfer of moment between slab and column, provisions of 11.12.2 must be applied with the permissible stresses on the critical section limited to those given in I.7.7.3.

CODE

COMMENTARY

I.7.3 — Whenever applicable, effects of torsion, in accordance with provisions of Chapter 11 of this code, shall be added. Shear and torsional moment strengths provided by concrete and limiting maximum strengths for torsion shall be taken as 55 percent of the values given in Chapter 11.

I.7.4 — Shear stress carried by concrete

I.7.4.1 — For members subject to shear and flexure only, shear stress carried by concrete v_c shall not exceed $1.1\sqrt{f_c'}$ unless a more detailed calculation is made in accordance with I.7.4.4.

I.7.4.2 — For members subject to axial compression, shear stress carried by concrete v_c shall not exceed $1.1\sqrt{f_c'}$ unless a more detailed calculation is made in accordance with I.7.4.5.

I.7.4.3 — For members subject to significant axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using

$$v_c = 1.1\left(1 + 0.004\frac{N}{A_g}\right)\sqrt{f_c'} \qquad \text{(I-2)}$$

where N is negative for tension. Quantity N/A_g shall be expressed in psi.

I.7.4.4 — For members subject to shear and flexure only, it shall be permitted to compute v_c by

$$v_c = \sqrt{f_c'} + 1300\rho_w\frac{Vd}{M} \qquad \text{(I-3)}$$

but v_c shall not exceed $1.9\sqrt{f_c'}$. Quantity Vd/M shall not be taken greater than 1.0, where M is design moment occurring simultaneously with V at section considered.

I.7.4.5 — For members subject to axial compression, it shall be permitted to compute v_c by

$$v_c = 1.1\left(1 + 0.0006\frac{N}{A_g}\right)\sqrt{f_c'} \qquad \text{(I-4)}$$

Quantity N/A_g shall be expressed in psi.

I.7.4.6 — Shear stresses carried by concrete v_c apply to normalweight concrete. When lightweight aggregate concrete is used, one of the following modifications shall apply:

(a) When f_{ct} is specified and concrete is proportioned in accordance with 5.2, $f_{ct}/6.7$ shall be substituted for $\sqrt{f_c'}$ but the value of $f_{ct}/6.7$ shall not exceed $\sqrt{f_c'}$.

(b) When f_{ct} is not specified, the value of $\sqrt{f_c'}$ shall be multiplied by 0.75 for "all-lightweight" concrete

CODE

and by 0.85 for "sand-lightweight" concrete. Linear interpolation shall be permitted when partial sand replacement is used.

I.7.4.7 — In determining shear stress carried by concrete v_c, whenever applicable, effects of axial tension due to creep and shrinkage in restrained members shall be included and it shall be permitted to include effects of inclined flexural compression in variable-depth members.

I.7.5 — Shear stress carried by shear reinforcement

I.7.5.1 — Types of shear reinforcement

Shear reinforcement shall consist of one of the following:

(a) Stirrups perpendicular to axis of member;

(b) Welded wire fabric with wires located perpendicular to axis of member making an angle of 45 deg or more with longitudinal tension reinforcement;

(c) Longitudinal reinforcement with bent portion making an angle of 30 deg or more with longitudinal tension reinforcement;

(d) Combinations of stirrups and bent longitudinal reinforcement;

(e) Spirals.

I.7.5.2 — Design yield strength of shear reinforcement shall not exceed 60,000 psi.

I.7.5.3 — Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fiber and shall be anchored at both ends according to 12.13 of this code to develop design yield strength of reinforcement.

I.7.5.4 — Spacing limits for shear reinforcement

I.7.5.4.1 — Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/2$ nor 12 in.

I.7.5.4.2 — Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45-deg line, extending toward the reaction from mid-depth of member ($d/2$) to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

I.7.5.4.3 — When $(v - v_c)$ exceeds $2\sqrt{f_c'}$, spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/4$, nor 12 in.

I.7.5.4.4 — When $(v - v_c)$ exceeds $2\sqrt{f_c'}$ spacing of inclined stirrups and bent longitudinal reinforcement shall be such that any 22.5 degree line drawn, extending toward the reaction from mid-depth of member ($d/2$) to longitudinal tension reinforcement, shall be crossed by a least one line of shear reinforcement.

COMMENTARY

RI.7.5.4 — Spacing limits for shear reinforcement

A 12 in. maximum spacing of reinforcement is required in liquid-containing structures for crack control.

CODE

COMMENTARY

I.7.5.5 — Minimum shear reinforcement

I.7.5.5.1 — A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members where design shear stress v is greater than one-half the permissible shear stress v_c carried by concrete, except:

(a) Slabs and footings;

(b) Concrete joist construction defined by 8.11 of this code;

(c) Beams with total depth not greater than 10 in., 2-1/2 times thickness of flange, or one-half the width of web, whichever is greatest.

I.7.5.5.2 — Minimum shear reinforcement requirements of I.7.5.5.1 shall be permitted to be waived if shown by test that required ultimate flexural and shear strength can be developed when shear reinforcement is omitted.

I.7.5.5.3 — Where shear reinforcement is required by I.7.5.5.1 or by analysis, minimum area of shear reinforcement shall be computed by

$$A_v = 50\frac{b_w s}{f_y} \tag{I-5}$$

where b_w and s are in inches.

I.7.5.6 — Design of shear reinforcement

I.7.5.6.1 — Where design shear stress v exceeds shear stress carried by concrete v_c, shear reinforcement shall be provided in accordance with I.7.5.6.2 through I.7.5.6.8.

I.7.5.6.2 — When shear reinforcement perpendicular to axis of member is used

$$A_v = \frac{(v - v_c)b_w s}{f_s} \tag{I-6}$$

I.7.5.6.3 — When inclined stirrups are used as shear reinforcement

$$A_v = \frac{(v - v_c)b_w s}{f_s(\sin\alpha + \cos\alpha)} \tag{I-7}$$

I.7.5.6.4 — When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support

$$A_v = \frac{(v - v_c)b_w d}{f_s \sin\alpha} \tag{I-8}$$

where $(v - v_c)$ shall not exceed $1.6\sqrt{f_c'}$.

I.7.5.6.5 — When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel

CODE

bent-up bars at different distances from the support, required area shall be computed by Eq. (I-7).

I.7.5.6.6 — Only the center three-quarters of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

I.7.5.6.7 — When more than one type of shear reinforcement is used to reinforce the same portion of a member, required area shall be computed as the sum of the various types separately. In such computations, v_c shall be included only once.

I.7.5.6.8 — Value of $(v - v_c)$ shall not exceed $4.4\sqrt{f_c'}$.

I.7.6 — Shear-friction

Where it is appropriate to consider shear transfer across a given plane, such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times, shear-friction provisions of 11.7 of this code shall be permitted to be applied, with limiting maximum stress for shear taken as 55 percent of that given in 11.7.5. Permissible stress in shear-friction reinforcement shall be that given in I.3.2.

I.7.7 — Special provisions for slabs and footings

I.7.7.1 — Shear capacity of slabs and footings in the vicinity of concentrated loads or reactions is governed by the more severe of two conditions:

I.7.7.1.1 — Beam action for slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from face of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with I.7.1 through I.7.5.

I.7.7.1.2 — Two-way action for slab or footing, with a critical section perpendicular to plane of slab and located so that its perimeter is a minimum, but need not approach closer than $d/2$ to perimeter of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with I.7.7.2 and I.7.7.3.

I.7.7.2 — Design shear stress v shall be computed by

$$v = \frac{V}{b_o d} \qquad \text{(I-9)}$$

where V and b_o shall be taken at the critical section defined in I.7.7.1.2.

I.7.7.3 — Design shear stress v shall not exceed v_c given by Eq. (I-10) unless shear reinforcement is provided

COMMENTARY

CODE

COMMENTARY

$$v_c = \left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'} \qquad \text{(I-10)}$$

but v_c shall not exceed $2\sqrt{f_c'}$. β_c is the ratio of long side to short side of concentrated load or reaction area. When lightweight aggregate concrete is used, the modifications of I.7.4.6 shall apply.

I.7.7.4 — If shear reinforcement consisting of bars or wires is provided in accordance with 11.12.3 of this code, v_c shall not exceed $\sqrt{f_c'}$, and v shall not exceed $3\sqrt{f_c'}$.

I.7.7.5 — If shear reinforcement consisting of steel I- or channel-shaped sections (shearheads) is provided in accordance with 11.12.4 of this code, v on the critical section defined in I.7.7.1.2 shall not exceed $3.5\sqrt{f_c'}$, and v on the critical section defined in 11.12.4.7 shall not exceed $2\sqrt{f_c'}$. In Eq. (11-39) and (11-40), design shear force V shall be multiplied by 2 and substituted for V_u.

I.7.8 — Special provisions for other members

For design of deep beams, brackets and corbels, and walls, the special provisions of Chapter 11 of this code shall be used, with shear strengths provided by concrete and limiting maximum strengths for shear taken as 55 percent of the values given in Chapter 11. In 11.10.6, the design axial load shall be multiplied by 1.2 if compression and 2.0 if tension, and substituted for N_u.

I.7.9 — Composite concrete flexural members

For design of composite concrete flexural members, permissible horizontal shear stress v_h shall not exceed 55 percent of the horizontal shear strengths given in 17.5.2 of this code.

INDEX

Notes

SUMMARY OF CHANGES FOR 350-06 CODE

CHAPTER 1 — GENERAL REQUIREMENTS

Technical revisions made to Chapter 1 of the 2006 Code and Commentary:

- Added a provision that the specified concrete compressive strength shall not be less than 4000 psi. Revision is to 1.1.1.
- Commentary has been revised to update the current scope. Revision is to R1.1.
- The scope of environmental engineering concrete structures includes further clarification. Revision is to 1.1.1.1.
- Updated information regarding the design of concrete piles and piers in regions of high seismic risk or assigned to high seismic performance or design categories. Revision is to 1.1.5.
- Provided clarification where special provisions for earthquake resistance are applied and regulated. Revisions are in 1.1.8.
- Added requirements to be shown in drawings, details, and specifications. Revision is to 1.2.1.
- Included additional requirements regarding the inspection of concrete construction. Revision is to 1.3.1.
- Added tightness testing to be included in inspection records. Revision is to 1.3.2.
- Updated inspection requirements for special moment frames resisting seismic loads in regions of high seismic risk or in structures assigned to high seismic performance or design categories. Revision is to 1.3.5.
- Revised Section R1.1.8.2 and Table R1.1.8.2.
- Additional guidance has been given regarding the design of above- and below-grade structures. Revision is to R1.1.1.
- Included additional information regarding cast-in-place and precast wall panels. Revision is to R1.1.8.1.

CHAPTER 2 — DEFINITIONS

Technical revisions made to Chapter 2 of the 2006 Code:

- Added definitions for anchorage zone, basic monostrand anchorage device, basic multistrand anchorage device, duct, moment frame, intermediate moment frame, ordinary moment frame, special moment frame, prestressing steel, sheathing, structural walls, intermediate precast structural wall, ordinary reinforced concrete structural wall, special precast structural wall, special reinforced concrete structural wall, unbonded tendon, backer rod, environmental durability factor, joint filler, joint sealant, and waterstop. Revision is to 2.1.
- Updated definitions for anchorage device, bonded tendon, effective prestress, jacking force, post-tensioning, pretensioning, reinforced concrete, reinforcement, and tendon. Revision is to 2.1.
- Commentary has been added for anchorage device, anchorage zone, basic anchorage devices, and sheathing.

CHAPTER 3 — MATERIALS

Technical revisions made to Chapter 3 of the 2006 Code and Commentary:

- Revised the requirement for record retention in 3.1.3.
- Update information in 3.3.2(c).
- Provided additional information in 3.5.3.1 regarding deformed reinforcing bars.
- Added certain requirements for some types of air-entraining admixtures. Revision is to 3.6.4 and R3.6.4.
- Updated the reference standards. Revision is to 3.8.
- Renumbered 3.8.3 to 3.8.4 and added new 3.8.3.
- Added 3.8.5 and 3.8.7.
- Renumbered 3.8.4 to 3.8.8.

CHAPTER 4 — DURABILITY REQUIREMENTS

2006 Code indicates technical changes in the following sections:

- 4.1, 4.1.2, and Table 4.2.2.

The aforementioned changes were due to:

- Adding a table to indicate minimum cementitious material content.
- Clarification of requirements for corrosive chemical exposures.

2006 Commentary indicates technical changes in the following sections:

- R4.1, R4.1.1, R4.2.1, and R4.5.1.4.

The aforementioned changes were due to:

- Discussion concerning limits of pozzolans.
- Revised chemical listing.

CHAPTER 5 — CONCRETE QUALITY, MIXING, AND PLACING

Technical revisions made to Chapter 5 of the 2006 Code and Commentary:

- Included the requirement that concrete compressive strength shall not be less than 4000 psi. Revision is to 5.1.1.
- Added a requirement that data shall not be more than 12 months old. Revision is to 5.3.1.1.
- Added Table 5.3.2.1.
- Modified Table 5.3.2.2.
- Renumbered 5.6.4.3 to 5.5.4.3 and modified.

- Renumbered 5.6.2.3(b) to 5.5.2.3(b) and modified.
- Revised R5.3.2.1.
- Added second paragraph to R5.3.3.
- Renumbered R5.6.1.2 to R5.5.1.2 and revised.
- Renumbered R5.6.2.3 to R5.5.2.3 and revised.
- Renumbered R5.6.4 to R5.5.4 and added second paragraph.

CHAPTER 6 — FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION AND MOVEMENT JOINTS

Technical revisions made to Chapter 6 of the 2006 Code and Commentary:

- Provided additional clarification regarding conduits and pipes embedded in concrete. Revision is to 6.3.1.
- Provided some precautionary information regarding deterioration of concrete surrounding embedded galvanized conduits and pipes exposed to corrosive liquids and gases. Revision is to R6.3.6.

CHAPTER 7 — DETAILS OF REINFORCEMENT

2006 Code indicates technical changes in the following sections:

- 7.0, 7.1, 7.1.4, 7.4, 7.5, 7.6.3, 7.6.7, 7.7.1, 7.7.2, 7.7.3, 7.10.4.5, 7.10.5.6, and 7.13.2.

The aforementioned changes were due to:

- Editorial revisions.
- Clarification of tolerances for bends and ends of reinforcement in brackets and corbels.
- Clarification of spacing limits for pretensioning tendons.
- Expansion of lap splice requirements for spiral reinforcement.
- Added lateral reinforcement requirements for anchor bolts at the top of columns or pedestals.
- Modified structural integrity requirements for beams.

CHAPTER 8 — ANALYSIS AND DESIGN—GENERAL CONSIDERATIONS

2006 Code and Commentary indicate technical changes in the following sections:

- 8.4.1, 8.4.3, and R8.4.

The aforementioned changes were due to:

- Specific permissible moment redistribution percentage in terms of tensile strain consistent with incorporation of the unified design provision into the body of the code.

CHAPTER 9 — STRENGTH AND SERVICEABILITY REQUIREMENTS

2006 Code indicates technical changes in the following sections:

- 9.0, 9.1, 9.2, and 9.3.

The aforementioned changes were due to:

- Changes in definitions and notation.
- Moving of strength design provisions of the 2001 Code to Appendix C.
- Use of ASCE 7-02 as the basis for load combinations.
- Incorporation of revised definition for the environmental durability factor.
- Incorporation of unified design approach as the basis for strength design.

CHAPTER 10 — FLEXURE AND AXIAL LOADS

2006 Code and Commentary indicate technical changes in the following sections:

- 10.3.3, 10.3.4, 10.3.5, 10.5.2, 10.5.4, 10.6 (entire section), 10.12.2, and 10.15.3.
- R10.3.3, R10.3.5, R10.5, R10.6 (entire section), and R10.15.3.

The aforementioned changes were due to:

- The introduction of compression-controlled and tension-controlled sections.
- Clarifications of minimum tensile reinforcement.
- Changes to distribution requirements for flexural reinforcement.
- Clarification of Eq. (10-10).
- Added limit on ratio of column concrete strength.

CHAPTER 11 — SHEAR AND TORSION

2006 Code and Commentary indicate technical changes in the following sections:

- 11.1.1, 11.1.2.1, R11.1.2, R11.1.2.1, 11.1.3, R11.1.3.1, 11.3.3, R11.3.3, 11.5.5.3, 11.5.6.3, R11.5.6.3, 11.6.1, R11.6.1, 11.8.9, R11.8.9, 11.8.10, R11.8.10, 11.12.3, 11.12.3.1, 11.12.3.3, 11.12.3.4, and R11.12.3.

The aforementioned changes were due to:

- Clarification for the use of Environmental Durability Factor.
- Gradual increase in the minimum area of transverse reinforcement.
- Clarification on loads applied to the top of the beam.
- Circular member shear calculation.
- Definition for calculating circular ties, hoops, and spirals.
- Modification for calculation of threshold torsion for hollow sections.
- Minimum amounts of horizontal and vertical shear reinforcement in deep beams.
- Shear reinforcement provisions for slabs and footings expanded.

CHAPTER 12 — DEVELOPMENT AND SPLICES OF REINFORCEMENT

2006 Code indicates technical changes in the following sections:

- 12.0, 12.2.1, 12.2.4, 12.3, 12.5.2, 12.5.3, 12.5.4, 12.9.1, 12.9.2, 12.10.5, 12.11.4, 12.13.1, 12.13.2, 12.14.3, 12.15.3, 12.15.4, 12.15.5, 12.16.3, 12.17.1, 12.17.3, 12.19.

The aforementioned changes were due to:

- Editorial revisions.
- Expanded compression development requirements to include deformed wire.
- Modified requirements for basic development of standard hooks.

CHAPTER 13 — TWO-WAY SLAB SYSTEMS

2006 Code and Commentary indicate technical changes in the following sections:

- 13.3.8, 13.3.8.4 and R13.3.8.4, 13.3.8.5 and R13.3.8.5, 13.3.8.6 and R13.3.8.6, 13.5.1.2 and R13.5.1.2, and R13.5.3.3.

The aforementioned changes were due to:

- Two-way slab systems allow the use of mechanical or welded tension splices in two-way slabs.

CHAPTER 14 — WALLS

2006 Code indicates technical changes in the following sections:

- 14.2.2 and 14.8.

The aforementioned changes were due to:

- Section 14.8 was added to address slender wall design requirements.

CHAPTER 15 — FOOTINGS

2006 Code indicates technical changes in the following sections:

- 15.5.3, 15.8.3, and 15.8.3.3.

The aforementioned changes were due to:

- Section 15.5.3 was added to address eccentricity between the axis of the pile and the axis of the column. Wording was added to 15.8.3 and 15.8.3.3 to ensure anchor bolts satisfy the new Appendix D.

CHAPTER 16 — PRECAST CONCRETE

2006 Code and Commentary indicate technical changes in the following sections:

- 16.6.2.3, R16.3.2, and R16.6.2.3.

The aforementioned changes were due to:

- Accurate description of positive mechanical connections.
- Changes in end tolerance for reinforcement in 7.5.2.2 can result in the end bearing of precast members being on plain concrete when supports satisfy the minimum dimensions of 16.6.2.2.

CHAPTER 17 — COMPOSITE CONCRETE FLEXURAL MEMBERS

2006 Commentary indicates technical changes in the following sections:

- R17.2.6.

The aforementioned changes were due to:

- Change the reference to z-factors to refer to the newer method of flexural steel distribution in Chapter 10.

CHAPTER 18 — PRESTRESSED CONCRETE

2006 Code indicates technical changes in the following sections:

- 18.0, 18.1.4, 18.3.3, 18.3.4, 18.3.5, 18.3.6, 18.3.7, 18.9.3.3, 18.9.4, 18.13, 18.13.1, 18.13.2.1, 18.13.2.2, 18.13.2.3, 18.13.3, 18.13.3.1, 18.13.3.2, 18.13.3.3, 18.13.4, 18.13.4.1, 18.13.4.2, 18.13.4.3, 18.13.5, 18.13.5.1, 18.13.5.2, 18.13.5.3, 18.13.5.4, 18.13.5.5, 18.13.5.6, 18.13.5.7, 18.13.5.8, 18.13.6, 18.14, 18.14.1, 18.14.2, 18.14.2.1, 18.14.2.2, 18.14.2.3, 18.14.2.4, 18.14.3, 18.15, 18.15.1, 18.15.2, 18.15.3, 18.16, 18.16.1, 18.16.2, 18.16.3, 18.16.4, 18.17, 18.17.1, 18.17.2, 18.17.3, 18.17.4, 18.18, 18.18.1, 18.18.2, 18.18.2.1, 18.18.2.2, 18.18.2.3, 18.18.2.4, 18.18.2.5, 18.18.3, 18.18.3.1, 18.18.3.2, 18.18.3.3, 18.18.3.4, 18.18.3.5, 18.18.4, 18.18.4.1, 18.18.4.2, 18.18.4.3, 18.19, 18.20, 18.20.1, 18.20.2, 18.20.3, 18.20.4, 18.21, 18.21.1, 18.21.2, 18.21.3, 18.21.4, 18.22, 18.22.1, 18.22.2, 18.22.3, and 18.22.4.

2006 Commentary indicates technical changes in the following sections:

- R18.0, R18.1.4, R18.2.5, R18.2.6, R18.3.3, R18.3.6, R18.3.7, R18.9.3.3, R18.13, R18.13.1, R18.13.3, R18.13.4, R18.13.5, R18.13.5.3, R18.13.5.4, R18.13.5.5, R18.14, R18.14.2, R18.14.3, R18.15, R18.15.1, R18.15.2, R18.16, R18.16.1, R18.16.2, R18.16.3, R18.16.4, R18.17, R18.17.4, R18.18, R18.18.1, R18.18.2, R18.18.3, R18.18.4, R18.20, R18.20.1, R18.20.4, R18.21, R18.21.1, R18.21.3, R18.21.4, R18.22, R18.22.3, and R18.22.4.

The aforementioned changes were due to:

- ACI 350-01 used ACI 318-95 as a base document. Current code incorporates ACI 318-99 and ACI 318-02 revision applicable to Environmental Concrete Structures. Such revisions include provisions for prestressed concrete design that introduce ϕ = 0.85 in post-tensioned anchorage zones and the introduction of Class U and Class T members, revised design methods for variety of post-tensioned tendon anchorage zones, guidelines for external post-tensioning systems, and revised chapter formatting.

CHAPTER 19 — SHELLS AND FOLDED PLATE MEMBERS

2006 Code and Commentary indicate technical changes in the following sections:

- 19.2.6 and R19.2.6.

The aforementioned changes were due to:

- Maintain nomenclature consistent with ACI 318-02.

CHAPTER 20 — STRENGTH EVALUATION OF EXISTING STRUCTURES

2006 Code indicates technical changes in the following sections:

- 20.0, 20.2.3, 20.2.4, 20.2.5, 20.4.1, 20.4.4, 20.4.6, and 20.5.2.

2006 Commentary indicates technical changes in the following sections:

- R20.0, R20.1, R20.1.2, R20.2, R20.2.2, R20.2.3, R20.2.5, R20.3.1, R20.3.2, R20.4.3, R20.5.1, R20.5.2, R20.5.3, and R20.5.4.

The aforementioned changes were do to:

- Editorial revisions.
- ACI 350-01 used ACI 318-95 as a base document. This code incorporates the ACI 318-99 and ACI 318-02 revisions applicable to Environmental Concrete Structures.
- Changes in strength reduction factors to be compatible with the load combination and strength reduction factors of Chapter 9.

CHAPTER 21 — SPECIAL PROVISIONS FOR SEISMIC DESIGN

2006 Code indicates technical changes in the following sections:

- 21.1, 21.2.1.2, 21.2.1.3, 21.2.1.4, 21.2.1.5, 21.2.2.4, 21.2.6, 21.2.7, 21.2.8, 21.4.2.2, 21.4.3.2, 21.4.4.1, 21.6, 21.7, 21.8, 21.9, 21.10, 21.11.2, 21.11.3.1, 21.11.4, 21.12.5, 21.12.5.4, 21.12.6.2, 21.12.6.6, 21.12.6.8, and 21.13.

2006 Commentary indicates technical changes in the following sections:

- R21.0, R21.1, R21.2.1, R21.2.4, R21.2.6, R21.2.7, R21.3.1, R21.4.1 to R21.4.4, R21.6, R21.7.1 to R21.7.7, R21.9.1 to R21.9.8, R21.10.1 to R21.10.4, R21.11, R21.12, and R21.13.

The aforementioned changes were due to:

- Adding definitions for earthquake-resisting elements and connections.
- Redefining requirements based on seismic risk levels.
- Clarifying applicability of requirements for liquid-containing structures.
- Relating mechanical and welded splices and anchorage to seismic risk.
- Revising requirements for special moment frames.
- Adding requirements for special moment frames constructed using precast concrete.
- Separating structural wall requirements from requirements for diaphragms and trusses.
- Adding revised requirements for special reinforced concrete structural walls and coupling beams.
- Revising requirements for structural diaphragms and trusses.
- Adding requirements for special structural walls constructed using precast concrete.
- Adding requirements for foundations.
- Adjusting requirements, and including precast concrete frame members, for frame members not proportioned to resist forces induced by earthquake motions.
- Revising requirements for intermediate moment frames.
- Adding requirements for intermediate precast structural walls.

APPENDIX B — ALTERNATE PROVISIONS FOR REINFORCED AND PRESTRESSED CONCRETE FLEXURAL AND COMPRESSION MEMBERS

2006 Code indicates technical changes in the following sections:

- New Appendix B.

The aforementioned changes were due to:

- The new Appendix B contains the provisions that were removed from Chapters 8 and 10 of ACI 350-01.

APPENDIX C — ALTERNATIVE LOAD FACTORS, STRENGTH REDUCTION FACTORS, AND DISTRIBUTION OF FLEXURAL REINFORCEMENT

2006 Code indicates technical changes in the following sections:

- New Appendix C.

The aforementioned changes were due to:

- Permit the continued use of the load factors, strength reduction factors, and distribution of flexure reinforcement that were formerly in Chapter 9 of ACI 350-01.

APPENDIX D — ANCHORING TO CONCRETE

2006 Code indicates technical changes in the following sections:

- New Appendix D

The aforementioned changes were due to:

- Basing it on Appendix D of the 318-02 Code with one change made to Section RD.2.2. This change, which was the addition of two sentences at the end of the section, concerned post-installed anchors in locations subject to being submerged.

APPENDIX G — CIRCULAR WIRE AND STRAND WRAPPED PRESTRESSED CONCRETE ENVIRONMENTAL STRUCTURES

2006 Code and Commentary indicate technical changes in the following sections:

- All of the section changed from F.X.X to G.X.X and RF.X.X to RG.X.X.

The aforementioned changes were due to:

- Code reformatting.

APPENDIX H — SLABS-ON-SOIL

2006 Code and Commentary indicate technical changes in the following sections:

- All of the section changed from G.X.X to H.X.X and RG.X.X to RH.X.X, H3.2, and H4.1.

The aforementioned changes were due to:

- Code reformatting, clarification of membrane slab thickness as a function of bar size and layers of reinforcement in a membrane slab.

APPENDIX I — ALTERNATE DESIGN METHOD

2006 Code indicates technical changes in the following sections:

- I.3.2, I.3.3, and I.7.5.4.4.

The aforementioned changes were due to:

- Revised allowable stress table.
- Increase in allowable stress levels permitted.
- Clarification of spacing for inclined stirrups used for shear reinforcement.