IN-LB	Inch-Pound Units
SI	International System of Units

An ACI Standard

Code Requirements for Environmental Engineering Concrete Structures (ACI 350-20) and Commentary (ACI 350R-20)

Reported by ACI Committee 350

ACI 350-20

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Code Requirements for Environmental Engineering Concrete Structures (ACI 350-20) and Commentary (ACI 350R-20)

An ACI Standard

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PREFACE

The "Code Requirements for Environmental Engineering Concrete Structures" (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 water and wastewater, other liquids, and solid waste. The term "solid waste" as used in the Code encompasses the heterogeneous mass of disposed-of materials, as well as more homogeneous agricultural, industrial, and mineral wastes.

The Code also covers the evaluation of existing environmental engineering concrete structures.

Environmental engineering concrete structures are subject to uniquely different loadings and severe exposure conditions that require more restrictive serviceability requirements and may provide longer service lives than non-environmental structures.

Loadings include normal dead and live loads, earth pressure loads, hydrostatic and hydrodynamic loads, and vibrating equipment loads. Exposures include concentrated chemicals, alternate wetting and drying, high-velocity flowing liquids, and freezing and thawing of saturated concrete. Serviceability requirements include liquid-tightness, gas-tightness, and durability.

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 relevant chemicals, with limited deflections and cracking. This includes minimizing leakage and control over the infiltration of, or contamination to, the environment or groundwater.

The Code presents additional material as well as modified portions of the ACI 318-05, ACI 318-08, and ACI 318-11 building codes that are applicable to environmental engineering concrete structures.

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: drawings and specifications, inspections, materials, concrete quality, mixing and placing, forming, embedded pipes, 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 A.

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 and gas-tightness testing may be found in ACI 350.1.

Keywords: chemical attack; coatings; concrete durability; concrete finishing (fresh concrete); concrete slabs, crack width and spacing; cracking (fracturing); environmental engineering; hydraulic structures; inspection; joints (junctions); joint sealers; liners; 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 materials ratio; water supply; water treatment.

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INTRODUCTION

The Code and Commentary includes excerpts from ACI 318 that are pertinent to ACI 350. The Commentary discusses some of the considerations of ACI Committee 350 in developing this Code. Emphasis is given to the explanation of provisions that may be unfamiliar to users of the standard.

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 Code provisions may be desirable for unusual structures. The Code and 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 adopted by government bodies having the power to regulate building design and construction. Where the Code has been adopted, 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. 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 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 contractual responsibility of the involved parties. General references requiring compliance with ACI 350 in the job specifications should be avoided, as the contractor is rarely in the position of accepting responsibility for architectural and engineering design details. Generally, the drawings, specifications, and contract documents should contain all 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 350.5, "Specifications for Environmental Concrete Structures," are written specifically for use as part of the contract documents for construction.

ACI 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, plant certification programs of the Precast/Prestressed Concrete Institute and the National Ready Mixed Concrete Association, and qualification standards of the American Society of Concrete Contractors. Also available are "Standard Specification for Agencies Engaged in Construction Inspection and/or Testing" (ASTM E329) and "Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation" (ASTM C1077).

Design aids (general concrete design aids are listed in ACI 318-11):

"Rectangular Concrete Tanks," Portland Cement Association, Skokie, IL, 1998, 182 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, eighth edition, 1981, 627 pp. (Presents technical information for the control of concrete construction, including linings for tunnels, impoundments, and canals.)

"Design of Liquid-Containing Concrete Structures for Earthquake Forces," Portland Cement Association, Skokie, IL, 2002, 60 pp. (Presents design examples for designing for hydrodynamic forces.)

"Moments and Reactions for Rectangular Plates" Engineering Monograph No. 27, United States Department of the Interior, Bureau of Reclamation, 1990, 100 pp. (Presents design aids for rectangular plates.)

GENERAL COMMENTARY

Environmental engineering concrete structures are subject to stringent service conditions and should be designed for extended service life expectancy and detailed with care. The quality of concrete is important, and rigorous quality control must be maintained during construction to obtain dense, durable concrete suitable for the expected service conditions.

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 liquid-tight, and where required, gas-tight, with minimal leakage under normal service conditions.

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

a) The concrete mixture is properly proportioned, mixed, placed, consolidated, finished, and 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 or gas 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 or gas-tightness can also be obtained by appropriate use of shrinkage-compensating concrete. However, 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.

Reduced permeability of the concrete is obtained by lowering the water-cementitious materials ratio as low as possible, without sacrificing acceptable workability and consolidation. Permeability decreases dramatically with extended periods of moist curing. In some cases, surface treatments can be an alternative to moist curing. Reduced permeability of the concrete surface can be achieved through the use of smooth forms or by troweling.

Air entrainment increases consolidation, reduces segregation and bleeding, increases workability, and provides resistance to the effect of freezing-and-thawing cycles. Other admixtures, such as water reducers, are useful, as they increase workability and improve consolidation while lowering the water-cementitious materials ratio (w/cm), which can increase strength characteristics. Use of some supplementary cementitious materials can also provide similar benefits. In addition, supplementary cementitious materials can also reduce permeability, increase durability, and extend service life.

Joint design should also account for movement resulting from thermal dimensional changes, differential settlements, and shrinkage strains induced by placement sequencing. Joints that form a barrier to the passage of liquids and gases are required to 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 entire environmental engineering concrete structure as well as all individual members should be in accordance with the Code, which has been adapted from ACI 318. When all relevant loading conditions are considered, the design should provide adequate safety and serviceability, with a service life significantly greater than the service life expected if these structures were designed following the provisions of ACI 318. 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 and stress limits to promote long service life.

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CONTENTS

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CODE

CHAPTER 1—GENERAL REQUIREMENTS

1.1—Scope

1.1.1 Except for primary containment of hazardous materials, this Code, where adopted under the requirements of the legally adopted building code, provides minimum requirements for the design and construction of reinforced concrete elements of environmental engineering concrete structures. In areas without a legally adopted building code, this Code defines minimum acceptable standards for materials, design, and construction practice. This Code also covers

COMMENTARY

CHAPTER R1—GENERAL REQUIREMENTS

R1.1—Scope

The American Concrete Institute **"Code Requirements for Environmental Engineering Concrete Structures (ACI 350-20),"** hereinafter referred to as this Code, provides minimum requirements for environmental engineering concrete design and construction practices.

The 2020 edition of this Code revised the previous code, ACI 350-06. This Code includes in one document the requirements for all reinforced concrete used for environmental engineering structures. This covers the spectrum 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 this Code apply to prestressed concrete except those that are specifically stated to apply to nonprestressed concrete.

Chapter 13 of this Code contains provisions for design and detailing of earthquake-resistant structures. Refer to 1.1.9.

Appendix A of this 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 B of this Code contains provisions for the design of regions near geometrical discontinuities, or abrupt changes in loadings.

Appendix C of this Code contains provisions for reinforcement limits based on $\mathbf{0.75}\rho_b$, determination of the strength reduction factor ϕ, and moment redistribution that have been in the ACI 318 codes for many years, including ACI 318-99. The provisions are applicable to reinforced and prestressed concrete members. When used, the provisions of Appendix C are to be used in their entirety.

Appendix D of this Code permits 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 D are equally acceptable as those based on the body of this Code, provided the provisions of Appendix D are used in their entirety.

Appendix E of this Code contains provisions for anchoring to concrete.

R1.1.1 A hazardous material may be defined as a liquid, solid, gas, or sludge waste that contains properties that are dangerous or potentially harmful to human health or the environment. The Environmental Protection Agency (EPA) listed wastes are organized into three categories under the Resource Conservation and Recovery Act (RCRA): source-specific wastes, generic wastes, and commercial chemical products. Source-specific wastes include sludges and waste-

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CODE

evaluation and condition assessment of environmental engineering concrete structures.

In this Code, the term "concrete" shall also include shotcrete except where specifically indicated otherwise.

The specified concrete compressive strength shall not be less than 4000 psi. No maximum specified compressive strength shall apply unless restricted by a specific Code provision.

COMMENTARY

waters 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 benzene, creosote, mercury, and various pesticides.

Shotcrete is pneumatically placed whereas other types of concrete are typically placed by gravity or pumped.

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 engineering 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.

1.1.1.1 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. For 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.

R1.1.1.1 Environmental engineering concrete structures include but are not limited to tanks, reservoirs, clarifiers, separators, lagoon liners, and secondary containment structures. Also included are hydraulic structures associated with flood control and water supply projects such as stilling basins, channels, portions of power houses, spillway piers, spray walls, training walls, flood walls, intake and outlet structures, flood control tunnels and shafts, energy-dissipating structures such as impact basins, lock walls, guide and guard walls, canal linings, and reinforced sections of concrete gravity dams.

1.1.2 Precast concrete environmental structures designed and constructed in accordance with ASTM standards are not covered in this Code.

R1.1.2 Although precast environmental concrete structures designed and constructed in accordance with ASTM standards are not included in the scope of this Code, the licensed design professional should evaluate whether additional requirements may be necessary to minimize deterioration and to achieve the desired durability and service life of the structure. For example, ASTM C478 allows a maximum *w/cm* of 0.53 and the standard has no criteria that will permit addressing service conditions where the concrete may be exposed to freezing and thawing, harmful chemicals and gases, or harmful sulfates, all of which could accelerate deterioration and reduce durability of the structure and the expected service life.

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

R1.1.3 The American Concrete Institute recommends that this Code be adopted in its entirety; however, it is recognized that when this Code is made a part of a legally adopted general building code, that general building code may modify some provisions of this Code.

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CODE

1.1.4 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.

1.1.5 The provisions of this Code shall govern for tanks, reservoirs, and other reinforced concrete elements of environmental engineering concrete structures. For special structures such as arches, bins and silos, blast-resistant structures, and chimneys, provisions of this Code shall govern where applicable. When an environmental concrete structure may be considered a building structure then it shall be designed to the requirements of this Code.

COMMENTARY

R1.1.5 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 as an administration building. The ACI 350 Code would apply to the environmental structures, while ACI 318 could apply to the administration building. Also, the following ACI publications could apply to other structures.

"Code Requirements for Reinforced Concrete Chimneys (ACI 307-08) and Commentary" by ACI Committee 307. This standard prescribes 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.

"Design Specification for Concrete Silos and Stacking Tubes for Storing Granular Materials (ACI 313-16) and Commentary" reported by ACI Committee 313. This specification provides 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 ACI 313 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 criteria are empirical, but this specification does not preclude the use of more sophisticated methods that give equivalent or better safety and reliability.

ACI 313 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. Appendixes provide recommended minimum values of overpressure and impact factors.

"Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06) and Commentary" reported by ACI Committee 349. This standard provides minimum requirements for design and construction of concrete structures that form part of a nuclear power plant and that have nuclear safety-related functions. This stan-

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dard does not cover concrete reactor vessels and concrete containment structures that are covered by ACI 359.

"Code for Concrete Reactor Vessels and Containments (ACI 359-07)" reported by Joint ACI-ASME Committee 359. This standard provides requirements for the design, construction, and use of concrete reactor vessels and concrete containment structures for nuclear power plants.

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

R1.1.6 The design and installation of concrete piles fully embedded in the ground is regulated by the general building code. For portions of a pile in air or water, or in soil not capable of providing adequate lateral restraint throughout the length of the pile to prevent buckling, the design provisions of this Code govern where applicable.

Recommendations for concrete piles are presented in detail in **"Guide to Design, Manufacture, and Installation of Concrete Piles (ACI 543R-12)"** reported by ACI Committee 543. This report provides recommendations for the design and use of most types of concrete piles for many kinds of construction.

Recommendations for drilled piers are presented in detail in **"Design and Construction of Drilled Piers (ACI 336.3R-93(06))"** reported by ACI Committee 336. This document 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 presented in **"Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling"** prepared by the PCI Committee on Prestressed Concrete Piling (1993).

1.1.7 This Code governs the design and construction of both structural and non-structural slabs-on-ground for environmental engineering concrete structures.

R1.1.7 Certain requirements that are only applicable to nonstructural liquid-containing slabs-on-ground are included in Chapter 22. These types of tank floor slabs frequently transfer the loads from liquid contents directly to the soil below. Otherwise, requirements for all slabs-on-ground are included in other chapters of this Code. An example of an environmental nonstructural, non-liquid-containing slab-on-ground is the floor of a transfer station or similar slab where enhanced durability is essential.

1.1.8 *Concrete on steel deck*

R1.1.8 *Concrete on steel deck*

1.1.8.1 Design and construction of structural concrete slabs cast on stay-in-place, non-composite steel form deck are governed by this Code.

R1.1.8.1 In its most basic application, the steel form deck serves as a form, and the concrete slab is designed to carry all loads, while in other applications the concrete slab may be designed to carry only superimposed loads. The design of the steel deck for this application is described in **"Standard for Non-Composite Steel Floor Deck (ANSI/SDI NC-2010)."** This standard refers to ACI 318 for the design and construction of the structural concrete slab.

The licensed design professional should consider the potential for corrosion of steel deck and make the neces-

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CODE

COMMENTARY

sary provisions for corrosion resistance. Corrosion losses of form deck do not directly affect the strength of the slab; however, such corrosion may be undesirable for reasons such as aesthetics, or the possibility of corrosion products dislodging from the underside of a slab. The presence of steel deck does not allow visual inspection of the underside of the concrete slab.

1.1.8.2 This Code does not govern the design of stay-in-place, composite steel form deck. However, the concrete design and construction shall be governed by this Code, where applicable.

R1.1.8.2 Another type of steel 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 and construction of composite slabs on steel deck is described in **"Standard for Composite Steel Floor Deck (ANSI/ASCE DI C1.0-2006)."** The standard refers to the appropriate portions of ACI 318 for the design and construction of the concrete portion of the composite assembly. International Building Code (IBC 2006) also provides guidance for design of composite slabs on steel deck. The design of negative moment reinforcement to create continuity at supports is a common example where a portion of the slab is designed in conformance with this Code. Corrosion losses of composite deck may result in reduced strength of the slab.

1.1.9 *Provisions for earthquake resistance*

R1.1.9 *Provisions for earthquake resistance*

1.1.9.1 The Seismic Design Category of a structure shall be determined in accordance with the legally adopted building code, or determined by other authority having jurisdiction in areas without a legally adopted building code.

R1.1.9.1 Design requirements for an earthquake-resistant structure in this Code are determined by the Seismic Design Category (SDC) to which the structure is assigned. In general, the SDC relates to seismic hazard level, soil type, occupancy, and use of the structure. Assignment of a structure to an SDC is under the jurisdiction of a general building code rather than ACI 350.

Seismic Design Categories in this Code are adopted directly from ASCE/SEI 7-10. Similar designations are used by the 2009 edition of the **"International Building Code (IBC 2009)"** and the **"Building Construction and Safety Code (NFPA 5000:2009)."** The **"BOCA National Building Code, 13th edition"** and **"Standard Building Code (SDC 1996)"** use Seismic Performance Categories. The 1997 **"Uniform Building Code (UBC 1997)"** relates design requirements for earthquake resistance to seismic zones, whereas previous editions of ACI 350 related design requirements for earthquake resistance to seismic risk levels. Table R1.1.9.1 correlates Seismic Design Categories to the low, moderate/intermediate, and high seismic risk terminology used in ACI 318 for several editions before the 2008 edition, and to the various methods of assigning design requirements in use in the United States under the various model building codes, the ASCE/SEI 7 standard and the NEHRP Recommended Provisions (FEMA 450-2003).

In the absence of a general building code that prescribes earthquake loads and seismic zoning, it is the intent of ACI Committee 350 that application of provisions for earthquake-resistant design be consistent with national standards

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CODE | COMMENTARY

Table R1.1.9.1—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 this Code		
ACI 350-20, ACI 318-11; IBC 2000, 2003, 2006, 2009; NFPA 5000, 2003, 2006, 2009; ASCE 7-98, 7-02, 7-05, 7-10; NEHRP 1997, 2000, 2003, 2009	SDC* A, B	SDC C	SDC D, E, F
ACI 350-06 and 350.3-06 and previous editions ACI 318-05 and previous editions	Low seismic risk	Moderate/intermediate seismic risk	High seismic risk
BOCA National Building Code 1993, 1996, 1999; Standard Building Code 1994, 1997, 1999; ASCE 7-93, 7-95; NEHRP 1991, 1994	SPC† A, B	SPC C	SPC D, E
Uniform Building Code 1991, 1994, 1997	Seismic Zone 0, 1	Seismic Zone 2	Seismic Zone 3, 4

*SDC = Seismic Design Category as defined in code, standard, or resource document.

†SPC = Seismic Performance Category as defined in code, standard, or resource document.

or model building codes such as ASCE/SEI 7-10, IBC 2006, and NFPA 5000:2009. The model building codes also specify overstrength factors, $\boldsymbol{\Omega_o}$, that are related to the seismic-force-resisting system used for the structure and used for the design of certain elements.

1.1.9.2 Applicable provisions based on Seismic Design Category are defined in Chapter 13 for structures not otherwise exempted by the legally adopted general building code.

1.1.10 For prestressed concrete environmental structures, Chapters 1 through 22 cover prestressing in general.

1.2—Contract documents

1.2.1 Design drawings, typical details, and specifications for all concrete construction shall bear the dated signature and seal of a licensed design professional. These drawings, details, and specifications shall indicate:

(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 concrete elements and reinforcement

(f) Requirements for type, size, location, and installation of anchors, and qualifications for post-installed anchor installers as required by E.9

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

(h) Magnitude and location of prestressing forces

(i) Anchorage of reinforcement and location and length of lap splices

(j) Type and location of mechanical and welded splices of reinforcement

R1.2—Contract documents

R1.2.1 The provisions for preparation of contract documents are, in general, consistent with those of most general building codes and are intended as supplements.

This Code lists some of the more important items of information that are to be included in the contract documents. This Code does not imply an all-inclusive list, and additional items may be required by the building official.

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(k) The design liquid level for each structure designed to contain liquid

(l) Minimum concrete compressive strength at time of post-tensioning

(m) Stressing sequence for post-tensioning tendons

(n) Statement if slab-on-ground is designed as a structural diaphragm; refer to 13.12.3.4

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

(p) Concrete properties and ingredients including type of cement, type of supplementary cementitious materials, maximum permitted water-cementitious materials ratio, and, if permitted, admixtures and additives

(q) Additional requirements, such as limitations on drying shrinkage

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

(s) Where Eq. (9.8) is used in the design: all limitations on tightness testing, future adjacent construction, operation, and maintenance required to ensure that H will continue to act simultaneously with F

(t) Details and locations of all joints

1.2.2 Calculations pertinent to design shall be filed with the contract documents when required by the building official:

(a) Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted.

(b) Model analyses shall be permitted to supplement calculations.

(c) Building official shall mean the officer or other designated authority charged with the administration and enforcement of this Code, or his duly authorized representative.

COMMENTARY

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.

This 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 a licensed design professional having experience in this technique.

"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 general 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.

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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.

1.3.2 The inspector shall observe the work for compliance with the contract documents. Unless specified otherwise in the legally adopted general building code, inspection records shall include:

COMMENTARY

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 necessary to observe that construction is in accordance with the contract documents. Proper performance of the structure depends on construction that accurately represents the design and meets Code requirements within the tolerances permitted. Qualification of the inspectors can be obtained from a certification program, such as the ACI Certification Program for Concrete Construction Special Inspector.

R1.3.1 Inspection of construction by or under the supervision of the licensed design professional responsible for the design is 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 qualification by becoming certified to inspect and record the results of concrete construction, including preplacement, placement, and post-placement operations through the ACI Inspector Certification Program: Concrete Construction Special Inspector.

When inspection is done independently of the licensed design professional responsible for the design, it is recommended that a licensed design professional be employed to oversee inspection and observe the work to see that the 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 all parties, including the owner, licensed design professional, inspector, 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 scope of inspection and equipment necessary to properly perform the inspection.

R1.3.2 By "inspection," this 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. The frequency should be at least

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(a) Delivery, placement, and testing reports documenting the quantity, location of placement, fresh concrete tests, strength, and other tests of all concrete mixtures

(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 gas-containing structures

(i) General progress of work

COMMENTARY

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's obligation to follow the contract documents and to provide the designated quality and quantity of materials and workmanship for all work stages. Some of the information regarding designated concrete mixtures on a project is often provided in a preconstruction submittal to the licensed design professional. For instance, concrete mixture ingredients and composition are often described in detail in the submittal and are subsequently identified by a mixture designation (reflected on a delivery ticket). The inspector should be present as frequently as 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.

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

Recommended procedures for organization and conduct of concrete inspection are given in detail in ACI 311.4. This guide 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 (ACI SP-2(07)),"** reported by ACI Committee 311. This manual describes methods of inspecting concrete construction that are generally accepted as good practice. It is intended as a supplement to project specifications and as a guide in matters not covered by project specifications.

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 provided for 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 surface temperature of the concrete. Surface temperatures may be determined by placing temperature sensors in contact with concrete surfaces or between concrete surfaces and covers used for curing, such as insulation blankets or plastic sheeting.

1.3.4 Records of inspection required in 1.3.2 and 1.3.3 shall be preserved by the inspector for at least 2 years after completion of the project.

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

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COMMENTARY

Records of inspection are required to 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 designed in accordance with Chapter 13, 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 licensed design professional of the structural design or under the supervision of a licensed design professional with demonstrated capability for supervising inspection of special moment frames.

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 perform the inspection. Qualifications of inspectors should be determined by the jurisdiction enforcing the general building code.

1.3.6 For thin shell spherical domes with prestressed dome rings designed in accordance with Chapters 19 and 20, continuous inspection of the placement of the prestressed and nonprestressed reinforcement and concrete shall be made by a qualified inspector. The inspector shall be under the supervision of the licensed design professional of the structural design or under the supervision of a licensed design professional with demonstrated capability for supervising inspection of thin shell spherical domes with prestressed dome rings.

R1.3.6 The thin shell spherical domes with prestressed dome rings that meet the tolerances stated in Chapter 12 have typically been designed and constructed by firms that specialize in this type of construction. Special inspection by qualified personnel becomes more critical for contractors without specific experience in the construction of these structural elements. The purpose of this section is to ensure that the construction is properly executed through inspection by personnel who are qualified to perform the inspection. Qualifications of inspectors should be determined by the licensed design professional in responsible charge of the design of the structure.

1.4—Approval of special systems of design or construction

Sponsors of any system of design or construction within the scope of this Code, for which the structural adequacy, durability, serviceability, and service life have 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. 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.

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.

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 this Code.

The provisions of this section do not apply to model tests used to supplement calculations under 1.2.2 or to strength evaluations of existing structures under Chapter 22.

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CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Code notation

The terms in this list are used in this Code and as needed in the Commentary.

a = depth of equivalent rectangular stress block as defined in 10.2.7.1, in., Chapters 10, 12

a_v = shear span, equal to distance from center of concentrated load to either: (a) face of support for continuous or cantilevered members, or (b) center of support for simply supported members, in., Chapter 11, Appendix B

A_b = area of an individual bar or wire, in.^{2}, Chapters 10, 12

A_{brg} = net bearing area of the head of stud, anchor bolt, or headed deformed bar, in.^{2}, Chapter 12, Appendix E

A_c = area of concrete section resisting shear transfer, in.^{2}, Chapters 11, 13

A_c = area of core of spirally reinforced compression member measured to outside diameter of spiral, in.^{2}, Chapters 10, 13

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}, Chapter 19

A_{ch} = cross-sectional area of a structural member measured to the outside edges of transverse reinforcement, in.^{2}, Chapters 10, 13

A_{cp} = area enclosed by outside perimeter of concrete cross section, in.^{2}, see 11.5.1, Chapter 11

A_{cs} = cross-sectional area at one end of a strut in a strut-and-tie model, taken perpendicular to the axis of the strut, in.^{2}, Appendix B

A_{ct} = area of that part of cross section between the flexural tension face and center of gravity of gross section, in.^{2}, Chapter 19

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 13

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COMMENTARY

CHAPTER R2—NOTATION AND DEFINITIONS

R2.1—Commentary notation

The terms used in this list are used in the Commentary, but not in the Code.

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.

The factored prestressing force $\boldsymbol{P_{su}}$ is the product of the load factor (1.2 from Section 9.2.5) and the maximum prestressing force permitted. Under 19.5.1, this is usually overstressing to $\mathbf{0.94}\boldsymbol{f_{py}}$ but not greater than $\mathbf{0.80}\boldsymbol{f_{pu}}$, which is permitted for short periods of time

$$P_{su} = (1.2)(0.80)f_{pu}A_{ps} = 0.96f_{pu}A_{ps}$$

CODE

COMMENTARY

A_{cw} = area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear, in.^{2}, Chapter 13

A_f = area of reinforcement in bracket or corbel resisting factored moment, in.^{2}, see 11.8, Chapter 11

A_g = gross area of concrete 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), see 11.5.1, Chapters 9-11, 13, 15-17, Appendixes C, D

A_h = total area of shear reinforcement parallel to primary tension reinforcement in a corbel or bracket, in.^{2}, see 11.8, Chapter 11

A_j = effective cross-sectional area within a joint in a plane parallel to plane of reinforcement generating shear in the joint, in.^{2}, see 13.7.4.1, Chapter 13

A_ℓ = total area of longitudinal reinforcement to resist torsion, in.^{2}, Chapter 11

$A_{\ell,min}$ = minimum area of longitudinal reinforcement to resist torsion, in.^{2}, see 11.5.5.3, Chapter 11

A_n = area of reinforcement in bracket or corbel resisting tensile force N_{uc}, in.^{2}, see 11.8, Chapter 11

A_{nz} = area of a face of a nodal zone or a section through a nodal zone, in.^{2}, Appendix A

A_{Na} = projected influence area of a single adhesive anchor or group of adhesive anchors, for calculation of bond strength in tension, in.^{2}, see E.5.5.1, Appendix E

A_{Nao} = projected influence area of a single adhesive anchor, for calculation of bond strength in tension if not limited by edge distance or spacing, in.^{2}, see E.5.5.1, Appendix E

A_{Nc} = projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension, in.^{2}, see E.5.2.1, Appendix E

A_{Nco} = projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing, in.^{2}, see E.5.2.1, Appendix E

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 prestressing reinforcement in flexural tension zone, in.^{2}, Chapter 19, Appendix C

A_s = area of nonprestressed longitudinal tension reinforcement, in.^{2}, Chapters 8 to 12, 15, 16, 19, Appendix C

A_s' = area of compression reinforcement, in.^{2}, Chapter 8, 9, 19, Appendix B

A_{sc} = area of primary tension reinforcement in a corbel or bracket, in.^{2}, see 11.8.3.5, Chapter 11

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CODE | COMMENTARY

$A_{se,N}$ = effective cross-sectional area of anchor in tension, in.^{2}, Appendix E

$A_{se,V}$ = effective cross-sectional area of anchor in shear, in.^{2}, Appendix E

A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension b_c, in.^{2}, Chapter 13

A_{si} = total area of surface reinforcement at spacing s_i in the i-th layer crossing a strut, with reinforcement at an angle α_i to the axis of the strut, in.^{2}, Appendix B

$A_{s,min}$ = minimum area of flexural reinforcement, in.^{2}, see 10.5, Chapter 10

A_{st} = total area of nonprestressed longitudinal reinforcement (bars or steel shapes), in.^{2}, Chapters 10, 13

A_{sx} = 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 spacing s, in.^{2}, Chapter 11

A_{tp} = area of prestressing steel in a tie, in.^{2}, Appendix B

A_{tr} = total cross-sectional area of all transverse reinforcement within spacing s that crosses the potential plane of splitting through the reinforcement being developed, in.^{2}, Chapter 12

A_{ts} = area of nonprestressed reinforcement in a tie, in.^{2}, Appendix A

A_v = area of shear reinforcement within spacing s, in.^{2}, Chapters 11, 12, 18

A_{vd} = total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam, in.^{2}, Chapter 13

A_{vf} = area of shear-friction reinforcement, in.^{2}, Chapters 11, 13

A_{vh} = area of shear reinforcement parallel to flexural tension reinforcement within spacing s_2, in.^{2}, Chapter 11

$A_{v,min}$ = minimum area of shear reinforcement within spacing s, in.^{2}, see 11.4.6.3 and 11.4.6.4, Chapter 11

A_{Vc} = projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear, in.^{2}, see E.6.2.1, Appendix E

A_{Vco} = projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness, in.^{2}, see E.6.2.1, Appendix E

A_1 = loaded area, in.^{2}, Chapter 10

A_2 = 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

b = width of compression face of member, in., Chapter 8, 9, 10, 19, Appendix C

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CODE

b_c = cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh}, in., Chapter 13

b_o = perimeter of critical section for shear in slabs and footings, in., see 11.11.1.2, Chapters 11

b_s = width of strut, in., Appendix B

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, wall thickness, or diameter of circular section, in., Chapters 10 to 12, Appendix C

b_1 = dimension of the critical section b_o measured in the direction of the span for which moments are determined, in., Chapter 14

b_2 = dimension of the critical section b_o measured in the direction perpendicular to b_1, in., Chapter 14

c = distance from extreme compression fiber to neutral axis, in., Chapters 9, 10, 13, 15

c_{ac} = critical edge distance required to develop the basic strength as controlled by concrete breakout or bond of a post-installed anchor in tension in uncracked concrete without supplementary reinforcement to control splitting, in., see E.8.6, Appendix E

$c_{a,max}$ = maximum distance from center of an anchor shaft to the edge of concrete, in., Appendix E

$c_{a,min}$ = minimum distance from center of an anchor shaft to the edge of concrete, in., Appendix E

c_{a1} = distance from the center of an anchor shaft to the edge of concrete in one direction, in. If shear is applied to anchor, c_{a1} is taken in the direction of the applied shear. If tension is applied to the anchor, c_{a1} is the minimum edge distance, Appendix E. Where anchors subject to shear are located in narrow sections of limited thickness, see E.6.2.4

c_{a2} = distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, in., Appendix E

c_b = smaller of: (a) the distance from center of a bar or wire to nearest concrete surface, and (b) one-half the center-to-center spacing of bars or wires being developed, in., Chapter 12

c_c = clear cover of reinforcement, in., see 10.6.4, Chapters 10, 19

c_{Na} = projected distance from center of an anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor, in., see E.5.5.1, Appendix E

COMMENTARY

c_{a1}' = limiting value of c_{a1} when anchors are located less than $1.5c_{a1}$ from three or more edges (refer to Fig. RE.6.2.4), Appendix E

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c_t = distance from the interior face of the column to the slab edge measured parallel to c_1, but not exceeding c_1, in., Chapter 13

c_1 = dimension 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, 14,

c_2 = dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to c_1, in., Chapters 11, 14

C = cross-sectional constant to define torsional properties of slab and beam, see 14.6.4.2, Chapter 14

C_m = factor relating actual moment diagram to an equivalent uniform moment diagram, Chapter 10

C_s = average coefficient of shrinkage for the reinforced concrete, in./in. See 9.2.8, Chapter 9

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in., Chapters 8 to 13, 15, 18, 19, Appendix C

d' = distance from extreme compression fiber to centroid of longitudinal compression reinforcement, in., Chapters 9, 11, 19, Appendix D

d_a = outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, in., see E.8.4, Appendix E

d_a' = value substituted for d_a when an oversized anchor is used, in., see E.8.4, Appendix E

d_b = nominal diameter of bar, wire, or prestressing strand, in., Chapters 10, 12, 13

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto, in., Chapter 10

d_p = distance from extreme compression fiber to centroid of prestressing steel, in., Chapters 11,19, Appendix C

d_{pile} = diameter of pile at footing base, in., Chapter 16

d_s = distance from extreme tension fiber to centroid of tension reinforcement, in., Chapter 9

d_t = distance from extreme compression fiber to centroid of extreme layer of longitudinal tension steel, in., Chapters 9, 10, Appendix D

D = dead loads, or related internal moments and forces, Chapters 8, 9, 19, 22, Appendix D

D_T = inside diameter of a circular tank, ft, Chapter 13

e = base of Napierian logarithms, Chapter 19

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 E

e_N' = distance between resultant tension load on a group of anchors loaded in tension and the centroid of the group of anchors loaded in tension, in.; e_N' is always positive, Appendix E

e_V' = distance between resultant shear load on a group of anchors loaded in shear in the same direction, and the centroid of the group of anchors loaded

COMMENTARY

C = compression force acting on a nodal zone, lb, Appendix B

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in shear in the same direction, in.; e_V' is always positive, Appendix E

E = load effects of earthquake, or related internal moments and forces, Chapters 9, 13, Appendix D

E_c = modulus of elasticity of concrete, psi, see 8.5.1, Chapters 8 to 10, 15, 20

E_{cb} = modulus of elasticity of beam concrete, psi, Chapter 14

E_{cs} = modulus of elasticity of slab concrete, psi, Chapter 14

E_p = modulus of elasticity of prestressing steel, psi, see 8.5.3, Chapter 8

E_s = modulus of elasticity of reinforcement and structural steel, psi, see Eq. (10-13) and Eq. (10-14), Chapters 8, 10, 14

EI = flexural stiffness of compression member, in.^{2}-lb, see 10.10.6, Chapter 10

f_c' = specified compressive strength of concrete, psi, Chapters 4, 5, 8-13, 15, 19, 20, Appendixes B to E

$\sqrt{f_c'}$ = square root of specified compressive strength of concrete, psi, Chapters 8, 9, 11, 12, 13, 19, 20, Appendix E

f_{ce} = effective compressive strength of the concrete in a strut or a nodal zone, psi, Chapter 16, Appendix A

f_{ci}' = specified compressive strength of concrete at time of initial prestress, psi, Chapters 12, 19

$\sqrt{f_{ci}'}$ = square root of specified compressive strength of concrete at time of initial prestress, psi, Chapters 13, 19

f_{cr}' = required average compressive strength of concrete used as the basis for selection of concrete proportions, psi, Chapter 5

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi, Chapters 5, 11, 12

$f_{c,tension}$ = service level tensile hoop stress in the transformed section of concrete in walls of nonprestressed circular tanks due to combined shrinkage and applied hoop tension, psi, Chapter 9

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 prestressing steel, psi, Chapter 19

f_g' = specified compressive strength of shotcrete, psi, Chapter 4, 5

f_{gr}' = required average compressive strength of shotcrete used as the basis for selection of shotcrete proportions, psi, Chapter 5

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

COMMENTARY

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

f_g' = specified compressive strength of shotcrete, psi

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

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within the flange, psi (In a composite member, f_{pc} is the 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 19

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} = stress in prestressing steel at nominal flexural strength, psi, Chapters 12, 19

f_{pu} = specified tensile strength of prestressing steel, psi, Chapters 11, 19

f_{py} = specified yield strength of prestressing steel, psi, Chapter 19

f_r = modulus of rupture of concrete, psi, see 9.5.2.3, Chapters 9, 15, 19, Appendix C

f_s = calculated tensile stress in reinforcement at service loads, psi, Chapters 10, 19

f_s' = stress in compression reinforcement under factored loads, psi, Appendix B

f_{se} = effective stress in prestressing steel (after allowance for all prestress losses), psi, Chapters 12, 19, Appendix B

$f_{s,max}$ = maximum allowable tensile stress in reinforcement at service loads, psi, Chapters 9-10

f_t = extreme fiber stress in tension in the precompressed tensile zone calculated at service loads using gross section properties, psi, see 19.3.3, Chapter 19

f_{uta} = specified tensile strength of anchor steel, psi, Appendix E

f_y = specified yield strength of nonprestressed reinforcement, psi, Chapters 3, 9-13, 15, 18-20, Appendixes B to D

f_{ya} = specified yield strength of anchor steel, psi, Appendix E

f_{yt} = specified yield strength f_y of transverse reinforcement, psi, Chapters 9 to 13

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, Appendix D

F_n = nominal strength of a strut, tie, or nodal zone, lb, Appendix B

F_{nn} = nominal strength at face of a nodal zone, lb, Appendix B

F_{ns} = nominal strength of a strut, lb, Appendix B

F_{nt} = nominal strength of a tie, lb, Appendix B

COMMENTARY

f_{si} = stress in the i-th layer of surface reinforcement, psi, Appendix B

f_y = specified yield strength of nonprestressed reinforcement, psi

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F_u = factored force acting in a strut, tie, bearing area, or nodal zone in a strut-and-tie model, lb, Appendix B

h = overall thickness or height of member, in., Chapters 9 to 13, 15, 18, 19, 22, Appendixes B, D

h_a = thickness of member in which an anchor is located, measured parallel to anchor axis, in., Appendix E

h_{ef} = effective embedment depth of anchor, in., see E.1, E.8.5, Appendix E. Where anchors subject to tension are close to three or more edges, see E.5.2.3

h_v = total depth of shearhead cross section, in., Chapter 11

h_w = height of entire wall from base to top, or clear height of wall segment or wall pier considered, in., Chapters 11, 13

h_x = maximum center-to-center horizontal spacing of crossties or hoop legs on all faces of the column, in., Chapter 13

H = loads due to lateral pressure of soil, water in soil, or other materials, or related internal moments and forces, lb, Chapter 9, Appendix D

H_L = depth of stored liquid, ft, Chapter 13

I = moment of inertia of section about centroidal axis, in.^{4}, Chapters 10, 11

I_b = moment of inertia of gross section of beam about centroidal axis, in.^{4}, see 14.6.1.6, Chapter 14

I_{cr} = moment of inertia of cracked section transformed to concrete, in.^{4}, Chapter 9

I_e = effective moment of inertia for computation of deflection, in.^{4}, see 9.5.2.3, Chapter 9

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.^{4}, Chapters 9, 10, 15

I_s = moment of inertia of gross section of slab about centroidal axis defined for calculating α_f and β_t, in.^{4}, Chapter 14

I_{se} = moment of inertia of reinforcement about centroidal axis of member cross section, in.^{4}, Chapter 10

I_{sx} = moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross section, in.^{4}, Chapter 10

k = effective length factor for compression members, Chapters 10, 15

k_c = coefficient for basic concrete breakout strength in tension, Appendix E

k_{cp} = coefficient for pryout strength, Appendix E

COMMENTARY

h_{anc} = dimension of anchorage device or single group of closely spaced devices in the direction of bursting being considered, in., Chapter 19

h'_{ef} = limiting value of h_{ef} when anchors are located less than $1.5h_{ef}$ from three or more edges (see Fig. RE.5.2.3), Appendix E

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K = wobble friction coefficient per foot of tendon, Chapter 19

K_{tr} = transverse reinforcement index, see 12.8.2.3, Chapter 12

ℓ = span length of beam or one-way slab; as defined in 8.7; clear projection of cantilever, in., see 8.9 and 9.5, Chapters 8 and 9

ℓ_a = additional embedment length beyond centerline of support or point of inflection, in., Chapter 12

ℓ_c = length of compression member in a frame, measured center-to-center of the joints in the frame, in., Chapters 10, 15

ℓ_d = development length in tension of deformed bar, deformed wire, plain and deformed welded-wire reinforcement, or pretensioned strand, in., Chapters 12, 13, 20

ℓ_{dc} = development length in compression of deformed bars and deformed wire, in., Chapter 12

ℓ_{dh} = development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter), in., Chapters 12, 13

ℓ_{dt} = development length in tension of headed deformed bar, measured from the critical section to the bearing face of the head, in. Chapter 12

ℓ_e = load-bearing length of anchor for shear, in., see E.6.2.2, Appendix E

ℓ_n = length of clear span measured face-to-face of supports, in., Chapters 8, 10, 11, 13, 14, 17, 19

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, Chapter 9

ℓ_o = length, measured from joint face along axis of structural member, over which special transverse reinforcement must be provided, in., Chapter 13

ℓ_t = span of member under load test, taken as the shorter span for two-way slab systems, in. Span is the smaller of: (a) distance between centers of supports, and (b) clear distance between supports plus thickness h of member. Span for a cantilever shall be taken as twice the distance from face of support to cantilever end, Chapter 22

ℓ_u = unsupported length of compression member, in., see 10.10.1.1, Chapter 10

COMMENTARY

K_t = torsional stiffness of torsional member; moment per unit rotation, see R14.7.5, Chapter 14

K_{05} = coefficient associated with the 5 percent fractile, Appendix E

ℓ_{anc} = length along which anchorage of a tie occurs, in., Appendix B

ℓ_b = width of bearing, in., Appendix B

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ℓ_v = length of shearhead arm from centroid of concentrated load or reaction, in., Chapter 11

ℓ_w = length of entire wall, or length of wall segment or wall pier considered in direction of shear force, in., Chapters 11, 13, 15

ℓ_x = length of prestressing steel element from jacking end to any point x, ft, see Eq. (19-1) and (19-2) Chapter 19

ℓ_1 = length of span in direction that moments are being determined, measured center-to-center of supports, in., Chapter 14

ℓ_2 = length of span in direction perpendicular to ℓ_1, measured center-to-center of supports, in., see 14.6.2.3 and 14.6.2.4, Chapter 14

L = live loads, or related internal moments and forces, Chapters 8, 9, 13, 19, 22, Appendix D

L_r = roof live load, or related internal moments and forces, Chapter 9

L_T = length of rectangular tank (inside dimension) parallel to the design earthquake direction being evaluated, ft, Chapter 13

LBA = pounds of portland cement alkali

M_a = maximum moment in member due to service loads at stage deflection is computed, in.-lb, Chapters 9, 15

M_c = factored moment amplified for the effects of member curvature used for design of compression member, in.-lb, see 10.10.6, Chapter 10

M_{cr} = cracking moment, in.-lb, see 9.5.2.3, Chapters 9, 15

M_{cre} = moment causing flexural cracking at section due to externally applied loads, in.-lb, Chapter 11

M_m = factored moment modified to account for effect of axial compression, in.-lb, see 11.2.2.2, Chapter 11

M_{max} = maximum factored moment at section due to externally applied loads, in.-lb, Chapter 11

M_n = nominal flexural strength at section, in.-lb, Chapters 11, 12, 13, 15, 19,

M_{nb} = nominal flexural strength of beam including slab where in tension, framing into joint, in.-lb, see 13.6.2.2, Chapter 13

M_{nc} = nominal flexural strength of column framing into joint, calculated for factored axial force, consistent with the direction of lateral forces considered, resulting in lowest flexural strength, in.-lb, see 13.6.2.2, Chapter 13

M_o = total factored static moment, in.-lb, Chapter 14

M_p = required plastic moment strength of shearhead cross section, in.-lb, Chapter 11

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 stress in the longitudinal bars of at least

COMMENTARY

M = moment acting on anchor or anchor group, Appendix E

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$1.25f_y$ and a strength reduction factor ϕ of 1.0, in.-lb, Chapter 13

M_s = factored moment due to loads causing appreciable sway, in.-lb, Chapter 10

M_{slab} = portion of slab factored moment balanced by support moment, in.-lb, Chapter 13

M_u = factored moment at section, in.-lb, Chapters 10, 11, 13, 14, 15,

M_{ua} = moment at midheight of wall due to factored lateral and eccentric vertical loads, not including $P\Delta$ effects, in.-lb, Chapter 15

M_v = moment resistance contributed by shearhead reinforcement, in.-lb, Chapter 11

M_1 = smaller factored end moment on a compression member, to be taken as positive if member is bent in single curvature, and 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 sidesway, calculated using a first-order elastic frame analysis, in.-lb, Chapter 10

M_2 = larger factored end moment on compression member. If transverse loading occurs between supports, M_2 is taken as the largest moment occurring in member. Value of M_2 is 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 sidesway, calculated using a first-order elastic frame analysis, in.-lb, Chapter 10

n = number of items, such as strength tests, bars, wires, monostrand anchorage devices, anchors, or shearhead arms, Chapters 5, 11, 12, 19, Appendix E

N_a = nominal bond strength in tension of a single adhesive anchor, lb, see E.5.5.1, Appendix E

N_{ag} = nominal bond strength in tension of a group of adhesive anchors, lb, see E.5.5.1, Appendix E

N_b = basic concrete breakout strength in tension of a single anchor in cracked concrete, lb, see E.5.2.2, Appendix E

N_{ba} = basic bond strength in tension of a single adhesive anchor, lb, see E.5.5.2, Appendix E

COMMENTARY

N = tension force acting on anchor or anchor group, Appendix E

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N_c = resultant tensile force acting on the portion of the concrete cross section that is subjected to tensile stresses due to the combined effects of service loads and effective prestress, lb, Chapter 19

N_{cb} = nominal concrete breakout strength in tension of a single anchor, lb, see E.5.2.1, Appendix E

N_{cbg} = nominal concrete breakout strength in tension of a group of anchors, lb, see E.5.2.1, Appendix E

N_n = nominal strength in tension, lb, Appendix E

N_p = pullout strength in tension of a single anchor in cracked concrete, lb, see E.5.3.4 and E.5.3.5, Appendix E

N_{pn} = nominal pullout strength in tension of a single anchor, lb, see E.5.3.1, Appendix E

N_{sa} = nominal strength of a single anchor or individual anchor in a group of anchors in tension as governed by the steel strength, lb, see E.5.1.1 and E.5.1.2, Appendix E

N_{sb} = side-face blowout strength of a single anchor, lb, Appendix E

N_{sbg} = side-face blowout strength of a group of anchors, lb, Appendix E

N_u = factored axial force normal to cross section occurring simultaneously with V_u or T_u; to be taken as positive for compression and negative for tension, lb, Chapter 11

N_{ua} = factored tensile force applied to anchor or individual anchor in a group of anchors, lb, Appendix E

$N_{ua,g}$ = total factored tensile force applied to anchor group, lb, Appendix E

$N_{ua,i}$ = factored tensile force applied to most highly stressed anchor in a group of anchors, lb, Appendix E

$N_{ua,s}$ = factored sustained tension load, lb, see E.3.5, Appendix E

N_{uc} = factored horizontal 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_b = nominal axial strength at balanced strain conditions, lb, see Appendixes B, C

P_c = critical buckling load, lb, see 10.10.6, Chapter 10

P_n = nominal axial load strength of cross section, lb, Chapters 9, 10, 15

$P_{n,max}$ = maximum allowable value of P_n, lb, see 10.3.6, Chapter 10

P_o = nominal axial strength at zero eccentricity, lb, Chapter 10

P_{pu} = factored prestressing force at anchorage device, lb, Chapter 19

P_{px} = prestressing force evaluated at distance ℓ_{px} from the jacking end, lb, Chapter 19

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P_s = unfactored axial load at the design (midheight) section including effects of self-weight, lb, Chapter 15

P_s = prestressing force at jacking end, lb, Chapter 19

P_u = factored axial force at given eccentricity; to be taken as positive for compression and negative for tension, lb, $\leq \phi P_n$, Chapters 10, 13, 15,

P_x = prestressing force at any point x, lb, Chapter 19

p_{cp} = outside perimeter of concrete cross section, in., see 11.5.1, Chapter 11

p_h = perimeter of centerline of outermost closed transverse torsional reinforcement, in., Chapter 11

q_{Du} = factored dead load per unit area, Chapter 14

q_{Lu} = factored live load per unit area, Chapter 14

q_u = factored load per unit area, Chapter 14

Q = stability index for a story, see 10.10.5, Chapter 10

r = radius of gyration of cross section of a compression member, in., Chapter 10

R = rain load, or related internal moments and forces, Chapter 9

R = numerical coefficient representing the combined effect of the structure's ductility, energy-dissipating capacity, and structural redundancy, Chapter 13

s = center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, prestressing tendons, wires, or anchors, in., Chapters 10 to 13, 18 to 20, 22, Appendix E

s_i = center-to-center spacing of reinforcement in the i-th layer adjacent to the surface of the member, in., Appendix B

s_o = center-to-center spacing of transverse reinforcement within the length ℓ_o, in., Chapter 13

s_s = sample standard deviation, psi, Chapter 5, Appendix E

s_{sk} = spacing of skin reinforcement, in., Chapter 10

s_{sg} = sample standard deviation for shotcrete, psi, Chapter 5

s_2 = center-to-center spacing of longitudinal shear or torsion reinforcement, in., Chapter 11

S = snow load, or related internal moments and forces, Chapters 9, 13

S_d = environmental durability factor, see 9.2.6, Chapter 9, 11, 13

S_e = moment, shear, or axial force at connection corresponding to development of probable strength at intended yield locations, based on the governing mechanism of inelastic lateral deformation, considering both gravity and earthquake effects, Chapter 13

S_n = nominal flexural, shear, or axial strength of connection, Chapter 13

S_y = yield strength of connection, based on f_y, for moment, shear, or axial force, Chapter 13

t = wall thickness of hollow section, in., Chapter 11

COMMENTARY

R = reaction, lb, Appendix B

s_{sg} = sample standard deviation for shotcrete, psi

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T = cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete, Chapter 9, Appendix D

T_{hoop} = service level hoop tension force in the concrete section under consideration, lb, Chapter 9

T_n = nominal torsional moment strength, in.-lb, Chapter 11

T_u = factored torsional moment at section, in.-lb, Chapter 11

U = required strength to resist factored loads or related internal moments and forces, Chapter 9, 13, Appendix D

v_n = nominal shear stress, psi, see 11.11.7.2, Chapters 11, 13

V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete, lb, see E.6.2.2 and E.6.2.3, Appendix E

V_c = nominal shear strength provided by concrete, lb, Chapters 8, 9, 11, 13, 14,

V_{cb} = nominal concrete breakout strength in shear of a single anchor, lb, see E.6.2.1, Appendix E

V_{cbg} = nominal concrete breakout strength in shear of a group of anchors, lb, see E.6.2.1, Appendix E

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 of a single anchor, lb, see E.6.3.1, Appendix E

V_{cpg} = nominal concrete pryout strength of a group of anchors, lb, see E.6.3.1, Appendix E

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from high 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 for load combinations including earthquake effects, lb, see 13.5.4.1 and 13.6.5.1, Chapter 13

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, lb, Chapters 8, 10 to 13, Appendix E

V_{nh} = nominal horizontal shear strength, lb, Chapter 18

V_p = vertical component of effective prestress force at section, lb, Chapter 11

V_s = nominal shear strength provided by shear reinforcement, lb, Chapters 9, 11

V_{sa} = nominal shear strength of a single anchor or individual anchor in a group of anchors as governed

COMMENTARY

T = tension force acting on a nodal zone, lb, Appendix A

V = shear force acting on anchor or anchor group, Appendix E

$V_{\parallel}$ = applied shear parallel to the edge, lb, Appendix E

$V_{\perp}$ = applied shear perpendicular to the edge, lb, Appendix E

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by the steel strength, lb, see E.6.1.1 and E.6.1.2, Appendix E

V_u = factored shear force at section, lb, Chapters 9, 11-14, 17,

V_u = factored horizontal shear in a story, lb, Chapter 10

V_{ua} = factored shear force applied to a single anchor or group of anchors, lb, Appendix E

$V_{ua,g}$ = total factored shear force applied to anchor group, lb, Appendix E

$V_{ua,i}$ = factored shear force applied to most highly stressed anchor in a group of anchors, lb, Appendix E

V_{ug} = factored shear force on the slab critical section for two-way action due to gravity loads, lb, see 13.13.6 Chapter 13

V_{us} = factored horizontal shear in a story, lb, Chapter 10

w_c = density (unit weight) of normalweight concrete or equilibrium density of lightweight concrete, lb/ft^3, Chapters 8, 9

w_u = factored load per unit length of beam or one-way slab, Chapter 8

W = wind load, or related internal moments and forces, Chapter 9, Appendix D

x = shorter overall dimension of rectangular part of cross section, in., Chapter 14

y = longer overall dimension of rectangular part of cross section, in., Chapter 14

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension face, in., Chapters 9, 11

α = angle defining the orientation of reinforcement, Chapters 11, 13, Appendix B

α_c = coefficient defining the relative contribution of concrete strength to nominal wall shear strength, see 13.9.4.1, Chapter 13

α_f = 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, see 14.6.1.6, Chapters 9, 14

α_{fm} = average value of α_f for all beams on edges of a panel, Chapter 9

α_{f1} = α_f in direction of ℓ_1, Chapter 14

α_{f2} = α_f in direction of ℓ_2, Chapter 14

α_i = angle between the axis of a strut and the bars in the i-th layer of reinforcement crossing that strut, Appendix B

COMMENTARY

w_s = width of a strut perpendicular to the axis of the strut, in., Appendix B

w_t = effective height of concrete concentric with a tie, used to dimension nodal zone, in., Appendix B

$w_{t,max}$ = maximum effective height of concrete concentric with a tie, in., Appendix A

W_a = service-level wind load, refer to R15.8.4

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α_{px} = total angular change of prestressing tendon profile in radians from tendon jacking end to any point x, Chapter 19

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 the surrounding composite slab section, see 11.11.4.5, Chapter 11

β = ratio of long to short dimensions: clear spans for two-way slabs, see 9.5.3.3; sides of column, concentrated load or reaction area, see 11.11.2.1; or sides of a footing, see 16.4.4.2, Chapters 9, 11, 16

β = ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement, Chapter 10

β_b = ratio of area of reinforcement cut off to total area of tension reinforcement at section, Chapter 12

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, Chapter 10

β_{dns} = ratio used to account for reduction of stiffness of columns due to sustained axial loads, see 10.10.6.2, Chapter 10

β_{ds} = ratio used to account for reduction of stiffness of columns due to sustained lateral loads, see 10.10.4.2, Chapter 10

β_n = factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone, Appendix B

β_p = factor used to compute V_c in prestressed slabs, Chapter 11

β_s = factor to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete in a strut, Appendix B

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, see 14.6.4.2, Chapter 14

β_1 = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, see 10.2.7.3, Chapters 10, 19, Appendix C

γ_f = factor used to determine the unbalanced moment transferred by flexure at slab-column connections, see 14.5.3.2, Chapters 11, 13, 14

γ_p = factor for type of prestressing steel, see 19.7.2, Chapter 19

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γ_s = factor used to determine the portion of reinforcement located in center band of footing, see 16.4.4.2, Chapter 16

γ_v = factor used to determine the unbalanced moment transferred by eccentricity of shear at slab-column connections, see 11.11.7.1, Chapter 11

δ = moment magnification factor to reflect effects of member curvature between ends of compression member, 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 13

Δ_{cr} = computed, out-of-plane deflection at midheight of wall corresponding to cracking moment M_{cr}, in., Chapter 15

Δ_n = computed, out-of-plane deflection at midheight of wall corresponding to nominal flexural strength M_n, in., Chapter 15

Δ_o = relative lateral deflection between the top and bottom of a story due to lateral forces computed using a first-order elastic frame analysis and stiffness values satisfying 10.11.1, in., Chapter 10

Δ_r = difference between initial and final (after load removal) deflections for load test or repeat load test, in., Chapter 22

Δ_s = computed, out-of-plane deflection at midheight of wall due to service loads, in., Chapter 15

Δ_u = computed deflection at midheight of wall due to factored loads, in., Chapter 15

Δ_1 = measured maximum deflection during first load test, in., see 22.5.2, Chapter 22

Δ_2 = maximum deflection measured during second load test relative to the position of the structure at the beginning of second load test, in., see 22.5.2, Chapter 22

Δf_p = increase in stress in prestressing steel due to factored loads, psi, Appendix B

Δf_{ps} = stress in prestressing steel at service loads less decompression stress, psi, Chapter 19

Δf_{pt} = f_{ps} at the section of maximum moment minus the stress in the prestressing steel due to prestressing and factored bending moments at the section under consideration, psi, see R11.5.3.10, Chapter 11

ε_t = net tensile strain in extreme layer of longitudinal tension reinforcement at nominal strength, excluding strains due to effective prestress,

COMMENTARY

Δf_{pt} = f_{ps} at the section of maximum moment minus the stress in the prestressing steel due to prestressing and factored bending moments at the section under consideration, psi, refer to R11.5.3.10, Chapter 11

ε_{cu} = maximum usable strain at extreme concrete compression fiber, Fig. R10.3.3

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creep, shrinkage, and temperature, Chapters 8 to 11, Appendix D

θ = angle between axis of strut, compression diagonal, or compression field and the tension chord of the member, Chapter 11, Appendix B

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normalweight concrete of the same compressive strength, see 8.6.1, 11.6.4.3, 11.2.1, 12.5.2, 12.8.2.4(d), E.3.6, Chapters 9, 11 to 13, 19, 20, and Appendixes B, E

λ_a = modification factor reflecting the reduced mechanical properties of lightweight concrete in certain concrete anchorage applications, see E.3.6, Appendix E

λ_Δ = multiplier for additional long-term deflection due to long-term effects, see 9.5.2.5, Chapter 9

μ = coefficient of friction, see 11.6.4.3, Chapters 11, 13

μ = curvature friction coefficient, Chapter 19

ξ = time-dependent factor for sustained load, see 9.5.2.5, Chapter 9

ρ = ratio of nonprestressed tension reinforcement, A_s/bd, Chapters 9 to 11, 13, 14, 19, Appendix C

ρ' = reinforcement ratio for nonprestressed compression reinforcement, A_s'/bd of A_s' to bd, Chapter 9, 19, Appendix C

ρ_b = reinforcement ratio of A_s to bd producing balanced strain conditions, see 10.3.2, Chapters 9 and 10, Appendix C

ρ_ℓ = ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement, Chapters 11, 13, 15

ρ_p = ratio of A_{ps} to bd_p, Chapter 19

ρ_s = ratio of volume of spiral reinforcement to total volume of core confined by the spiral (measured out-to-out of spirals), Chapters 10, 13

ρ_t = ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement, Chapters 11, 13, 15

ρ_v = ratio of tie reinforcement area to area of contact surface, see 18.5.3.3, Chapter 18

ρ_w = ratio of A_s to b_wd, Chapter 11

ϕ = strength reduction factor, see 9.3, Chapters 8 to 11, 13 to 14, 16 to 20, Appendixes B to E

ϕ_K = stiffness reduction factor, see R10.12.3, Chapter 10

τ_{cr} = characteristic bond stress of adhesive anchor in cracked concrete, psi, see E.5.5.2, Appendix E

τ_{uncr} = characteristic bond stress of adhesive anchor in uncracked concrete, psi, see E.5.5.2, Appendix E

$\psi_{c,N}$ = factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete, see E.5.2.6, Appendix E

COMMENTARY

ϕ_K = stiffness reduction factor, refer to R10.10, Chapter 10

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$\psi_{cp,N}$ = factor used to modify tensile strength of post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, see E.5.2.7, Appendix E

$\psi_{cp,Na}$ = factor used to modify tensile strength of adhesive anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, see E.5.5.5, Appendix E

$\psi_{c,P}$ = factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete, see E.5.3.6, Appendix E

$\psi_{c,V}$ = factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement, see E.6.2.7 for anchors in shear, Appendix E

ψ_e = factor used to modify development length based on reinforcement coating, see 12.8.2.4, Chapter 12

$\psi_{ec,N}$ = factor used to modify tensile strength of anchors based on eccentricity of applied loads, see E.5.2.4, Appendix E

$\psi_{ec,Na}$ = factor used to modify tensile strength of adhesive anchors based on eccentricity of applied loads, see E.5.5.3, Appendix E

$\psi_{ec,V}$ = factor used to modify shear strength of anchors based on eccentricity of applied loads, see E.6.2.5, Appendix E

$\psi_{ed,N}$ = factor used to modify tensile strength of anchors based on proximity to edges of concrete member, see E.5.2.5, Appendix E

$\psi_{ed,Na}$ = factor used to modify tensile strength of adhesive anchors based on proximity to edges of concrete member, see E.5.5.4, Appendix E

$\psi_{ed,V}$ = factor used to modify shear strength of anchors based on proximity to edges of concrete member, see E.6.2.6, Appendix E

$\psi_{h,V}$ = factor used to modify shear strength of anchors located in concrete members with $h_a < 1.5c_{a1}$, see E.6.2.8, Appendix E

ψ_s = factor used to modify development length based on reinforcement size, see 12.8.2.4, Chapter 12

ψ_t = factor used to modify development length based on reinforcement location, see 12.8.2.4, Chapter 12

ψ_w = factor used to modify development length for welded deformed wire reinforcement in tension, see 12.8.7, Chapter 12

ω = tension reinforcement index, see 19.7.2, Chapter 19, Appendix C

ω' = compression reinforcement index, see 19.7.2, Chapter 19, Appendix C

ω_p = prestressing steel index, see 19.8.1, Appendix C

ω_w, ω_{pw},

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ω_w' = reinforcement indexes 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, see 19.8.1, Chapter 19, Appendix C

Ω_o = amplification factor to account for overstrength of the seismic-force-resisting system determined in accordance with the legally adopted general building code, Chapter 13, Appendix E

2.2—Definitions

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

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 meeting the requirements of ASTM C330 and having a loose bulk density of 70 lb/ft^3 or less, determined in accordance with ASTM C29.

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.

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

base of structure—level at which the horizontal earthquake ground motions are assumed to be imparted to an environmental engineering concrete structure. This level does not necessarily coincide with the ground level. Refer to Chapter 13.

basic monostrand anchorage device—anchorage device used with any single strand or a single 5/8 in. or smaller diameter bar that satisfies 19.21.1 and the anchorage device requirements of ACI 423.7.

basic multistrand anchorage device—anchorage device used with multiple strands, bars, or wires, or with single bars

COMMENTARY

R2.2—Definitions

For consistent application of the Code, it is necessary that terms be defined where they have specific 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 "Concrete Terminology" available on the ACI website.

anchorage device—Most anchorage devices for post-tensioning are standard manufactured devices available from commercial sources. In some cases, "special" details or assemblages are developed that combine various wedges and wedge plates for anchoring prestressing steel. These informal designations as standard anchorage devices or special anchorage devices have no direct relation to the Code and AASHTO "Standard Specifications for Highway Bridges" 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—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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larger than 5/8 in. diameter, that satisfies 19.21.1 and the bearing stress and minimum plate stiffness requirements of AASHTO Bridge Specifications, Division I, Articles 9.21.7.2 and 9.21.7.3 of Division I and Article 10.3.2.3 of Division II of AASHTO "Standard Specification for Highway Bridges" (AASHTO 2002).

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

boundary element—portion along structural wall and structural diaphragm edge 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 13.9.6. Refer to Chapter 13.

building official—the officer or other designated authority charged with the administration and enforcement of this Code, or a duly authorized representative.

building official—The term used by many general building codes to identify the person charged with administration and enforcement of provisions of the building code. Such terms as building commissioner or building inspector 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.

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 slag cement.

chemical attack of concrete—deterioration of the concrete matrix due to exposure to aggressive chemicals.

collector element—element that acts in axial tension or compression to transmit earthquake-induced forces between a structural diaphragm and a vertical element of the seismic-force-resisting system. Refer to Chapter 13.

column—member with a ratio of height-to-least lateral dimension exceeding 3 used primarily to support axial compressive load. For a tapered member, the least lateral dimension is the average of the top and bottom dimensions of the smaller side.

column—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, pedestals, and wall piers 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 (refer to 14.4), as well as by the empirical method (refer to 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.

In the ACI 318-08, the definitions for column and pedestal were revised to provide consistency between the definitions.

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composite concrete flexural members—concrete flexural members of precast or cast-in-place concrete elements, or both, constructed in separate placements but so interconnected that all elements respond to loads as a unit.

compression-controlled section—a cross section in which the net tensile strain in the extreme tension steel at nominal strength is less than or equal to the compression-controlled strain limit.

compression-controlled strain limit—the net tensile strain at balanced strain conditions. Refer to 10.3.3.

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

concrete, all-lightweight—lightweight concrete containing only lightweight coarse and fine aggregates that conform to ASTM C330.

concrete, lightweight—concrete containing lightweight aggregate and an equilibrium density, as determined by ASTM C567, between 90 and 115 lb/ft^3.

concrete, normalweight—concrete containing only aggregate that conforms to ASTM C33.

concrete, sand-lightweight—lightweight concrete containing only normalweight fine aggregate that conforms to ASTM C33 and only lightweight aggregate that conforms to ASTM C330.

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 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).

connection—a region that joins two or more members. In Chapter 13, a connection also refers to a region that joins members of which one or more is precast, for which the following more specific definitions apply:

ductile connection—connection that experiences yielding as a result of the earthquake design displacements.

strong connection—connection that remains elastic while adjoining members experience yielding as a result of the earthquake design displacements.

COMMENTARY

concrete, lightweight—In 2000, ASTM C567 adopted "equilibrium density" as the measure for determining compliance with specified in-service density requirements. According to ASTM C567, equilibrium density may be determined by measurement or approximated by calculation using either the measured oven-dry density or the oven-dry density calculated from the mixture proportions. Unless specified otherwise, ASTM C567 requires that equilibrium density be approximated by calculation.

By Code definition, sand-lightweight concrete is structural lightweight concrete with all 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 should be stated, with interpolation when partial sand replacement is used.

concrete, normalweight—Normalweight concrete typically has a density (unit weight) between 135 and 160 lb/ft^3, and is normally taken as 145 to 150 lb/ft^3.

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construction joint—an intentionally created interface between concrete placements. Refer to Chapter 7.

contract documents—documents, including the project drawings and project specifications, covering the required work.

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. Refer to Chapter 7.

convective pressure—the hydrodynamic pressure on a liquid-containing structure during an earthquake, due to the upper (sloshing) portion of its contents. Refer to Chapter 13.

cover, specified concrete—the distance between the outermost surface of embedded reinforcement and the closest outer surface of the concrete indicated in contract documents.

cover, specified concrete—Tolerances on specified concrete cover are provided in 7.5.2.1.

crosstie—a continuous reinforcing bar having a seismic hook at one end and a hook not less than 90 degrees with at least a six-diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90-degree hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end. Refer to Chapter 13.

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

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

deformed reinforcement—Deformed reinforcement is defined as that meeting the deformed reinforcement specifications of 3.5.3.1, or the specifications of 3.5.3.3, 3.5.3.4, 3.5.3.5, 3.5.3.6, or 3.5.3.7. No other reinforcement qualifies. This definition permits accurate statement of anchorage lengths. Bars or wire not meeting the deformation requirements or welded wire reinforcement not meeting the spacing requirements are "plain reinforcement," for Code purposes, and may be used only for spirals.

design displacement—total lateral displacement expected for the design-basis earthquake, as required by the governing code for earthquake-resistant design. Refer to Chapter 13.

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 ASCE/SEI 7 and the International Building Code, 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.

design load combination—combination of factored loads and forces in 9.2.

design story drift ratio—relative difference of design displacement between the top and bottom of a story, divided by the story height. Refer to Chapter 13.

development length—length of embedded reinforcement, including pretensioned strand, required to develop the design strength of reinforcement at a critical section. Refer to 9.3.3 and 12.8.2.

drop panel—a projection below the slab used to reduce the amount of negative reinforcement over a column or the

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CODE

minimum required slab thickness, and to increase the slab shear strength. Refer to 14.2.5 and 14.3.7.

duct—a conduit (plain or corrugated) to accommodate prestressing steel for post-tensioned installation. Requirements for post-tensioning ducts are given in 19.17.

effective depth of section (*d*)—distance measured from extreme compression fiber to centroid of longitudinal tension reinforcement.

effective prestress—stress remaining in prestressing steel after all losses have occurred.

embedment length—length of embedded reinforcement provided beyond a critical section.

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

equilibrium density—density of lightweight concrete after exposure to a relative humidity of 50 ± 5 percent and a temperature of 73.5 ± 3.5°F for a period of time sufficient to reach constant density (refer to ASTM C567).

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

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

headed deformed bars—deformed reinforcing bars with heads that satisfy 3.5.8 attached at one or both ends.

headed shear stud reinforcement—reinforcement consisting of individual headed studs, or groups of studs, with anchorage provided by a head at each end or by a common base rail consisting of a steel plate or shape.

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. Refer to Chapter 13.

impulsive pressures—the hydrodynamic pressure on a liquid-containing structure during an earthquake, due to the lower portion of its contents. Refer to Chapter 13.

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 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.

COMMENTARY

headed deformed bars—The bearing area of a headed deformed bar is, for the most part, perpendicular to the bar axis. In contrast, the bearing area of the head of headed stud reinforcement is a nonplanar spatial surface of revolution, as shown in Fig. R3.5.5. The two types of reinforcement differ in other ways. The shanks of headed studs are smooth, not deformed as with headed deformed bars. The minimum net bearing area of the head of a headed deformed bar is permitted to be as small as four times the bar area. In contrast, the minimum stud head area is not specified in terms of the bearing area, but by the total head area which must be at least ten times the area of the shank.

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joint—portion of structure common to intersecting members. The effective cross-sectional area of a joint of a special moment frame, A_j, for shear strength computations is defined in 13.7.4.1. Refer to Chapters 7 and 13.

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.

lateral-force-resisting system—that portion of the structure composed of members proportioned to resist forces related to wind and earthquake effects. Refer to Chapter 13.

licensed design professional—an individual who is licensed to practice structural design as defined by the statutory requirements of the professional licensing laws of the state or jurisdiction in which the project is to be constructed and who is in responsible charge of the structural design; in other documents, also referred to as registered design professional.

lightweight-aggregate concrete—all-lightweight or sand-lightweight aggregate concrete made with lightweight aggregates conforming to 3.3. Refer to Chapter 13.

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. Refer to Chapter 13.

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. Refer to 8.1.1 and 9.2.

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

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. Refer to 8.5.

moment frame—frame in which members and joints resist forces through flexure, shear, and axial force. Moment frames designated as part of the seismic-force-resisting system shall be categorized as follows:

ordinary moment frame—a cast-in-place or precast concrete frame complying with the requirements of

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COMMENTARY

loads—Numerous definitions for loads are given, as the Code contains requirements that are to 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 1971 ACI 318 Code edition to refer to loads multiplied by the appropriate load factors, was discontinued in the 1977 ACI 318 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 1977 Code, clarifies when the load factors are applied to a particular load, moment, or shear value as used in the Code provisions.

CODE

Chapters 1 through 12 and 17 through 19 and, in the case of ordinary moment frames assigned to Seismic Design Category B, also complying with 13.2.

intermediate moment frame—a cast-in-place frame complying with the requirements of 13.3 in addition to the requirements for ordinary moment frames.

special moment frame—a cast-in-place frame complying with the requirements of 13.1.3 through 13.1.7, 13.5 through 13.7, or a precast frame complying with the requirements of 13.1.3 through 13.1.7 and 13.5 through 13.8. In addition, the requirements for ordinary moment frames shall be satisfied.

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

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

pedestal—member with a ratio of height-to-least lateral dimension less than or equal to 3 used primarily to support axial compressive load. For a tapered member, the least lateral dimension is the average of the top and bottom dimensions of the smaller side.

permeability—the ease with which fluids, both liquids and gases, can enter into, and move through, the concrete.

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. Refer to 3.5.4.

plastic hinge region—length of frame element over which flexural yielding is intended to occur due to earthquake design displacements, extending not less than a distance h from the critical section where flexural yielding initiates. Refer to Chapter 13.

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.

precompressed tensile zone—portion of a prestressed member where flexural tension, calculated using gross section properties, would occur under unfactored dead and live loads if the prestress force was not present.

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

COMMENTARY

pedestal—In ACI 318-08, the definitions for column and pedestal were revised to provide consistency between the definitions.

plain concrete—The presence of reinforcement (nonprestressed or prestressed) does not prohibit the member from being classified as plain concrete, provided all requirements of Chapter 22 are satisfied.

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

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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 Appendixes A through D.

reinforcement—material that conforms to 3.5.

reinforcement, structural—reinforcement provided to resist externally applied forces.

reinforcement, shrinkage and temperature—reinforcement provided to control cracking due to strains resulting from drying and thermal shrinkage and temperature gradients.

reshores—shores placed snugly under a concrete slab or other structural member after the original forms 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.

Seismic Design Category—a classification assigned to a structure based on its risk category and the severity of the design earthquake ground motion at the site, as defined by the legally adopted general building code.

seismic-force-resisting system—portion of the structure designed to resist earthquake design forces required by the legally adopted general building code using the applicable provisions and load combinations.

seismic hook—a hook on a stirrup, or crosstie having a bend not less than 135 degrees, except that circular hoops shall have a bend not less than 90 degrees. Hooks shall have a $6d_b$ (but not less than 3 in.) extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop. Refer to Chapter 13.

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

shear cap—a projection below the slab used to increase the slab shear strength. Refer to 14.2.6.

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—refer to 8.9.

special anchorage device—anchorage device that satisfies 19.15.1 and the standardized acceptance tests of AASHTO "Standard Specifications for Highway Bridges," Division II, Article 10.3.2.3.

special boundary element—boundary element required by 13.9.6.

specified lateral forces—lateral forces corresponding to the appropriate distribution of the design base shear force

COMMENTARY

sheathing—Typically, sheathing is a continuous, seamless, high-density polyethylene material extruded directly on the coated prestressing steel.

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CODE

COMMENTARY

prescribed by the legally adopted general building code for earthquake-resistant design. Refer to Chapter 13.

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 C496 as described in ASTM C330. Refer to 5.1.1.4.

steel fiber-reinforced concrete—concrete containing dispersed randomly oriented steel fibers.

stirrup—reinforcement used to resist shear and torsion stresses in a structural member; typically bars, wires, or welded wire reinforcement 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 transverse reinforcement in flexural members and the term "ties" to transverse reinforcement in compression members.) See also **tie**.

strength, design—nominal strength multiplied by a strength reduction factor ϕ. Refer to 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. Refer to 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. Refer to 9.1.1.

stress—intensity of force per unit area.

structural concrete—all concrete used for structural purposes, including plain and reinforced concrete.

structural diaphragm—structural member, such as a floor or roof slab, that transmits inertial forces acting in the plane of the member to the vertical elements of the seismic-force-resisting system. Refer to Chapter 13 for requirements in the earthquake-resisting structures.

structural truss—assemblage of reinforced concrete members subjected primarily to axial forces.

structural wall—wall proportioned to resist combinations of shears, moments, and axial forces. A shear wall is a structural wall. A structural wall designated as part of the seismic-force-resisting system shall be categorized as follows:

intermediate precast structural wall—a wall complying with all applicable requirements of Chapters 1 through 12, 15, and 17 through 19 in addition to 13.4.

ordinary reinforced concrete structural wall—a wall complying with the requirements of Chapters 1 through 12, 15, and 17 through 19.

special structural wall—a cast-in-place or precast wall complying with the requirements of 13.1.3 through 13.1.7, 13.9, and 13.10, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls.

strut—an element of a structural diaphragm used to provide continuity around an opening in the diaphragm. Refer to Chapter 13.

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CODE

COMMENTARY

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.

tension-controlled section—a cross section in which the net tensile strain in the extreme tension steel at nominal strength is greater than or equal to 0.005.

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. Refer also to **stirrup**.

tie elements—elements that serve to transmit inertia forces and prevent separation of building components such as footings and walls. Refer to Chapter 13.

tightness—resistance to leakage of liquids or gases from one face through the opposite face of an element.

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

transfer length—length of embedded pretensioned strand required to transfer the effective prestress to the concrete.

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.

vertical wall segment—a segment of a structural wall, bounded horizontally by two openings or by an opening and an edge. Wall piers are vertical wall segments.

wall—member, usually vertical, used to enclose or separate spaces.

wall pier—a vertical wall segment within a structural wall, bounded horizontally by two openings or by an opening and an edge, with ratio of horizontal length to wall thickness (ℓ_w/b_w) less than or equal to 6.0, and ratio of clear height to horizontal length (h_w/ℓ_w) greater than or equal to 2.0.

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.

welded wire reinforcement—reinforcing elements consisting of carbon-steel plain or deformed wires, fabricated into sheets or rolls in accordance with ASTM A1064; or reinforcing elements consisting of stainless-steel plain or deformed wires fabricated into sheets or rolls conforming to ASTM A1022.

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

work—the entire construction or separately identifiable parts thereof that are required to be furnished under the contract documents.

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.

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CHAPTER 3—MATERIALS

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 3.10.

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

3.2—Cementitious materials

3.2.1 *Cements*

Cement shall conform to one of the following specifications:

(a) Portland cement: ASTM C150

(b) Blended hydraulic cements: ASTM C595 excluding Type IS (≥70), which is not intended as principal cementing constituents in concrete. ASTM C595 cements that incorporate ASTM C1157 cements are not permitted

(c) Expansive hydraulic cement: ASTM C845

3.2.2 *Supplementary cementitious materials*

Supplementary cementitious materials shall conform to one of the following specifications:

(a) Fly ash and natural pozzolans: ASTM C618

(b) Slag cement: ASTM C989

(c) Silica fume: ASTM C1240

COMMENTARY

CHAPTER 3—MATERIALS

R3.1—Tests of materials

R3.1.3 The record of tests of materials and of concrete are to 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—Cementitious materials

R3.2.1 *Cements*

Type IS (≥70) is a blended cement under ASTM C595 that contains slag cement as an interground component in a quantity equal to or exceeding 70 percent by weight. This was called Type S or Type SA cement in versions of ASTM C595 before 2006.

The limitations for ASTM C1157 cements are based on concerns of durability and the lack of satisfactory performance tests to evaluate ASTM C1157 cement types.

Concrete made with expansive cement can be used to reduce drying-shrinkage cracking in environmental engineering concrete structures, but ACI Committee 350 is not yet in a position to recommend detailed requirements for its use. For the design to be successful, the licensed design professional should recognize the characteristics and properties of shrinkage-compensating concrete and cement as described in ACI 223R and ASTM C845 (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 223R.

All cement(s) should be obtained from the same source(s) as used in the trial mixtures accepted for use on a project. Should the source(s) change, a new set of trial batches should be required to demonstrate compatibility with previously accepted mixtures for strength, color, and other pertinent properties.

R3.2.2 *Supplementary cementitious materials*

Slag cement conforming to ASTM C989 is used as a supplementary cementitious material in concrete in much the same way as fly ash. Generally, it should be used with portland cements conforming to ASTM C150 and only rarely would it be appropriate to use ASTM C989 slag cement with an ASTM C595 blended cement, which already contains a pozzolan or slag cement. Such use with ASTM

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COMMENTARY

C595 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 C989 includes appendixes that discuss effects of slag cement on concrete strength, sulfate resistance, and alkali-aggregate reaction.

Depending on the circumstances, the provision of 3.2.2 may require the same type of supplementary cementitious materials or may require cementitious materials from the same respective sources. The latter would be the case if the sample standard deviation (ASTM C1157) of strength tests used in establishing the required strength margin was based on cementitious materials from a particular source. If the sample standard deviation were based on tests involving supplementary cementitious materials obtained from several sources, the former interpretation would apply. All cementitious materials should be obtained from the same source(s) as used in the trial mixtures approved for use on a project. Should the source(s) change, a new set of trial batches should be required to demonstrate compatibility with previously approved mixtures for strength, color, and other pertinent properties.

3.2.3 Cementitious materials used in the work shall correspond to those used as the basis for selecting concrete mixture proportions. Refer to 5.2.

R3.2.3 Depending on the circumstances, the provision of 3.2.3 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 (ACI 214R) of strength tests used in establishing the required strength margin was based on a cement from a particular source. If the standard deviation were based on tests involving a given type of cement obtained from several sources, the former interpretation would apply. All cement(s) should be obtained from the same source(s) as used in the trial mixtures approved for use on a project. Should the source(s) change, a new set of trial batches should be required to demonstrate compatibility with previously approved mixtures for strength, color, and other pertinent properties.

3.3—Aggregates

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

(a) Normalweight aggregates: ASTM C33

(b) Lightweight aggregates: ASTM C330

Exceptions:

(a) 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.

(b) The use of crushed concrete as aggregate is not permitted.

R3.3—Aggregates

R3.3.1 ASTM C33 includes soundness criteria for concrete aggregates, with testing per ASTM C88. ASTM C33 provides higher permissible limits when magnesium sulfate is used because the test is more severe. Because environmental engineering structures are frequently exposed to extreme environments, the licensed design professional should consider specifying ASTM C88 testing using magnesium sulfate. 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.

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CODE

COMMENTARY

The use of crushed concrete as aggregate may adversely affect the durability of environmental engineering concrete structures. Dependence upon an acceptable service history is not applicable because the source of crushed concrete would be different for the new work. Partially deteriorated concrete used as aggregate may reduce resistance to freezing and thawing, affect air void properties, or degrade during handling, mixing, or placing. Crushed concrete may have constituents that would be susceptible to alkali-aggregate reactivity or sulfate attack in the new concrete or may bring sulfates, chlorides, or organic material to the new concrete in its pore structure.

Wet-mix shotcrete with nonsaturated lightweight aggregate may be difficult to pump or shoot because the aggregate absorbs water, which reduces the plasticity of the mixture. Presaturating the lightweight aggregate before batching reduces loss of pumpability.

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

(a) One-fifth the narrowest dimension between sides of form, nor

(b) One-third the depth of slabs, nor

(c) Three-fourths 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 licensed design professional, workability and methods of consolidation are such that concrete can be placed without honeycomb or voids.

R3.3.2 The size limitations on aggregates are provided to ensure proper encasement of reinforcement and to minimize honeycombing. Note that the limitations on maximum size of the aggregate may be waived if, in the judgment of the licensed design professional, the workability and methods of consolidation of the concrete are such that the concrete can be placed without honeycombs or voids. In this instance, the licensed design professional should decide whether the limitations on maximum size of aggregate may be waived.

3.3.3 Aggregates for shotcrete shall be a blend of sizes as required to produce a combined grading within the limits as indicated in Table 3.3.3. Gradings shall be used as follows:

(a) Grading number 1 should be used for fine aggregate shotcrete

(b) Grading number 2 should be used for all other shotcrete

Aggregates failing to comply with the gradations indicated in Table 3.3.3 may be used if preconstruction testing proves that they will provide satisfactory performance for

Table 3.3.3—Grading limits for combined aggregates

Sieve size, U.S. standard square mesh	Percent by weight passing individual sieves	
	Grading No. 1	Grading No. 2
3/4 in.	—	—
1/2 in.	—	100
3/8 in.	100	90 to 100
No. 4	95 to 100	70 to 85
No. 8	80 to 98	50 to 70
No. 16	50 to 85	35 to 55
No. 30	25 to 60	20 to 35
No. 50	10 to 30	8 to 20
No. 100	2 to 10	

R3.3.3 The particle size distribution of aggregates for in-place shotcrete will be markedly finer than when batched because the larger particles have proportionally larger rebound loss. Rebound losses can cause an approximate 30 percent change in the cement-to-aggregate ratio. For that reason, it is not unusual for a mixture of 1:3 entering the gun to result in a 1:2 mixture in place.

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expected service conditions. Maximum aggregate sizes for shotcrete shall be the lesser of 1/2 in. or one-third the diameter of the material hose.

3.3.4 Impose restrictions on materials to minimize deterioration due to alkali reactivity in accordance with Chapter 4.

3.4—Water

3.4.1 Potable water shall be used in concrete except as modified in this Code.

3.4.2 Mixing water for concrete, including that portion of mixing water contributed in the form of free moisture on aggregates, shall not contain deleterious amounts of chloride ion. Refer to 4.3.

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 mixtures using water from the same source as that to be used in the work.

COMMENTARY

R3.3.4 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. Refer to 4.1.3 and 4.6.1 and corresponding commentary concerning the use of supplementary cementitious materials and lithium nitrate admixtures. On projects where alkali reactivity is a known problem, prescreening of aggregate sources before completing design of the project may be advisable.

Nonreactive aggregates may need to be imported if local aggregates exhibit unacceptable potential reactivity.

R3.4—Water

R3.4.3 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 increase the potential for 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 additives to those that might be contained in the mixing water. These additional amounts should be considered in evaluating the acceptability of the total impurities that may be deleterious to concrete or steel.

ASTM C1602 permits the use of potable water without testing and includes methods for qualifying nonpotable sources of water with consideration of effects on setting time and strength. Testing frequencies are established to ensure continued monitoring of water quality.

Chemical limits for qualifying nonpotable water are indicated in Table 3.4.3.5. The limits indicated are maximums and may need to be specified more stringent for specific projects.

Limits of chloride ions are taken from tables in ACI 318 and ACI 350. Other limit values presented in Table 3.4.3.5 have been taken from Neville (1995).

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CODE

3.4.3.2 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.

3.4.3.3 Nonpotable water shall comply with ASTM C1602 except as modified in this Code.

3.4.3.4 Nonpotable water shall not contain wash water that has been used to wash out concrete mixer drums unless it is accurately measured and complies with ASTM C1602 and 3.4.3.5.

3.4.3.5 Nonpotable water shall comply with the applicable chemical limits indicated in Table 3.4.3.5.

Table 3.4.3.5—Chemical limits for nonpotable mixing water*§

		Limits	Test method
Maximum concentration in nonpotable mixing water, ppm†			
A.	Chlorides as Cl, ppm		
1.	In prestressed concrete, or otherwise designated	500{	#4500‖ or ASTM C114
2.	Other reinforced concrete in moist environments or containing aluminum embedments or dissimilar metals or with stay-in-place galvanized metal forms	1000.	#4500‖ or ASTM C114
B.	Sulfate as SO_4, ppm	1500	ASTM D516 or D4130
C.	Equivalent alkalis as (Na_2O + 0.658 K_2O), ppm	300}	ASTM C114
D.	Total inorganic solids by mass, ppm	5000	ASTM C1603
E. 8.	Organic solids by mass, ppm	w 300	AASHTO T 26
F.	pH	4.0 to 9.0	AASHTO T 26
G	Presence of oil	None to slight	Visual observation

*Limits from this table may be specified as individual items.
†ppm is the abbreviation for parts per million.
§Testing shall be conducted once every 6 months and whenever there is a reason to believe a change has occurred in the characteristics of the source.
‖Method #4500 – Argentometric Method from "Standard Methods for the Examination of Water and Wastewater."

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, steel diaphragm, or steel tubing shall be permitted as specified in this Code.

3.5.2 Welding of reinforcing bars shall conform to AWS D1.4 of the American Welding Society. Type and location of welded splices and other required welding of reinforcing bars shall be indicated in the contract documents. ASTM reinforcing bar specifications, except for ASTM A706, shall be supplemented to require a report of material properties necessary to conform to the requirements in AWS D1.4.

COMMENTARY

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 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 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 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

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CODE

COMMENTARY

carbon equivalents and reinforcing bar sizes. Carbon equivalent is calculated from the chemical composition of the reinforcing bars. The Welding Code has two expressions for calculating carbon equivalent. A relatively short expression, considering only the elements carbon and manganese, is to be used for bars other than ASTM A706 material. A more comprehensive expression is given for ASTM A706 bars. The CE formula in the Welding Code for ASTM A706 bars is identical to the CE formula in the ASTM A706 specification.

The licensed design professional should realize that the chemical analysis, for bars other than ASTM A706, required to calculate the carbon equivalent is not routinely provided by the producer of the reinforcing bars. Hence, for welding reinforcing bars other than ASTM A706 bars, the contract documents should specifically require results of the chemical analysis to be furnished.

The ASTM A706 specification covers low-alloy steel reinforcing bars intended for applications requiring controlled tensile properties or welding. Weldability is accomplished in the ASTM A706 specification by limits or controls on chemical composition and on carbon equivalent (Gustafson and Felder 1991). The producer is required by the ASTM A706 specification to report the chemical composition and carbon equivalent.

The 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 which shows the information required for such a specification for each joint welding procedure.

At times it is 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. 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 ASTM A706 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 ASTM A706 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 should then be performed in accordance with AWS D1.4. It should also be determined if additional precautions are necessary, 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 reinforcement to reinforcing bars or structural steel elements are not covered by AWS D1.4. If welding of this type is required on a project, the licensed design professional 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

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and ductility achieved by the cold-working process (during manufacture) when such wires are heated by welding. These potential concerns are not an issue for machine and resistance welding as used in the manufacture of welded plain and deformed wire reinforcement covered by ASTM A1064.

3.5.3 *Deformed reinforcement*

3.5.3.1 Deformed reinforcing bars shall conform to the requirements for deformed bars in one of the following specifications, except as permitted by 3.5.3.2 and 3.5.3.3:

(a) Carbon steel: ASTM A615

(b) Low-alloy steel: ASTM A706

(c) Stainless steel: ASTM A955

(d) Rail-steel and axle-steel: ASTM A996. Bars from rail-steel shall be Type R.

R3.5.3 *Deformed reinforcement*

R3.5.3.1 ASTM A615 covers deformed carbon 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 A706 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 A615 and A706 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 A996, 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. Type R rail-steel bars are considered a mandatory requirement whenever ASTM A996 is referenced in the Code.

Previous standards ASTM A616 and ASTM A617 have been replaced by ASTM A996.

Stainless steel bars are used in applications where high corrosion resistance or controlled magnetic permeability are required. The physical and mechanical property requirements for stainless steel bars under ASTM A955 are the same as those for carbon-steel bars under ASTM A615. Controlled magnetic permeability refers to the Supplementary Requirements option in ASTM A955. The option is required when the bars need to be nonmagnetic.

3.5.3.2 Deformed reinforcing bars shall conform to one of the ASTM specifications listed in 3.5.3.1, except that for bars with f_y less than 60,000 psi, the yield strength shall be taken as the stress corresponding to a strain of 0.5 percent, and for bars with f_y at least 60,000 psi, the yield strength shall be taken as the stress corresponding to a strain of 0.35 percent. Refer to 9.4.

R3.5.3.2 The ASTM specifications require that yield strength be determined by the offset method (0.2 percent offset) and also include, for bars with f_y at least 60,000 psi, the additional requirement that the stress corresponding to a tensile strain of 0.35 percent be at least f_y. 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. Therefore, the Code defines yield strength in terms of the stress corresponding to a strain of 0.5 percent for f_y less than 60,000 psi and the stress corresponding to a strain of 0.35 percent for f_y at least 60,000 psi.

3.5.3.3 Deformed reinforcing bars conforming to ASTM A1035 shall be permitted to be used as transverse reinforcement in 13.6.4 or spiral reinforcement in 10.9.3.

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3.5.3.4 Bar mats for concrete reinforcement shall conform to ASTM A184. Reinforcing bars used in bar mats shall conform to ASTM A615 or ASTM A706.

3.5.3.5 Deformed wire reinforcement shall conform to ASTM A1064 (carbon steel) or ASTM A1022 (stainless steel), except that wire shall not be smaller than size D-4 or larger than D-31 unless as permitted in 3.5.3.7, and is not permitted in special seismic systems for flexure, axial force, and shrinkage and temperature. For wire with f_y exceeding 60,000 psi, the yield strength shall be taken as the stress corresponding to a strain of 0.35 percent.

3.5.3.6 Welded plain and welded deformed wire reinforcement shall:

(a) Conform to ASTM A1064 (carbon steel) or ASTM A1022 (stainless steel), except that for wire with f_y exceeding 60,000 psi, the yield strength shall be taken as the stress corresponding to a strain of 0.35 percent.

(b) Not be permitted for spirals, and in special seismic systems for: flexure; axial force; shrinkage and temperature; and lateral support of longitudinal bars, concrete confinement, and shear, where the weld is required to resist stresses in response to confinement, lateral support of longitudinal bars, shear, or other actions.

(c) Have welded intersections spaced no farther apart than 12 in. in the direction of calculated stress, except for welded wire reinforcement used as stirrups in accordance with 12.8.13.2.

3.5.3.7 Deformed wire larger than D-31 is permitted when used in welded wire reinforcement conforming to ASTM A1064 (carbon steel) and ASTM A1022 (stainless steel) but shall be treated as plain wire for development and splice design.

3.5.3.8 Zinc-coated (galvanized) reinforcing bars shall conform to ASTM A767. Epoxy-coated reinforcing bars shall conform to ASTM A775 or to ASTM A934. Zinc and epoxy dual-coated reinforcing bars shall conform to ASTM A1055. Bars to be zinc-coated (galvanized), epoxy-coated, or zinc and epoxy dual-coated shall conform to one of the specifications listed in 3.5.3.1.

3.5.3.9 Epoxy-coated wires and welded wire reinforcement shall conform to ASTM A884. Wires to be epoxy-coated shall conform to 3.5.3.5 and welded wire reinforcement to be epoxy-coated shall conform to 3.5.3.6.

COMMENTARY

R3.5.3.5 An upper limit is placed on the size of deformed wire because tests show that D-45 wire will achieve only approximately 60 percent of the bond strength in tension given by Eq. (12.1) (Rutledge and DeVries 2002).

R3.5.3.6 Welded plain wire reinforcement is made of wire conforming to ASTM A1064, which specifies 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.

Welded plain and welded deformed wire reinforcement are not permitted in special seismic systems where the weld is required to resist stresses in response to confinement, lateral support of longitudinal bars, shear, or other actions. This is because ASTM A1064 and A1022 only require the welds to develop 35,000 psi in the interconnected wires. Hoops, stirrups, and other elements used in special seismic systems should have anchorages that are capable of $1.25f_y$ or $1.25f_{yt}$, as applicable, or tensile strength of the bar or wire, whichever is less, so that moderate ductility capacity can be achieved. A welded product that is capable of developing these stress limits could be approved for use through Code Section 1.4.

R3.5.3.7 Welded deformed wire reinforcement is made of wire conforming to ASTM A1064, which specifies 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.

R3.5.3.8 Zinc-coated (galvanized) reinforcing bars (ASTM A767), epoxy-coated reinforcing bars (ASTM A775 and A934), and zinc and epoxy dual-coated reinforcing bars (ASTM A1055) are used in applications where corrosion resistance of reinforcement is of particular concern. They have typically been used in parking structures, bridge structures, and other highly corrosive environments. Zinc-coated (galvanized) reinforcing bars conforming to ASTM A767 are coated using the hot-dipped process.

R3.5.3.9 The use of epoxy-coated reinforcement in shotcrete applications is not recommended due to the abrasive nature of the shotcrete process, especially the dry-mix process. If epoxy-coated reinforcement is desired, a preconstruction mockup should be fabricated and the effect of the shotcrete process on the epoxy coating should be examined

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by washing off the freshly applied shotcrete; coring; or by carefully dissecting the hardened shotcrete and examining the epoxy coating.

3.5.3.10 Zinc-coated (galvanized) welded wire reinforcement shall conform to ASTM A1060. Deformed and plain wires to be zinc-coated and fabricated into welded wire reinforcement shall conform to 3.5.3.5. Welded wire reinforcement to be zinc-coated (galvanized) shall conform to ASTM A1064 and 3.5.3.6. Zinc-coated (galvanized) welded deformed wire reinforcement shall be treated as welded plain wire reinforcement for development and splice design.

R3.5.3.10 Stainless steel wire and welded wire are used in applications where high corrosion resistance or controlled magnetic permeability are required. The physical and mechanical property requirements for deformed stainless-steel wire and deformed and plain welded wire under ASTM A1022 are the same as those for deformed wire, deformed welded wire, and plain welded wire under ASTM A1064.

3.5.4 *Plain reinforcement*

R3.5.4 *Plain reinforcement*

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

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

3.5.4.2 Plain wire for spiral reinforcement shall conform to ASTM A1064, except that for wire with f_y exceeding 60,000 psi, the yield strength shall be taken as the stress corresponding to a strain of 0.35 percent.

3.5.4.3 Plain stainless-steel wire for spiral reinforcement shall conform to ASTM A1022.

3.5.5 *Prestressing reinforcement*

R3.5.5 *Prestressing steel*

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

(a) Strand: ASTM A416

(b) Low-relaxation wire: ASTM A421, including Supplementary Requirement S1 "Low Relaxation Wire and Relaxation Testing"

(c) Wire: ASTM A648, Classes I, II, and III

(d) High-strength bar: ASTM A722

(e) Die-drawn or mechanically tensioned wire: ASTM A821

R3.5.5.1 Because low-relaxation prestressing steel is addressed in a supplementary requirement to ASTM A421, which applies only when low-relaxation material is specified, the appropriate ASTM reference is listed as a separate entity.

3.5.5.2 Wire, strands, and bars not specifically listed in ASTM A416, A421, A722, or A821 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 A416, A421, A722, or A821.

R3.5.5.2 Existing wire-wrapped prestressed concrete tank walls have been constructed using prestressing wire that meets other ASTM standards such as A227 and A648. These standards are not commonly used now for wire-wrapped prestressed concrete tanks. For more information on prestressing wire used in the construction of wire-wrapped prestressed concrete tank walls, refer to ACI 372R.

3.5.5.3 When galvanized wire or strand is used for prestressed reinforcement, the wire or strand shall have a zinc coating of at least 0.85 oz/ft^2 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

R3.5.5.3 A small percentage of wire-wrapped tanks have been constructed with galvanized wire reinforcement. Uncoated steel is generally used for wire-wrapped prestressed reinforcement unless galvanized prestressing wire is specified by the licensed design professional. The

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to be reduced to 0.50 oz/ft^2 of wire surface after stressing. The coating shall meet the requirements for Table 4, Class A coating specified in ASTM A586.

3.5.5.4 Strand for wall-to-footing restraint cables shall be galvanized or epoxy-coated. Galvanized strand shall meet the requirements of ASTM A416, Grade 250 or 270, prior to galvanizing, and ASTM A586, A603, or A475 after galvanizing. Zinc coating shall meet the requirements of ASTM A475 Table 4, Class A, or ASTM A603 Table 2, Class A. Epoxy-coated strand shall meet the requirements of ASTM A416, Grade 250 or 270, with a fusion-bonded epoxy coating and grit impregnated on the surface, conforming to ASTM A882.

3.5.5.5 Mechanical splices for wrapped prestressed reinforcement shall be ferrous material and shall be galvanized when the prestressed reinforcement is galvanized.

3.5.6 *Steel diaphragms*

3.5.6.1 Steel diaphragms, when used as reinforcement in circular prestressed concrete tank walls or other structures, 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, shall conform to ASTM A1008, and shall have a minimum thickness of 0.0179 in. (26 gauge).

3.5.6.2 When galvanized diaphragms are used, hot-dipped galvanized sheet steel shall comply with ASTM A653. Weight of zinc coating shall be not less than G90 of Table 1 of ASTM A653.

3.5.6.3 Steel diaphragm shall be continuous to within 3 in. of the top and bottom of the wall. Horizontal splices are not permitted.

3.5.7 *Structural steel, steel pipe, or tubing*

3.5.7.1 Structural steel used with reinforcing bars in composite compression members meeting requirements of

COMMENTARY

vast majority of strand-wrapped tanks have used galvanized prestressing strand.

R3.5.5.4 Wall-to-footing restraint cables are commonly used in circular tanks to resist tangential movement but permit limited radial movement of the wall. Restraint cables are typically used to limit tangential movement caused by seismic accelerations, differential backfill, and blast pressures.

R3.5.6 *Steel diaphragms*

R3.5.6.1 Steel diaphragms are incorporated into the walls of some types of circular wire-wrapped prestressed concrete tanks to serve as a liquid barrier as well as vertical reinforcement. The diaphragm is fabricated with adjacent and opposing channels as shown in Fig. R3.5.6.1.

Some tanks use steel diaphragms, which are fabricated from galvanized steel. Consideration should be given to the potential interruption of bond due to hydrogen caused by the reaction of portland cement and zinc.

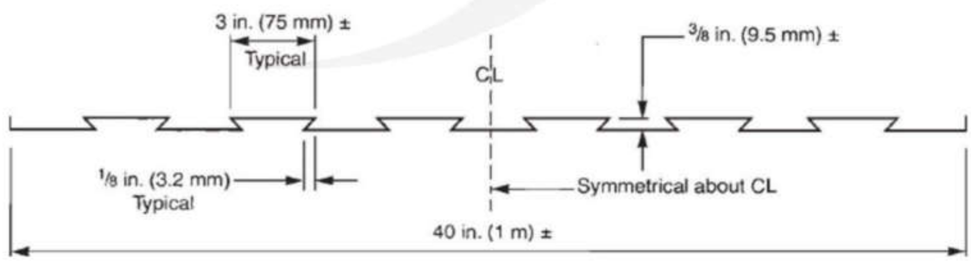

Fig. R3.5.6.1—Example diaphragm sheet.

R3.5.7 *Structural steel, steel pipe, or tubing*

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10.13.7 or 10.13.8 shall conform to one of the following specifications:
(a) Carbon steel: ASTM A36
(b) High-strength low-alloy structural steel: ASTM A242
(c) High-strength low-alloy Columbium-Vanadium steels: ASTM A572
(d) High-strength low-alloy structural steel with 50 ksi: ASTM A588
(e) Structural steel shapes: ASTM A992

3.5.7.2 Steel pipe or tubing for composite compression members composed of a steel-encased concrete core meeting requirements of 10.13.6 shall conform to one of the following specifications:
(a) Black steel, hot-dipped and zinc-coated: Grade B of ASTM A53
(b) Cold-formed welded seamless carbon steel structural tubing: ASTM A500
(c) Hot-formed welded and seamless carbon steel structural tubing: ASTM A501

3.5.8 Headed deformed bars shall conform to ASTM A970 including Annex A1 Requirements for Class HA Head Dimensions.

3.6—Joint accessories

3.6.1 *Waterstops*

3.6.1.1 Waterstops shall be polyvinyl chloride (PVC), thermoplastic elastomeric rubber (TPE-R), or stainless steel. Material for use in the work shall be certified by the manufacturer based on laboratory tests or other tests that will provide confirmation of compliance with the specifications per 3.6.1.2, 3.6.1.3, and 3.6.1.4.

3.6.1.2 Polyvinyl chloride for waterstops shall meet the requirements of CRD-C-572.

3.6.1.3 Thermoplastic elastomeric rubber for waterstops shall meet the requirements of the tensile strength, elongation, and low temperature brittleness requirements of CRD-C-572. Chemical resistance of thermoplastic elastomeric rubber waterstops shall be tested in accordance with ASTM D471 for exposure to the chemical(s) that could potentially contact the waterstop. Physical properties of the waterstop material upon completion of testing shall be at

COMMENTARY

R3.5.7.2 Structural tubing is referred to as Hollow Structural Sections (HSS) in the latest AISC steel specifications.

R3.5.8 The limitation to Class HA head dimensions from Annex A1 of ASTM A970 is due to a lack of test data for headed deformed bars that do not meet Class HA dimensional requirements. Heads not conforming to Class HA limits on bar deformation obstructions and bearing face features could cause unintended splitting forces in the concrete that may not be characteristic of the heads used in the tests that were the basis for 12.6.1 and 12.6.2. For heads conforming to Class HA dimensional requirements, the net bearing area of the head can be assumed to be equal to the gross area of the head minus the area of the bar. This assumption may not be valid for heads not conforming to Class HA dimensional requirements.

R3.6—Joint accessories

R3.6.1.3 Thermoplastic elastomeric rubber waterstop is often used in ozone environments, secondary chemical containment structures, or high service temperature applications where traditional PVC waterstop does not offer suitable chemical or temperature resistance. Chemical resistance of the waterstop should be verified with all chemicals that may come in contact with the waterstop over the life of the structure. ASTM D471 identifies which properties will be observed for change/deterioration—for example, mass,

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least 70 percent of the physical properties of the waterstop material before exposure.

3.6.1.4 Stainless steel for waterstops shall meet the requirements of ASTM A240.

3.6.2 *Bearing pads*

Bearing pads shall consist of neoprene, natural rubber, or polyvinyl chloride.

3.6.2.1 Neoprene bearing pads shall have a minimum ultimate tensile strength of 1500 psi, a minimum elongation of 500 percent (ASTM D412), and a maximum compressive set of 50 percent (ASTM D395, Method A), with a hardness of 30 to 60 durometers (ASTM D2240, Type A Durometer). Neoprene bearing pads shall contain only virgin, crystallization-resistant polychloroprene as the raw polymer, and physical properties shall comply with ASTM D2000, Line Call-Out M2BC4105A14B14.

3.6.2.2 Natural rubber bearing pads shall contain only virgin natural polyisoprene as the raw polymer, and physical properties shall comply with ASTM D2000, Line Call-Out M4AA41413.

3.6.2.3 Polyvinyl chloride for bearing pads shall meet the requirements of CRD-C-572.

3.6.3 *Joint filler*

3.6.3.1 Sponge filler shall be closed-cell neoprene or rubber meeting the requirements of ASTM D1056, Grade 2A1 through Grade 2A4. Minimum grade sponge filler used with cast-in-place concrete walls shall be Grade 2A3.

3.6.3.2 Preformed sponge rubber or cork expansion joint fillers shall meet the requirements of ASTM D1752.

3.6.5 *Sealer and sealant*

3.6.5.1 *Sealants*

Polysulfide and polyurethane sealants shall be an elastomeric compound meeting the requirements of ASTM C920 and shall have permanent characteristics of bond to concrete and 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 suitable for submerged service.

3.6.5.2 *Epoxy sealer*

COMMENTARY

volume, and tensile strength. Testing reflects the concentration, duration, and temperature to which the waterstop will be exposed.

R3.6.1.4 Stainless steel waterstop is often used in ozone environments and extreme service temperature applications where traditional PVC waterstop does not offer suitable chemical or temperature resistance. Chemical resistance of the waterstop should be verified with all chemicals that may come in contact with the waterstop over the life of the structure. Testing reflects the concentration, duration, and temperature to which the waterstop will be exposed.

R3.6.5.1 *Sealants*

Refer to R4.11.3 and R7.5.2.

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Epoxy sealer shall be suitable for bonding to concrete and steel. Epoxy sealer shall conform to the requirements of ASTM C881, Type III, Grade 1, unless other types are approved by the licensed design professional, 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.

3.6.6 Smooth steel dowels used for load transfer in joints shall conform to one of the following specifications:

(a) Carbon structural steel: ASTM A36
(b) Carbon-steel bars: ASTM A615
(c) Carbon hot-wrought steel: ASTM A675
(d) 316 stainless steel: ASTM A276
(e) Low-alloy steel bars: ASTM A706

3.7—Fibers

3.7.1 Synthetic fibers shall not be used as a replacement for structural or shrinkage and temperature reinforcement.

3.7.2 Concrete with microsynthetic fibers shall conform to ASTM C1116. Dosage of microsynthetic fibers shall be based on the minimum cracking reduction ratio (CRR) specified by the licensed design professional, when tested in accordance with ASTM C1579.

COMMENTARY

R3.7—Fibers

Details of microsynthetic and macrosynthetic fibers are provided in ACI 360R, ACI 506.1R, ACI 544.1R, ACI 544.3R, and ACI 544.5R. These fibers are used both in concrete and shotcrete.

The committee consensus is that there is insufficient data on the use of steel fibers in environmental engineering concrete structures covered by ACI 350, where durability is a primary criterion. ACI 318 only permits the use of steel fibers to replace minimum shear reinforcement for specific beams meeting specific criteria and provides the following commentary: "There are no data for the use of steel fibers as shear reinforcement in concrete beams exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources. Where steel fibers are used as shear reinforcement in corrosive environments, corrosion protection should be considered."

R3.7.1 Synthetic fibers may be used in conjunction with structural or shrinkage and temperature reinforcement to improve early-age crack control and post-crack performance. Committee consensus for environmental engineering concrete structures covered by ACI 350 is to not permit synthetic fibers to be used as replacement of structural or shrinkage and temperature reinforcement. Synthetic fibers are not included in ACI 318, and so are not permitted as replacement of structural or shrinkage and temperature reinforcement as defined in that Code.

R3.7.2 Microsynthetic fibers are used to reduce crack width and length propagation due to plastic shrinkage of concrete. These fibers may be monofilament or fibrillated. Microsynthetic fibers are generally shorter than macrosynthetic fibers and are typically used at dosages not exceeding 1.5 lb/yd^3. Concrete with microsynthetic fibers should provide a minimum cracking reduction ratio (CRR) of 40 percent when tested in accordance with ASTM C1579, unless specified otherwise by the licensed design professional.

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3.7.3 Concrete with macrosynthetic fibers shall conform to ASTM C1116. Dosage shall be based on one of the following, as specified by the licensed design professional:

R3.7.3 Macrosynthetic fibers may be used in addition to shrinkage and temperature reinforcement. These fibers are also called "macropolymeric fibers" in some ACI documents. Minimum dosage of macrosynthetic fibers is typically 3 lb/yd^3. The industry is moving toward performance-based specifications for fiber-reinforced concrete that require post-crack flexural performance in accordance with ASTM C1609, in particular. As noted in ASTM C1116 Section 18.2, "When post-crack flexural performance is used as the basis for acceptance of fiber-reinforced concrete, make, condition, and test sets of test specimens in accordance with Test Method, C1550 or C1609 as specified." Typical ranges of the parameters used to quantify the flexural performance of fiber-reinforced concrete, based on industry practice, are provided in the following sections as guidelines to the licensed design professional.

3.7.3.1 Average residual strength (ARS) in accordance with ASTM C1609.

R3.7.3.1 Average residual strength (ARS) from ASTM C1609: 80 to 200 psi.

3.7.3.2 Equivalent flexural strength (f^D_{150}) or equivalent flexural strength ratio ($R^D_{T,150}$) in accordance with ASTM C1609.

R3.7.3.2 Equivalent flexural strength ratio ($R^D_{T,150}$) from ASTM C1609: 20 to 35 percent.

3.7.3.3 Toughness in accordance with ASTM C1550.

R3.7.3.3 Toughness from ASTM C1550: 350 to 450 Joules.

3.8—Admixtures

3.8.1 Admixtures to be used in concrete shall be subject to prior approval by the licensed design professional.

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

3.8.3 Calcium chloride or admixtures containing chloride ions other than from impurities in admixture ingredients shall not be used.

3.8.4 Air-entraining admixtures shall conform to ASTM C260. All air-entraining admixtures that are not vinsol resin or vinsol-resin-based shall be tested in accordance with ASTM C260 at the highest percent air permitted in the project specifications.

R3.8—Admixtures

R3.8.3 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.

R3.8.4 Vinsol-resin-based air-entraining admixtures have been used in the concrete industry since the 1960s. They have performed the function of entraining air in concrete with few problems. The use of non-vinsol resin or non-vinsol resin-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).

Air-entraining admixtures should be used in wet-mix shotcrete that is subjected to cycles of freezing and thawing. In general, air-entraining admixtures are not added to dry-mix shotcrete; however, powder forms of air-entraining admixtures can be added to dry-mix shotcrete.

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3.8.5 Water-reducing admixtures, retarding admixtures, accelerating admixtures, water-reducing and retarding admixtures, and water-reducing and accelerating admixtures shall conform to ASTM C494, and flowing concrete admixtures shall conform to ASTM C1017.

COMMENTARY

R3.8.5 Water-reducing admixtures increase the workability of shotcrete and are used for wet-mix shotcrete. Water-reducing admixtures are not normally used for dry-mix shotcrete.

Chemical set accelerators are used in both dry-mix and wet-mix shotcrete to enhance the maximum buildup thickness by increasing the early stiffness and to reduce the incidence of shotcrete fallouts. The performance, durability, and strength of shotcrete with accelerators should be verified through testing using the provisions in ASTM C1140 prior to their use.

3.8.6 Admixtures used in concrete containing ASTM C845 expansive cements shall be compatible with the cement and produce no deleterious effects.

R3.8.6 The use of admixtures in concrete containing ASTM C845 expansive cements has reduced levels of expansion or increased shrinkage values. Refer to ACI 223R.

3.8.7 Lithium nitrate admixtures for mitigation of alkali-silica reaction shall meet the requirements of ASTM C494, Type S, Specific Performance Admixtures.

R3.8.7 It was discovered in the 1950s that lithium compounds can be used to control expansion due to alkali-silica reaction (Folliard et al. 2003). Recently, there has been increased interest in using lithium-based admixtures. Various lithium-based compounds have been tested and lithium nitrate is found to be the most efficient to control expansion due to alkali-silica reaction. The Federal Highway Administration (FHWA) has published guidelines for the use of lithium nitrate to mitigate or prevent alkali-silica reaction (Folliard et al. 2003; McCoy and Caldwell 1951).

3.8.8 Concrete mixtures containing shrinkage-reducing admixtures shall be tested in accordance with ASTM C157 and shall meet the criteria established by the licensed design professional. Measurements to determine shrinkage, expressed as percentage of base length, shall be made and reported separately for 7, 14, 21, and 28 days of drying after 7 days of moist curing.

R3.8.8 The use of shrinkage-reducing admixtures may not be required for any or all concrete if low-shrinkage material properties are demonstrated in field concrete, extended curing is specified, or if the sections are designed to accommodate the anticipated shrinkage of concrete without the use of shrinkage-reducing admixtures (Folliard and Berke 1997).

3.8.9 Admixtures for inhibiting chloride-induced corrosion shall conform to ASTM C1582.

R3.8.9 Corrosion-inhibiting admixtures can be added to concrete during batching to delay the onset, or reduce the rate, of chloride-induced corrosion on the steel surface, either electrochemically (anodic, cathodic, mixed-inhibitor) or chemically (chemical barrier) to inhibit chloride-induced corrosion. Inorganic chemical compounds that protect steel against chloride attack in a basic pH concrete environment include borates, chromates, molybdates, nitrites, and phosphates. Calcium nitrite is a well-researched inorganic inhibitor that is widely used. Organic compounds used in admixtures to protect steel from chloride-induced corrosion include alkanolamines and an aqueous mixture of amines and fatty-acid esters. The dosage depends on the chloride loading. Refer to ACI 222.3R for additional information.

3.8.10 Permeability-reducing admixtures (PRA) for liquid-containing concrete structures shall be permeability-reducing admixtures for concrete exposed to hydrostatic conditions (PRAH).

R3.8.10 Permeability-reducing admixtures (PRAs), described in ACI 212.3R, have been specified and successfully used in the construction of liquid-containing concrete structures for many years. Their use results in concrete with increased resistance to water penetration. ACI 212.3R classifies permeability-reducing admixtures into two categories:

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COMMENTARY

namely, PRAs for concrete exposed to nonhydrostatic conditions (PRAN) and PRAs for concrete exposed to hydrostatic conditions (PRAH). Only PRAH are recommended for liquid-containing concrete structures.

There is no ASTM method to test the efficacy of a PRA. Several test methods based on other industry standards are recommended in ACI 212.3R to evaluation their effectiveness. The licensed design professional should evaluate data based on CRD-C-48, DIN 1048, or BS EN 12390-8 test methods to verify the performance. For further information, refer to ACI 212.3R.

3.9—Storage of materials

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

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

3.10—Referenced standards

3.10.1 The ASTM standards 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:

ASTM A36/A36M-19—Standard Specification for Carbon Structural Steel

ASTM A53/A53M-20—Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless

ASTM A108-18—Standard Specification for Steel Bar, Carbon and Alloy, Cold-Finished

ASTM A184/A184M-19—Standard Specification for Welded Deformed Steel Bar Mats for Concrete Reinforcement

ASTM A240/A240M-19—Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications

ASTM A242/A242M-13(2018)—Standard Specification for High-Strength Low-Alloy Structural Steel

ASTM A276/276M-17—Standard Specification for Stainless Steel Bars and Shapes

ASTM A307-14$^{\varepsilon 1}$—Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60,000 psi Tensile Strength

ASTM A336/A336-19—Standard Specification for Alloy Steel Forgings for Pressure and High-Temperature Parts

ASTM A416/A416M-18—Standard Specification for Low-Relaxation, Seven-Wire Steel Strand for Prestressed Concrete

ASTM A421/A421M-15—Standard Specification for Stress-Relieved Steel Wire for Prestressed Concrete

ASTM A475-03(2014)—Standard Specification for Zinc-Coated Steel Wire Strand

R3.10—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. However, such a procedure obligates the user of the specification to evaluate if any changes in the later edition are significant in the use of the specification.

Many of the ASTM standards are combined standards as denoted by the dual designation, such as ASTM A36. For simplicity, these combined standards are referenced without the metric (M) designation within the text of the Code and Commentary. In 3.10, however, the complete designation is given because that is the official designation for the standard.

Standard specifications or other material to be legally adopted by reference into a building code are to refer to a specific document. This can be done by simply using the complete serial designation since the first part indicates the subject and the second part the year of adoption. All standards referenced in this Code are listed in 3.10, 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 3.10.

ASTM standards are available from ASTM International, 100 Barr Harbor Drive, West Conshohocken, PS, 19428 (www.astm.org).

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CODE

COMMENTARY

ASTM A500/A500M-20—Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A501/A501M-14—Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing

ASTM A572/A572M-18—Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel

ASTM A586-18—Standard Specification for Metallic-Coated Parallel and Helical Steel Wire Structural Strand

ASTM A588/A588M-19—Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi [345 MPa] Minimum Yield Point, with Atmospheric Corrosion Resistance

ASTM A603-19—Standard Specification for Metallic-Coated Steel Structural Wire Rope

ASTM A615/A615M-20—Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement

ASTM A648-18—Standard Specification for Steel Wire, Hard-Drawn for Prestressed Concrete Pipe

ASTM A653/A653M-20—Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

ASTM A675/675M-14(2019)—Standard Specification for Steel Bars, Carbon, Hot-Wrought, Special Quality, Mechanical Properties

ASTM A706/A706M-16—Standard Specification for Plain Low-Alloy Steel Bars for Concrete Reinforcement

ASTM A722/A722M-18—Standard Specification for High-Strength Steel Bars for Prestressed Concrete

ASTM A767/A767M-19—Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement

ASTM A775/A775M-19—Standard Specification for Epoxy-Coated Steel Reinforcing Bars

ASTM A821/A821M-15—Standard Specification for Steel Wire, Hard-Drawn for Prestressed Concrete Tanks

ASTM A881/A881M-10—Standard Specification for Steel Wire, Deformed, Stress-Relieved or Low-Relaxation for Prestressed Concrete Railroad Ties

ASTM A882/A882M-20—Standard Specification for Filled Epoxy-Coated Seven-Wire Steel Prestressing Strand

ASTM A884/A884M-19—Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement

ASTM A934/A934M-19—Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars

ASTM A955/A955M-20c—Standard Specification for Deformed and Plain Stainless Steel Bars for Concrete Reinforcement

ASTM A970/A970M-18—Standard Specification for Headed Steel Bars for Concrete Reinforcement

ASTM A992/A992M-20—Standard Specification for Structural Steel Shapes

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COMMENTARY

ASTM A996/A996M-16—Standard Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement

ASTM A1008/A1008M-18—Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable

ASTM A1022/A1022M-16b—Standard Specification for Deformed and Plain Stainless Steel Wire and Welded Wire for Concrete Reinforcement

ASTM A1035/A1035M-20—Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement

ASTM A1060/A1060M-16b—Standard Specification for Zinc-Coated (Galvanized) Steel Welded Wire Reinforcement, Plain and Deformed, for Concrete

ASTM A1064/A1064M-18a—Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete

ASTM C29/C29M-17a—Standard Test Method for Bulk Density ("Unit Weight") and Voids in Aggregate

ASTM C31/C31M-19a—Standard Practice for Making and Curing Concrete Test Specimens in the Field

ASTM C33/C33M-18—Standard Specification for Concrete Aggregates

ASTM C39/C39M-20—Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens

ASTM C42/C42M-20—Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

ASTM C94/C94M-20—Standard Specification for Ready-Mixed Concrete

ASTM C114-18—Standard Test Methods for Chemical Analysis of Hydraulic Cement

ASTM C138-17a—Standard Test Method for Density (Unit Weight), Yield and Air Content (Gravimetric) of Concrete

ASTM C144-18—Standard Specification for Aggregate for Masonry Mortar

ASTM C150/C150M-20—Standard Specification for Portland Cement

ASTM C157/C157M-17—Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete

ASTM C172/C172M-17—Standard Practice of Sampling Freshly Mixed Concrete

ASTM C192/C192M-19—Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory

ASTM C231/C231M-17a—Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method

ASTM C260/C260M-10a(2016)—Standard Specification for Air-Entraining Admixtures for Concrete

ASTM C330/C330M-17a—Standard Specification for Lightweight Aggregates for Structural Concrete

ASTM C494/C494M-19—Standard Specification for Chemical Admixtures for Concrete

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COMMENTARY

ASTM C496/C496M-17—Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens

ASTM C567/C567M-19—Standard Test Method for Determining Density of Structural Lightweight Concrete

ASTM C586-19—Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks as Concrete Aggregates (Rock-Cylinder Method)

ASTM C595/C595M-20—Standard Specification for Blended Hydraulic Cements

ASTM C618-19—Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete

ASTM C685/685M-17—Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing

ASTM C779/C779M-19—Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces

ASTM C845/C845M-18—Standard Specification for Expansive Hydraulic Cement

ASTM C881/C881M-20—Standard Specification for Epoxy-Resin-Base Bonding Systems for Concrete

ASTM C920-18—Standard Specification for Elastomeric Joint Sealants

ASTM C989/C989M-18a—Standard Specification for Slag Cement for Use in Concrete and Mortars

ASTM C1012/C1012M-18b—Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution

ASTM C1017/C1017M-13$^{\varepsilon1}$—Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete

ASTM C1064/1064M-17—Standard Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete

ASTM C1077-17—Standard Practice for Agencies Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Testing Agency Evaluation

ASTM C1105-08—Standard Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction

ASTM C1116/C1116M-10a(2015)—Standard Specification for Fiber-Reinforced Concrete

ASTM C1138M-19—Standard Test Method for Abrasion Resistance of Concrete (Underwater Method)

ASTM C1140/C1140-11(2019)—Standard Practice for Preparing and Testing Specimens from Shotcrete Test Panels

ASTM C1152/C1152M-04(2012)$^{\varepsilon1}$—Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete

ASTM C1157/C1157M-20—Standard Performance Specification for Hydraulic Cement

ASTM C1218/C1218M-17—Standard Test Method for Water-Soluble Chloride in Mortar and Concrete

ASTM C1240-20—Standard Specification for Silica Fume Used in Cementitious Mixtures

ASTM C1260-14—Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)

ASTM C1293-20—Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction

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ASTM C1550-19—Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel)

ASTM C1567-13—Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method)

ASTM C1579-13—Standard Test Method for Evaluation of Plastic Shrinkage Cracking of Restrained Fiber Reinforced Concrete (Using a Steel Form Insert)

ASTM C1582/C1582M-11(2017)$^{\varepsilon 1}$—Standard Specification for Admixtures to Inhibit Chloride-Induced Corrosion of Reinforcing Steel in Concrete

ASTM C1602/C1602M-18—Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete

ASTM C1603-16—Standard Test Method for Measurement of Solids in Water

ASTM C1609/C1609M-19a—Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading)

ASTM D395-18—Standard Test Methods for Rubber Property—Compression Set

ASTM D412-16—Standard Test Methods for Vulcanized Rubber and Thermoplastic Elastomers—Tension

ASTM D471-16a—Standard Test Method for Rubber Property—Effect of Liquids

ASTM D516-16—Standard Test Method for Sulfate Ion in Water

ASTM D570-98—Standard Test Method for Water Absorption of Plastics

ASTM D746-20—Standard Test Method for Brittleness Temperature of Plastics and Elastomers by Impact

ASTM D1056-14—Standard Specification for Flexible Cellular Materials—Sponge or Expanded Rubber

ASTM D1171-18—Standard Test Method for Rubber Deterioration—Surface Ozone Cracking Outdoors (Triangular Specimens)

ASTM D1752-18—Standard Specification for Preformed Sponge Rubber, Cork and Recycled PVC Expansion Joint Fillers for Concrete Paving and Structural Construction

ASTM D2000-18—Standard Classification System for Rubber Products in Automotive Applications

ASTM D2240-15$^{\varepsilon 1}$—Standard Test Method for Rubber Property—Durometer Hardness

ASTM D4130-15—Standard Test Method for Sulfate Ion in Brackish Water, Seawater, and Brines

ASTM E96/E96M-16—Standard Test Methods for Water Vapor Transmission of Materials

3.10.2 "Structural Welding Code—Steel Reinforcing Bars" (AWS D1.4/D1.4M:2018) of the American Welding Society is declared to be part of this Code as if fully set forth herein.

COMMENTARY

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3.10.3 Section 2.3 Combining Factored Loads Using Strength Design of "Minimum Design Loads for Buildings and Other Structures" (ASCE/SEI 7-10) is declared to be part of this Code as if fully set forth herein, for the purpose cited in 9.2.4.

3.10.4 "Structural Welding Code—Steel (AWS D1.1/D1.1M:2020)" of the American Welding Society is declared to be part of this Code as if fully set forth herein.

3.10.5 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:

3.10.6.1 *American Water Works Association*
AWWA C 652-19—Disinfection of Water-Storage Facilities

3.10.6.2 *U.S. Army Corps of Engineers*
CRD-C-572-74—U.S. Army Corps of Engineers Specification for Polyvinyl Chloride Waterstop

3.10.6.3 *Federal specifications*
TT-S-00227E (1969)—Sealing Compound: Elastomeric Type, Multi-Component (for Calking, Sealing, and Glazing in Buildings and Other Structures)

3.10.6.4 *American Concrete Institute*
ACI 350.1-10—Specification for Tightness Testing of Environmental Containment Structures and Commentary
ACI 350.3-06—Seismic Design of Liquid-Containing Concrete Structures and Commentary
ACI 355.2-19—Qualification of Post-Installed Mechanical Anchors in Concrete
ACI 355.4-19—Qualification of Post-Installed Adhesive Anchors in Concrete
ACI 374.1-05(14)—Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary
ACI 423.7-14—Specification for Unbonded Single Strand Tendon Materials
ACI 437.2-13—Code Requirements for Load Testing of Existing Concrete Structures and Commentary
ACI 506.2-13—Specification for Shotcrete
ACI ITG-5.1-07—Acceptance Criteria for Special Unbonded Post-Tensioned Precast Structural Walls Based on Validation Testing

COMMENTARY

R3.10.3 ASCE/SEI 7-10 is available from ASCE Book Orders, Box 79404, Baltimore, MD, 21279-0404 (www.asce.org).

R3.10.5 ACI 355.2 is a test method that defines the level of performance required for post-installed anchors. The test method contains requirements for the testing and evaluation of post-installed anchors for both cracked and uncracked concrete applications.

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COMMENTARY

3.10.6.5 *American Association of State Highway and Transportation Officials*

AASHTO T 26-79(2008)—Standard Method of Test for Quality of Water to Be Used in Concrete

AASHTO T 260-97 (2020)—Standard Method of Test for Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials

3.10.6.6 *American Water Works Association*

Standard Methods for the Examination of Water and Wastewater

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COMMENTARY

CHAPTER 4—DURABILITY REQUIREMENTS

CHAPTER 4—DURABILITY REQUIREMENTS

Chapter 4 of ACI 318-11 has been expanded for the ACI 350 Code for environmental engineering 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 concrete cover over the reinforcement. In 2008, the format of Chapter 4 of ACI 318 was revised extensively by introducing exposure categories and classes with applicable durability requirements for concrete in a unified format.

The Code includes provisions for very severe exposures where high resistance to chemical attack, alternate wetting and drying, freezing-and-thawing cycles, erosion, and exposure to the elements are required. It also includes special requirements for concrete cover and spacing of reinforcement (Chapter 12) in such environments.

The Code requires the addition of protection systems in the case of especially severe exposures to acids or other chemicals, and exposures to erosion by abrasion and cavitation. The Code does not include provisions for exposure to high temperatures, and aesthetic considerations such as surface finishes. These items are beyond the scope of the Code and should be covered specifically in the contract documents.

The Code includes the requirements for environmental engineering concrete structures constructed of concrete with properties and characteristics that are suitable for long-term durability. Concrete mixtures using different types of cement, supplementary cementitious materials, and admixtures should demonstrate low permeability, acceptable durability, workability, the ability to be properly consolidated, and finishability characteristics based on the requirements in the Code.

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 other chemical attack.

The Code requires the use of an environmental durability factor S_d when using strength design. Refer to R9.2.6.

4.1—General

4.1.1 Environmental engineering concrete structures shall have a minimum f_c' of 4000 psi and a maximum water-cementitious materials ratio (***w/cm***) of 0.45. The value of f_c' and f_g' shall be the greater of that required for durability in Chapter 4 and that required for strength. Mixture proportioning shall comply with Chapters 4 and 5. Concrete mixtures shall be proportioned to comply with the maximum ***w/cm***, minimum cementitious materials content, and other requirements based on the exposure class assigned to the element in 4.2. All cementitious materials specified in 3.2 and the combinations of these materials shall be included in calculating the ***w/cm*** of the concrete mixture.

R4.1—General

R4.1.1 Because it is difficult to accurately verify the ***w/cm*** of concrete, the specified compressive strength should be reasonably consistent with the ***w/cm*** required for durability. Selection of a specified compressive strength that is consistent with the ***w/cm*** selected for durability will help ensure that the required ***w/cm*** is obtained in the field. Because the usual emphasis during inspection is on compressive strength, test results substantially higher than the specified compressive strength may lead to a lack of concern for quality, and production of concrete that exceeds the maximum ***w/cm***. For example, a minimum specified compressive strength of 4500 psi, consistent with a maximum ***w/cm*** of 0.42, should

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CODE

COMMENTARY

be specified for an environmental structure exposed to freezing and thawing in a moist condition.

4.1.2 Alternative cementitious materials contents to those indicated in Table 4.1.3 may be permitted by the licensed design professional when testing and documentation of the proposed concrete mixture verifies compliance with the Code requirements for durability and workability in addition to the required concrete strength.

Minimum cementitious materials content for shotcrete shall be as follows:

a. Shotcrete in contact with prestressing wires or strands shall consist of one part cementitious materials and not more than three parts fine aggregate by mass.

b. All other shotcrete shall consist of one part cementitious materials and not more than four parts fine aggregate by mass.

4.1.3 When used, supplementary cementitious materials (SCMs) shall comply with the minimum and maximum percent by mass of the cement plus SCMs as indicated in Table 4.1.3, except as provided in the footnotes. When testing is conducted, the results shall verify the proposed percentage of SCMs will provide the durability required by the Code for the expected conditions of service.

R4.1.3 The lower limits on the percentage of supplementary materials that are required for the durability of environmental engineering concrete structures are indicated in Table 4.1.3(a). If higher percentages are proposed, performance testing of the proposed materials to determine their compliance with the Code or proven field performance is required.

In cases where multiple durability issues are of concern, use the higher of the two minimums for that supplementary cementitious material. For example, if corrosion and alkali-silica reaction are of concern and a Class F fly ash is to be used, use a minimum of 25 percent Class F fly ash with alkali content (Na_2O_e) less that 3.0 percent, when the portland cement alkali content is less than 1.00 percent, to control both ASR and reinforcement corrosion.

Table 4.1.3(a) is based on ASTM C1778 with Prevention Level Y, Risk Level 3, Structure Class SC4 (includes water and wastewater treatment facilities), and an Aggregate

Table 4.1.3(a)—Minimum supplementary cementitious materials (SCMs) replacement for alkali-silica reaction (following the prescriptive based approach in 4.6.1.3)*†

Type of supplementary cementitious materials (SCMs)	Alkali content of SCM (%Na_2O_{eq})	Minimum replacement level,‡ percent by mass	
		Portland cement alkali content (%Na_2O_{eq}) < 1.00	Portland cement alkali content (%Na_2O_{eq}) 1.00 to 1.25§
Class F Fly ash‖ (CaO ≤ 18 percent)	< 3.0	25	35
	3.0 to 4.0	30	40
Slag cement	< 1.0	50	65
Silica fume#	< 1.0	1.8 × pounds of portland cement alkali (LBA)	2.5 × pounds of portland cement alkali (LBA)

*All concrete exposed to alkalis in service shall not exceed a maximum portland cement alkali loading of 3.0 lb/yd^3. Refer to Section 4.6.1 for additional requirements for alkali-silica reaction.

‡The use of high levels of SCMs in concrete may increase the risk of scaling in concrete subjected to Exposure Class EF3, if the concrete is not properly proportioned, air-entrained, finished, and cured. Refer to Tables 4.1.3(d) and 4.1.3(e) for upper limits on SCM content for concrete subjected to Exposure Class EF3.

†No prescriptive measures are provided for natural pozzolans, as this class of materials covers a wide variety of pozzolan types with a broad range of properties. Where natural pozzolans are used for mitigation of ASR, the efficacy of the particular aggregate-pozzolan combination shall be determined by performance testing in accordance with 4.6.1.2.1.

§For Na_2O_{eq} > 1.25% testing is required per 4.6.1.2.1.

‖The use of Class C fly ash (CaO greater than 18%) is not permitted for mitigation of alkali-silica reaction unless the concrete mixture is tested in accordance with 4.6.1.2.1.

#The minimum level of silica fume (as a percentage of cementitious material) is calculated on the basis of the alkali (Na_2O_{eq}) loading of the concrete contributed by the portland cement and expressed in units of lb/yd^3 (LBA in Table 4.1.3(a)).

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Table 4.1.3(b)—Sulfate attack[||]

Supplementary cementitious materials (SCMs)*	Minimum percentage of total cementitious materials by mass
Class F fly ash[‡]	15
Class C fly ash	§
Natural pozzolans	§
Silica fume	5
Slag cement	50

Note: Refer to "General footnotes for 4.1.3 tables" for description of this table's footnotes.

Table 4.1.3(c)—Reinforcement corrosion[||]

Supplementary cementitious materials (SCMs)*	Minimum percentage of total cementitious materials by mass
Class F fly ash	15
Class C fly ash	15
Natural pozzolans	15
Silica fume	5
Slag cement	35

Note: Refer to "General footnotes for 4.1.3 tables" for description of this table's footnotes.

Table 4.1.3(d)—Concrete subject to Exposure Class EF3*

Supplementary cementitious materials (SCMs)[#]	Maximum percentage of total cementitious materials by mass[†]
Class F fly ash	25
Class C fly ash	25
Natural pozzolans	25
Silica fume	10
Slag cement	50
Total of fly ash,[‡] other pozzolans and silica fume	35[**]
Total of fly ash,[‡] other pozzolans, slag cement, and silica fume	50[††]

[#]Refer to Table 4.2.a for definition of Exposure Class EF3.

[**]Fly ash and other pozzolans, except silica fume, shall constitute no more than 25 percent, slag cement shall constitute no more than 50 percent, and silica fume shall constitute no more than 10 percent of the total mass of the cementitious materials. The combined supplementary cementitious materials shall not exceed 50 percent of the total mass of the cementitious materials.

[††]Fly ash and other pozzolans shall constitute no more than 25 percent, and silica fume shall constitute no more than 10 percent, of the total mass of the cementitious materials.

Note: Refer to "General footnotes for 4.1.3 tables" for description of this table's footnotes.

COMMENTARY

Reactivity Class of R1 (moderately reactive). ASTM C1778, Table 7, allows a reduction of the minimum SCM replacement percentage when the portland cement alkali content (Na_2O_{eq}) is below 0.70%. The ACI 350 Committee has chosen to be more conservative than ASTM C1778 in not permitting a reduction in the minimum replacement percentage when Na_2O_{eq} is less than 0.70 percent, except with acceptable test results per 4.6.1.2.1.

Section 4.1.3 and Table 4.1.3(d) establish limitations on the amount of fly ash, other pozzolans, silica fume, and slag cement that can be included in concrete exposed to Class EF3 (ASTM C1012; Sivasundaram et al. 1989; Whiting 1989).

For guidance on the use of the various types of supplementary cementitious materials, refer to: ACI 232.1R for natural pozzolans, ACI 232.2R for fly ash, ACI 233R for slag cement, and ACI 234R for silica fume. For other guidance on the use of various types of supplemental cementitious materials, refer to ACI 211.4R. Research conducted concerning the use of Class F fly ash has been reported in the literature (Malhotra and Ramezanianpour 1994; Malhotra and Mehta 2002; Helmuth 1987; Shon et al. 2003; Nixon et al. 1986; Thomas et al. 1992).

Per ASTM C618, Class C fly ash has a calcium oxide content (CaO) greater than 18% and Class F fly ash has a CaO of 18% maximum. The use of fly ashes with CaO greater than 15 percent is not permitted for mitigation of alkali-silica reaction or sulfate attack unless the concrete mixture is tested in accordance with ASTM C1567, ASTM C1293, or ASTM C1012. Typically, Class C fly ash does not bind alkalis as well as Class F fly ash to control alkali-silica reaction, and it also has a reactive calcium aluminate phase that can make it more prone to sulfate attack.

The calcium oxide content of the ASTM C618 fly ashes, when used for the purpose of mitigating alkali-silica reaction and sulfate attack, is limited to a maximum of 15 percent. Research conducted in Canada on cementitious materials including fly ash with various calcium oxide contents has been reported in the literature (Thomas et al. 1999; Shashiprakash and Thomas 1999).

Lithium nitrate admixtures have been found suitable for mitigation of alkali-silica reaction (Thomas et al. 2007;

General footnotes for 4.1.3 tables

Tables 4.1.3(a), (b), (c), and (d) apply when a single SCM is used. Table 4.1.3(e) applies when more than one SCM is used.

*The total cementitious material also includes ASTM C150, C595, and C845 cement. The maximum and minimum percentages indicated above shall include fly ash and other pozzolans, slag cement, and silica fume present in ASTM C595 blended cements per 3.2.1b.

[†]These maximums may be exceeded provided the proposed mixture is tested as follows: in accordance with ASTM C1012 (and complies with Table 4.5) if specific to sulfate exposure; or when the licensed design professional determines that field performance data indicates either that more is required to control sulfate attack or reinforcement corrosion, as applicable, or more is acceptable for concrete in Exposure Class EF3 per 4.2.

[‡]The calcium oxide content of ASTM C618 Class F fly ash shall be limited to 15 percent when used to mitigate sulfate attack, except Class F fly ash with greater than 15% CaO may be used if the proposed mixture is tested in accordance with ASTM C1012, and complies with Table 4.5.

[§]The proposed mixture shall be tested in accordance with ASTM C1012 and shall comply with Table 4.5.

[||]The minimum replacement percentages for the individual SCMs may be reduced from those indicated in Tables 4.1.3(a), (b), and (c) if the sum of the fractional portions of those percentages provided is greater than or equal to one. For example, when silica fume and slag are used together, the silica fume percentage may be reduced to one-third of the minimum silica fume percentage given in the tables, provided that the slag percentage is at least two-thirds of the minimum slag percentage.

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CODE

COMMENTARY

Folliard 2003). The dosage of lithium nitrate admixture is based on the alkali content of the cement in the concrete mixture. For guidance, refer to ACI 212.3R. Refer to AASHTO PP65 for a procedure for calculating a lithium nitrate admixture addition rate.

4.1.4 The maximum ***w/cm*** limits in Chapter 4 do not apply to lightweight concrete.

R4.1.4 Maximum ***w/cm*** is not specified for lightweight-aggregate concrete because determination of the absorption of these aggregates is uncertain, making calculation of ***w/cm*** uncertain. The requirement of a minimum specified strength will ensure the use of a high-quality cement paste. For normalweight-aggregate concrete, requiring both a minimum specified strength and maximum ***w/cm*** provides additional assurance that this objective is met.

4.2—Exposure categories and classes

The licensed design professional shall assign exposure classes based on the severity of the anticipated exposure of environmental concrete elements for each exposure category according to Tables 4.2(a) to (e).

Table 4.2(a)—Exposure Category EF: Freezing-and-thawing exposure

Class	Severity	Condition
EF0	Negligible	Concrete not exposed to freezing-and-thawing cycles
EF1	Moderate	Concrete exposed to freezing-and-thawing cycles and occasional exposure to moisture
EF2	Severe	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture
EF3	Very severe	Concrete exposed to freezing-and-thawing cycles that will be in continuous contact with moisture and exposure to an external source of chlorides in service—from deicing chemicals, salt, seawater, or spray from these sources

Table 4.2(b)—Exposure Category ES: Sulfate exposure

Class	Severity	Water-soluble sulfate (SO_4) in soil, percent by weight[*]	Sulfate (SO_4) in water, ppm[‡]
ES0	Negligible	$SO_4 < 0.10$	$SO_4 < 150$
ES1	Moderate	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 < 1500$ seawater
ES2	Severe	$0.20 \leq SO_4 \leq 2.00$	$1500 \leq SO_4 \leq 10{,}000$
ES3	Very severe	$SO_4 > 2.00$	$SO_4 > 10{,}000$

[*]Using ASTM C1580.

[†]Using ASTM D4130 or D516.

[‡]Sulfate expressed as SO_4 is related to sulfate expressed as SO_3 as given in reports of chemical analysis of portland cements as follows: $SO_3 \times 1.2 = SO_4$

R4.2—Exposure categories and classes

The Code defines exposure conditions of concrete structures in 4.2. Exposure categories are subdivided into exposure classes, depending on the severity of the exposure. A classification of "0" is assigned when the category does not apply, or the exposure is negligible. Associated requirements for concrete relative to the exposure classes are provided in 4.3. Exposure classes from ACI 318 have been changed by indicating they are environmental exposure classes (EXX) because the requirements for environmental engineering concrete structures are more stringent than those in ACI 318.

The Code addresses six exposure categories that affect the requirements of concrete to ensure adequate durability:

Exposure Category EF applies to concrete exposed to moisture, either occasionally or continuously, and cycles of freezing and thawing, with or without exposure to an external source of chlorides in service.

Examples of Class EF1 are exterior walls, beams, girders, and slabs not in direct contact with soil. An example of Class EF2 is a liquid-containment tank.

For additional guidance on exposure to freezing and thawing refer to ACI 362.1R Fig. 3.1, Hershfield (1974), and ASTM C666.

Exposure Category ES applies to concrete in contact with soil or water containing deleterious amounts of water-soluble sulfate ions as defined in Table 4.2(b). Concrete exposed to injurious concentrations of sulfates from soil, water, and wastewater should be made with sulfate-resisting cementitious materials and a low ***w/cm***.

Class ES0 is assigned for conditions where the water-soluble sulfate concentration in contact with concrete is low, and injurious sulfate attack is not a concern. **Classes ES1, ES2, and ES3** are assigned for concrete elements in direct contact with soluble sulfates in soil or water. The severity of exposure increases from Class ES1 to ES3 based on the more critical value of measured water-soluble sulfate concentration in soil or the concentration of dissolved sulfate in water.

Table 4.2(b) lists seawater as moderate exposure, ES1, even though it generally contains more than 1500 ppm SO_4.

Testing for water-soluble sulfate in soil should be conducted in accordance with ASTM C1580. Testing for

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Table 4.2(c)—Exposure Category EC: Conditions requiring corrosion protection of reinforcement

Class	Severity	Condition
EC0	Negligible	Concrete that will be dry or protected from moisture in service
EC1	Moderate	Concrete that will be exposed to moisture but to no more than 500 ppm of chlorides from external source
EC2	Severe	Concrete that will be exposed to moisture and an external source of chlorides in service—from deicing chemicals, salt, seawater, or spray from these sources
EC3	Very severe	Concrete that will be exposed to chemicals, including gases, that are more corrosive than those described in EC2

Table 4.2(d)—Exposure Category ECA: Conditions requiring protection of concrete from chemical attack

Class	Severity	Condition
ECA0	Negligible	An environment where aggressive chemicals do not react with or stain the concrete
ECA1	Limited	An environment where aggressive chemicals react slowly with the concrete
ECA2	Severe	An environment where aggressive chemicals react moderately with the concrete
ECA3	Very severe	An environment where aggressive chemicals react rapidly with the concrete

Table 4.2(e)—Exposure Category EE: Conditions requiring protection of concrete from erosion

Class	Severity	Condition
EE0	Negligible	Concrete that will not be subjected to erosion in service
EE1	Severe	Concrete that will be subjected to abrasion in service
EE2	Severe	Concrete that will be subjected to cavitation in service

sulfates in water should be conducted in accordance with ASTM D4130 or ASTM D516, with ASTM D4130 being the preferred method.

Exposure Category EC applies to concrete exposed to conditions that require additional protection against corrosion of reinforcement.

The ASTM C1202 method measures the current that passes through a concrete specimen exposed to electrolyte solutions and an electric potential. This test method is commonly referred to as the rapid chloride permeability test (RCPT).

It should be noted that the coulomb rating obtained using ASTM C1202 depends on many factors, including type of supplementary cementitious materials and admixtures. Age should be specified, as concrete containing some supplementary cementitious materials may take longer to achieve the specified coulomb rating. Also, as mentioned in ASTM C1202, the test method can produce misleading results by producing higher coulomb values for concrete containing calcium nitrite or other admixtures that are ionic in nature. This could be misinterpreted as lower resistance to chloride ion penetration compared to control concrete. ASTM C1202 recommends ponding tests if an admixture effect is suspected. Refer to ACI 212.3R, ACI SP-108, Perraton et al. (1988), and Whiting (1988) for further guidance.

Exposure Category ECA applies to concrete exposed to chemical attack in service. For more information concerning chemical attack of concrete such as sulfate and carbonation, refer to Neville (1995) and Biczok (1967).

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COMMENTARY

The descriptions of chemical attack conditions are based on the various groups indicated for the chemicals listed in R4.9.1. The licensed design professional should evaluate what exposure conditions will be expected on a project to determine if any special requirements will be needed to prevent or reduce concrete deterioration. Design requirements may include protection of the concrete as indicated in R4.9.1.

Concrete is rarely, if ever, attacked by solid, dry chemicals. To produce a significant attack on concrete, aggressive chemicals must be in a solution above some minimum threshold concentration to drive the chemical reactions that alter the concrete and diminish its engineering properties. Although concrete will perform satisfactorily in a variety of exposure conditions where aggressive chemicals are present, some kinds of chemical environments will significantly diminish the service life of even the best concrete unless specific measures are taken.

Construction elements subjected to aggressive solutions under hydraulic pressure from one side may be more vulnerable than otherwise, because the hydraulic pressures can accelerate the infiltration of the aggressive solution into the concrete. Producing concrete with lower permeability will reduce the rate of infiltration. Reducing the potential reactivity of the cementitious paste, via cement chemistry and the use of supplementary cementitious materials, is also an effective tool for mitigating chemical exposures.

Exposure Category EE applies to concrete exposed to erosion in service.

Severity of exposure within each category is defined by classes with numerical values representing different exposure conditions.

4.3—Requirements for concrete mixtures

Based on the exposure classes assigned from Tables 4.2(a) to (e), concrete mixtures shall comply with the most restrictive requirements according to Tables 4.3(a) to (e).

R4.3—Requirements for concrete mixtures

The maximum ***w/cm*** requirements in this table are primarily intended to reduce the permeability of the uncracked concrete. Very dense concrete microstructures with a low ***w/cm*** are more susceptible to autogenous shrinkage cracking. The risk of autogenous shrinkage increases as the ***w/cm*** decreases below 0.42. Autogenous shrinkage is a volume reduction of the cementitious binder materials by chemical reaction and/or self-desiccation that is dependent on design mixture and constituents. It is a condition where water demand for hydration exceeds the free water available in the mixture and generally occurs after initial setting within the first few days following concrete placement.

Table 4.3(a)—Requirements for Exposure Category EF: Freezing-and-thawing exposure

Exposure Class	Maximum *w/cm*	Minimum f_c' or f_g', psi	Additional minimum requirements	
EF0	0.45	4000	—	
EF1	0.45	4000	Table 4.4	—
EF2	0.42	4500	Table 4.4	—
EF3	0.40	5000	Table 4.4	Maximum percent of SCMs in Tables 4.1.3(d)

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Table 4.3(b)—Requirements for Exposure Category ES: Sulfate exposure

Exposure Class	Maximum *w/cm*	Mininum f_c' or f_g', psi	Required cementitious materials[*]: Types		Additional requirements
			ASTM C150	ASTM C595[†]	
ES0	0.45	4000	—	—	No calcium chloride admixtures
ES1	0.42	4500	II[‡,§]	All (MS) designated cements	No calcium chloride admixtures
ES2	0.40	5000	V[§]	All (HS) designated cements	No calcium chloride admixtures
ES3	0.40	5000	V + pozzolan or slag cement[‖,#]	All (HS) designated cements + pozzolans or slag cement[**,‖]	No calcium chloride admixtures[**]

[*]Alternative combinations of cementitious materials to those listed in Table 4.3(b) shall be permitted when meeting the criteria in 4.5.

[†]ASTM C595 cements that include ASTM C1157 cements are not permitted.

[‡]For seawater exposure, other types of portland cements with tricalcium aluminate (C_3A) contents up to 10 percent are permitted if the *w/cm* does not exceed 0.40.

[§]Other available types of cement such as Type III or Type I are permitted in Exposure Classes ES1 and ES2 if the C_3A contents are less than 8 and 5 percent, respectively.

[‖]The amount of the specific source of the pozzolan or slag cement to be used shall not be less than the amount that has been determined by service record to improve sulfate resistance when used in concrete and shotcrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slag cement to be used shall not be less than the amount tested in accordance with ASTM C1012 and meeting the criteria in 4.5.

[#]Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

[**]Additional corrosion barriers such as coatings or liners shall be required for very severe exposure.

Table 4.3(c)—Requirements for Exposure Category EC: Conditions requiring corrosion protection of reinforcement

Exposure Class	Maximum *w/cm*	Minimum f_c' or f_g', psi	Maximum water-soluble chloride ion (Cl^-) content in concrete, percent by weight of total cementitious materials[*]	Additional minimum requirement
			Reinforced concrete	
EC0	0.45	4000	0.10	Cover[†]
EC1	0.45	4000	0.10	Cover[†]
EC2	0.42	4500	0.10	Cover[†]
EC3	0.40	5000	0.10	Cover plus supplementary cementitious materials[‡]
			Prestressed concrete	
EC0	0.42	4500	0.06	Cover
EC1	0.42	4500	0.06	Cover
EC2	0.40	5000	0.06	Cover
EC3	0.40	5000	0.06	Cover plus qualified supplementary cementitious materials[‡]

[*]Water-soluble chloride ion content that is contributed from the ingredients, including water, aggregates, cementitious materials, and admixtures, shall be determined on the concrete mixture by ASTM C1218 at age between 28 and 42 days.

[†]Refer to 12.7.

[‡]Refer to Table 4.1.3(c).

Note: Refer to 19.16 for unbonded tendons.

Table 4.3(d)—Requirements for Exposure Category ECA: Conditions requiring corrosion protection of concrete from chemical attack

Exposure Class	Maximum *w/cm*	Minimum f_c' or f_g', psi	Additional minimum requirements
ECA0	0.45	4000	—
ECA1	0.42	4500	As stated in 4.9.
ECA2	0.40	5000	As stated in 4.9 and 4.11.
ECA3	0.40	5000	As stated in 4.9 and 4.11.

Reducing pore water tension through the use of shrinkage-reducing admixtures more commonly used to reduce drying shrinkage, increased cement paste hydration through internal curing and moist curing methods to reduce self-desiccation, use of fibers to resist crack propagation, aggregate gradation optimization to minimize paste content, and use of coarser

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Table 4.3(e)—Requirements for Exposure Category EE: Conditions requiring protection from erosion

Exposure Class	Maximum *w/cm**	Minimum f_c' or f_g', psi	Additional minimum requirements
EE0	0.45	4000	—
EE1	0.40	5000	As stated in 4.10.3.
EE2	0.40	5000	As stated in 4.10.2, 4.10.4, 4.10.5, and 4.10.6.

cements have all helped reduce the impact of autogenous shrinkage cracking. ASTM C1698 may be used to predict early-age shrinkage potential by eliminating the curing period from the test (Bentz and Jensen 2004; ASTM C1698; Sant et al. 2006).

Specified compressive strength should correspond to a maximum ***w/cm*** of 0.40 to 0.45 for concrete exposed to freezing and thawing, sulfates, water, wastewater, and to corrosive gases, or for preventing corrosion of reinforcement, as indicated in Tables 4.3(a) to (e). This will typically be equivalent to requiring a specified compressive strength of 5000 to 4000 psi, respectively. Tables 4.3(a) to (e) give the requirements for concrete on the basis of the assigned exposure classes. When an element is assigned more than one exposure class, the most restrictive requirements are applicable. For example, an element assigned to Exposure Classes EC2 and EF3 would be required to comply with a maximum ***w/cm*** of 0.40 and a minimum specified compressive strength of 5000 psi. In this case, the requirement for corrosion protection is more restrictive than the requirement for resistance to freezing and thawing.

Exposure Category EF: In addition to complying with a maximum ***w/cm*** and a minimum strength requirement, concrete subject to freezing-and-thawing exposures should be air-entrained in accordance with 4.4.

Exposure Category ES: Concrete exposed to injurious concentrations of sulfates from soil and water should be made with sulfate-resisting cementitious materials. Table 4.3(b) lists the appropriate types of cement and the maximum ***w/cm*** and minimum specified compressive strengths for various exposure conditions. In selecting a cement for sulfate resistance, the principal consideration is its tricalcium aluminate (C_3A) content. For Class ES1 (moderate exposure), Type II cement is limited to a maximum C_3A content of 8.0 percent under ASTM C150. The blended cements under ASTM C595 with the MS designation are appropriate for use in Class ES1. The appropriate cement types under ASTM C595 are any of the (MS) and (HS) designated cements. For Class ES2 (severe exposure), ASTM C150 Type V cement with a maximum C_3A content of 5 percent is specified. 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, sulfuric acid. The contract documents should cover all special cases.

The use of fly ash (ASTM C618 Class F), natural pozzolans (ASTM C618 Class N), silica fume (ASTM C1240), or

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slag cement (ASTM C989) also has been shown to improve the sulfate resistance of concrete. ASTM C1012 can be used to evaluate the sulfate resistance of mixtures using combinations of cementitious materials as determined in 4.5. For Exposure ES3, the alternative in ACI 350-xx allowing use of Type V plus pozzolan, based on records of successful service, instead of meeting the testing requirements of 4.5, still exists and has been expanded to consider the use of slag cement and blended cements.

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

Stark (1989) showed that reducing the ***w/cm***, thereby reducing permeability, is a major factor in increasing the resistance of concrete to sulfate attack.

Exposure Category EC: For reinforced concrete in Exposure Category EC, the maximum ***w/cm***, minimum specified compressive strength, and minimum cover are the basic requirements to be considered. Conditions in structures where chlorides may be present, such as structures near seawater, should be evaluated for greater corrosion-protection requirements. Epoxy- or zinc-coated bars or cover greater than the minimum required in 7.7 may be desirable. Use of slag cement meeting ASTM C989 or fly ash meeting ASTM C618 Class F or Class C, and increased levels of specified compressive strength, provide increased protection. Use of silica fume meeting ASTM C1240 with an appropriate high-range water reducer, ASTM C494 Types F and G, or ASTM C1017 can also provide additional protection (Ozyildirim and Halstead 1988). Satisfactory results for concrete tested in accordance with ASTM C1202 can provide additional assurance of chloride resistance.

The use of corrosion-inhibiting admixtures may also be considered.

The chloride ion limits apply to **Exposure Category EC**. The permitted maximum amount of water-soluble chloride ions incorporated into reinforced and prestressed concrete, measured by ASTM C1218 at ages between 28 and 42 days, are 0.10 and 0.06 percent chloride ion by weight of total cementitious materials, respectively, regardless of exposure. For prestressed concrete, the limit of 0.06 percent chloride ion by weight of total cementitious materials applies regardless of exposure.

Additional information on the effects of chlorides on the corrosion of reinforcing steel is given in ACI 201.2R, which provides guidance on concrete durability, and ACI 222R, which provides guidance on factors that impact corrosion of metals in concrete.

An initial evaluation of the chloride ion content of the proposed concrete mixture may be obtained by testing individual concrete ingredients for chloride ion content. Evalu-

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ation of the chloride content of the proposed concrete ingredients may be performed as follows:

1. The chloride content of fine and coarse aggregates, cementitious materials, and mixing water should be determined by independent testing. Chloride content of aggregate may be determined in accordance with ASTM C1524. Chloride content of cement may be determined in accordance with the Reference Test Method for chloride as defined in ASTM C114. Chloride ion content of water may be determined in accordance with ASTM D512. If potable mixing water is used, it is permissible to use the water supplier's published maximum chloride content in lieu of additional testing.

2. The chloride content of each admixture or additive should be determined by independent testing. Alternately, the admixture or additive supplier could submit documentation acceptable to the Engineer showing that the admixture or additive does not contain intentionally added chlorides and that the chloride content of the admixture or additive does not exceed 0.5 percent by weight of the admixture or additive. In this case, the chloride content of the admixture or additive should be assumed to be equal to 0.5 percent by weight of the admixture or additive.

3. Document the chloride ion content of each ingredient as a percent by weight of the ingredient. Multiply the percent chloride content by the weight of the ingredient included in the concrete mixture to determine the chloride ion content of that ingredient.

4. Add up the chloride ion contents contributed by each of the ingredients to obtain the total chloride ion content contributed to the concrete mixture.

5. Divide the total chloride ion content by the total weight of cement in the concrete mixture. The result is the total chloride ion content as a percent by weight of cement.

This method is a conservative approach. If the total chloride ion content as a percent by weight of cement exceeds the maximum chloride content permitted in Table 4.3(c), it may be necessary to test samples of the hardened concrete for water-soluble chloride ion content in accordance with ASTM C1218. Note that some chloride ions present in the ingredients may either be insoluble in water or may react with the cementitious materials during hydration and become insoluble. When concrete is tested for water-soluble chloride ion content, the tests should be made at an age of 28 to 42 days. The limits in Table 4.3(c) 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 4.3(c) differ from those recommended in ACI 318, ACI 201.2R, and ACI 222R. The limits were reduced for environmental engineering concrete structures due to their greater susceptibility to corrosion of metals when experiencing prolonged exposures to chloride-saturated liquids. For reinforced concrete that will be dry in service (Class EC0), a limit of 0.10 percent has been included to control the water-soluble chlorides introduced by the ingredients of the concrete mixture.

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COMMENTARY

ACI 222R has adopted slightly different categories and limits as indicated in the table below. ACI 201.2R has adopted these same limits by referring to ACI 222R.

Chloride limits for new construction (Table 3.1 from ACI 222R)

Category	Chloride limit for new construction, percent by mass of cement		
	Test method		
	Acid-soluble	Water-soluble	
	ASTM C1152	ASTM C1218	Soxhlet*
Prestressed concrete	0.08	0.06	0.06
Reinforced concrete wet in service	0.10	0.08	0.08
Reinforced concrete dry in service	0.20	0.15	0.15

*The Soxhlet test method is described in ACI 222.1.

The categories from ACI 222R are similar to Class EC1 and EC0 defined in Table 4.2(c), for reinforced concrete wet and dry in service, respectively. The recommended limit for prestressed concrete in ACI 222R is the same as in this Code.

Exposure Category ECA applies to concrete exposed to chemical attack in service. For more information concerning chemical attack of concrete, such as sulfate and carbonation, refer to Neville (1995) and Biczok (1967).

Table R4.3.1 summarizes the effects of some of the more common chemicals that lead to concrete deterioration. Table R4.3.1 is a preliminary guide and the licensed design professional should consult the listing of chemicals in R4.8.1 and other references for more complete information. Table R4.3.2 summarizes some of the factors which may affect the rate of chemical attack.

ASTM C267 is intended to evaluate the chemical resistance of materials, including hydraulic cement, and provides for the determination of changes in the following properties of the test specimens: weight, appearance, and compressive strength. The results can serve as a guide for selection of one chemically resistant material versus another. For mortars,

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Table R4.3.1—Effect of common chemicals on concrete*

Rate of attack at ambient temperature	Inorganic acids	Organic acids	Alkaline solutions	Salt solutions	Miscellaneous
Rapid	Hydrochloric Nitric Sulfuric				
Moderate	Phosphoric				
Slow	—	Acetic Formic Lactic	—	Aluminum chloride	—
	—	Tannic	Sodium* hydroxide > 20 percent	Ammonium nitrate Ammonium sulfate Sodium sulfate Magnesium sulfate Calcium sulfate	Bromine (gas) sulfite liquor
	Carbonic	—	Sodium† hydroxide 10-20 percent Sodium hypochlorite	Ammonium chloride Magnesium Chloride Sodium cyanide	Chlorine (gas) seawater soft water
Negligible	—	Oxalic Tartaric	Sodium† hydroxide < 10% Ammonium hydroxide	Calcium chloride Sodium chloride Zinc nitrate Sodium chromate	Ammonia (liquid)

*The effect of potassium hydroxide is similar to that of sodium hydroxide.

†Refer to R4.8.1, PCA IS001, and ACI 515.2R for a more complete list of chemicals and their potential effects on concrete.

Table R4.3.2—Factors influencing chemical attack on concrete

Factors that accelerate or aggravate attack	Factors that mitigate or delay attack
1. High permeability due to: High water content in the concrete mixture proportions a. High *w/cm* b. Poor consolidation c. Poor curing d. Microcracking	1. Dense concrete achieved by: a. Proper mixture proportioning* b. Reduced unit water content 1. Increased use of supplementary cementitious materials 2. Air entrainment 3. Adequate consolidation 4. Effective curing†
2. Cracks and separations due to: a. Loading/Stress concentrations b. Thermal stress c. Shrinkage	2. Reduced tensile stress in concrete by:‡ a. Using tensile reinforcement of adequate size, correctly located b. Inclusion of pozzolan (to reduce temperature rise) c. Provision of adequate movement joints Effective curing†
3. Leaching and liquid penetration due to: a. Flowing liquid§ b. Ponding c. Hydraulic pressure	3. Structural design a. To minimize areas of contact and turbulence b. Provision of membranes and protective-barrier system(s)‖ to reduce penetration c. Provision of adequate drainage and through-flow

*The mixture proportions affect concrete homogeneity and density.

†Poor curing procedures may result in a weak concrete surface with flaws and cracks.

‡Resistance to cracking depends on strength and strain capacity.

§Movement of water-carrying deleterious substances increases reactions that depend on both the quantity and velocity of flow.

‖Concrete that will be frequently exposed to chemicals known to produce rapid deterioration should be protected with a chemically resistant protective-barrier system. Evaluation of chemically resistant protective-barrier systems should be conducted to verify suitability of the system for the intended service.

the test specimens are specified as 2 in. cubes, but cylinders can be used. When testing concrete, use chemically resistant aggregates that will be used in the project's concrete. Test specimens are subjected directly to the medium and conditions of exposure after the manufacturer's recommended conditioning period. Conduct the test at the ages of 1, 7, 14, 28, 56, and 84 days. Weight and dimensions of speci-

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CODE

COMMENTARY

mens are determined by analytical balance and micrometer. In addition, compressive strength determinations are made on specimens stored in the same conditions as the mass and dimension specimens. The following can be included when reporting test results: material ID, design mixture, conditioning procedure, test conditions, appearance of samples, average percentage change in compressive strength, graph of change in mass versus time, and graph of strength change versus time.

Testing according to ASTM C267 should be extended past 84 days if there is any reaction of the test specimens to the chosen exposure conditions, to determine if deterioration will continue that may lead to catastrophic failure. Catastrophic failure or the loss of all physical integrity is considered to be when the compressive strength falls below a minimum specified strength target using ASTM C579.

To determine the chemical resistance of concrete aggregates, use the procedure described in test method ASTM C1370. When using this method, a chemically resistant aggregate will lose no more than 1 percent of its initial mass after exposure to the environment of intended use.

Exposure Category EE: Refer to ACI 210R for guidance. Erosion Class EE3 covers erosion from chemical attack. Refer to 4.8 and the references indicated in those paragraphs.

Table R4.3.3—Evaluation of protection system materials and visual inspection of concrete in service

Class	Severity	Condition for evaluation of protection system materials selection*	Visual inspection of concrete in service based on field observation
ECA0	Negligible	1 percent mass loss and no strength loss	Loss of 50 percent surface paste, exposed fine aggregate
ECA1	Moderate	Any mass loss and less than 10 percent strength loss	Loss of all surface paste, exposed fine aggregate
ECA2	Severe	Any mass loss and greater than 50 percent strength loss	Loss of all surface paste, fine and coarse aggregates exposed
ECA3	Very severe	Greater than 50% mass loss and greater than 50% strength loss	Loss of all surface paste, fine and coarse aggregates exposed to depth of maximum coarse aggregate size

*Based on testing in accordance with ASTM C267.

4.4—Additional requirements for freezing-and-thawing exposures

Concrete and wet-mix shotcrete subject to Exposure Category EF (Table 4.2(a)) shall be air-entrained in accordance with Table 4.4. Tolerance on air content as delivered shall be ±1.5 percent, except that no concrete shall have less than 3.5 percent air content. For specified compressive strength greater than 5000 psi, reduction of air content indicated in Table 4.4 by 1.0 percent shall be permitted.

Dry-mix shotcrete subject to Exposure Class EF3 (Table 4.2(a)) shall be air-entrained in accordance with Table 4.4.

R4.4—Additional requirements for freezing-and-thawing exposures

A table of required air contents for freezing-and-thawing resistant concrete is included in the Code based on ACI 211.1. Values are provided for Class EF1 (moderate) and both Class EF2 and EF3 (severe) exposures, 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.

Section 4.4 permits an air content of 1 percentage point lower for specified compressive strength greater than 5000 psi. Such high-strength concrete will have a lower

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Table 4.4—Total air content for frost-resistant concrete

	Air content, percent	
Nominal maximum aggregate size, in.*	Exposure Class EF2 and EF3[†]	Exposure Class EF1
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

*Refer to ASTM C33 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 calculated from the value determined on the minus 1-1/2 in. fraction.

†Wet-mix shotcrete that is subjected to Exposure Classes EF2 or EF3 shall have a minimum air content of 6 percent before pumping.

‡Dry-mix shotcrete that is subjected to Exposure Class EF3 shall have a minimum in-situ air content of 6 percent.

4.5—Additional requirements for sulfate exposures

Alternative combinations of cementitious materials to those listed in Table 4.1.3(b) shall be permitted when tested for sulfate resistance in accordance with ASTM C1012. Expansion limits for these alternative combinations are based on the water-soluble sulfate representation of the anticipated placement environment as determined by ASTM C1580 for soils or ASTM D4130 or ASTM D516 for water, whichever is the more stringent according to Table 4.2(b). The mixtures selected shall also meet the criteria in Table 4.5 and the minimum f_c' or f_g' and maximum w/cm indicated in Table 4.3(b).

COMMENTARY

w/cm and lower 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 (refer to Slater et al. [1987]).

The air entrainment requirements for freezing-and-thawing-resistant shotcrete included in the Code are based on ACI 506R. When wet-mix shotcrete is placed, a significant amount of air content is lost during shooting. Therefore, a minimum of 6 percent entrained air should be in the shotcrete mixture prior to shooting to compensate for the air that is lost during shooting. Some shotcrete contractors add as much as 10 percent air in the shotcrete before shooting to enhance pumpability and reduce rebound, even though the resulting in-place air content will only be 4 to 6 percent.

Research has shown that properly placed non-air-entrained dry-mix shotcrete has equal durability to that of air-entrained concrete and wet-mix shotcrete when exposed to freezing-and-thawing conditions in the presence of fresh water moisture (ACI 506R; Morgan 1989; Seegebrecht et al. 1989; Vezina 2001). Air-entraining admixtures should be used to ensure the durability of dry-mix shotcrete exposed to freezing-and-thawing conditions in the presence of external sources of chlorides such as deicing chemicals, salt, seawater, or spray from these sources. This may be accomplished by adding air-entraining admixtures to the mixing water or through the use of prepackaged bag mixtures that contain dry-powdered air-entraining admixtures.

Experience has shown that application of dry-mix shotcrete outside of vertical or overhead surfaces present distinct challenges in controlling rebound and overspray. Thus, dry mix is not recommended for more horizontally oriented sections, as in low-rise domes over tanks. In these more horizontally oriented sections, wet-mix shotcrete, cast-in-place concrete, or precast concrete should be used for construction.

R4.5—Additional requirements for sulfate exposures

In ACI 350-06 and ACI 318-08, ASTM C1012 is permitted to be used to evaluate the sulfate resistance of concrete mixtures using alternative combinations of cementitious materials to those listed in Table 4.3(b) for all classes of sulfate exposure. More detailed guidance on qualifications of such mixtures using ASTM C1012 is given in ACI 201.2R. The expansion criteria in Table 4.5, for testing in accordance with ASTM C1012, are the same as those in ASTM C595 for moderate sulfate resistance (Optional Designation MS) in Exposure Class ES1.

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Table 4.5—Requirements for establishing suitability of cementitious materials combinations exposed to water-soluble sulfate

Exposure Class	Maximum expansion when tested using ASTM C1012
ES0	—
ES1	0.10 percent at 6 months
ES2	0.05 percent at 6 months, or 0.10% at 12 months*
ES3	0.10 percent at 18 months

*The 12-month expansion limit only applies when the 6-month limit is not met.

4.6—Additional requirements for alkali-aggregate reactions

AAR describes both alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR).

4.6.1 *Alkali-silica reaction*

4.6.1.1 *Determining alkali-silica reactivity of aggregates*

Except as permitted by 4.6.1.2, the potential reactivity of aggregates shall be determined by ASTM C1293, ASTM C1260, or long-term field performance.

4.6.1.1.1 *ASTM C1293 (concrete prism test)*

Average expansion of less than 0.040 percent in ASTM C1293 at 1 year indicates an aggregate is nonreactive and the aggregate shall be permitted to be used in concrete without further ASR testing. If the average expansion is greater than or equal to 0.040 percent at 1 year, the aggregate shall be considered potentially reactive and preventive measures in 4.6.1.2 shall be required prior to using the aggregate.

4.6.1.1.2 *ASTM C1260 (accelerated mortar bar test)*

Average expansion of less than 0.10 percent in ASTM C1260 at 16 days after casting indicates an aggregate is nonreactive and the aggregate shall be permitted to be used in concrete without further alkali-silica reactivity testing. If the expansion is greater than or equal to 0.10 percent at 16 days, the aggregate is potentially reactive and preventative measures in 4.6.1.2 shall be required prior to using the aggregate, except as per 4.6.1.1.1.

4.6.1.1.3 *Long-term field performance*

If laboratory testing is not performed, the long-term field performance shall be used to determine the aggregate reactivity. The determination of long-term field performance shall be done by the licensed design professional. An existing concrete structure may be used for long-term field performance if the following criteria are met:

(a) Existing structure shall be at least 10 years old

(b) Cement content and alkali content shall be not greater for the proposed structure than the concrete used in the existing structure

COMMENTARY

R4.6—Additional requirements for alkali-aggregate reactions

R4.6.1.1.1 *ASTM C1293 (concrete prism test)*

If the potentially reactive aggregate being tested is a coarse aggregate, use a nonreactive fine aggregate and vice versa. The coarse-fine aggregate combination should be used to produce concrete prisms with a specified high-alkali loading. This test method is intended to evaluate coarse and fine aggregates separately and should not be used to evaluate combinations of coarse and fine aggregates to be used on a project.

R4.6.1.1.3 *Long-term field performance*

Where long-term field performance is used, as many structures as practical should be included in the survey and these structures should, where possible, represent different types of construction (for example, foundations, walls, bridges, pavements, sidewalks, and structural elements). ASTM C823 provides useful guidance when surveying structures to establish field performance history. The following information should be documented for each structure:

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(c) Exposure conditions at the proposed structure shall be no more severe than for the existing structure

(d) Petrographic examination shall demonstrate that the aggregate in the existing structure is from the same quarry and is of similar mineralogy as that to be used in the proposed structure, and to determine if SCMs were used in the existing structure

Preventative measures required in 4.6.1.2 shall be followed if long-term field performance shows that ASR is of concern.

4.6.1.2 *Determining preventative measure for alkali-silica reaction*

When an aggregate is determined to be potentially reactive, or instead of determining alkali-silica reactivity of aggregates as indicated in 4.6.1, one of the following two approaches shall be used to minimize the potential for ASR:

COMMENTARY

(a) Age: Structures should be at least 10 years old and preferably more than 15 years old, as visible damage from ASR can take more than 10 years to develop

(b) Portland cement content and portland cement alkali content of the cement used during construction

(c) Use and content of pozzolans or slag cement or blended cements during construction

(d) Exposure condition: Availability of moisture, use of deicing chemicals, or exposure to salts

(e) Symptoms of distress as a result of ASR or other causes

Cores should be taken from a representative number of these structures and a petrographic examination conducted using ASTM C856 to establish the following:

(a) The aggregate used in the structure surveyed is of similar mineralogical composition as determined by ASTM C295 to that of the aggregate to be used

(b) Any evidence of damage as a result of ASR

(c) The presence, quantity, and composition (if known) of fly ash, slag cement, or other supplementary cementitious materials because these materials are capable of mitigating ASR

If the results of the field survey indicate that the aggregate is nonreactive, the aggregate may be used in new construction provided that the new concrete is not produced with a higher concrete alkali content; a lower amount of pozzolan, slag cement, or blended cement; or placed in a more aggressive exposure condition than the structures included in the survey.

There is a certain level of uncertainty associated with accepting aggregates solely on the basis of field performance because of difficulties in establishing unequivocally that the materials and proportions used more than 10 to 15 years ago are sufficiently similar to those to be used in new construction. If field performance indicates that an aggregate source is potentially deleteriously reactive, laboratory testing can be conducted to determine the level of aggregate reactivity and evaluate preventive measures. The use of long-term performance is considered a reliable method in determining the effectiveness of an aggregate; however, it is often very difficult to acquire the information and background for existing structures. For more information, refer to CSA A23.2-27A, AASHTO PP65, Thomas et al. (2008), and ASTM C1778.

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4.6.1.2.1 *Performance-based approach*

The concrete shall be tested using one of the following test methods to evaluate if the use of SCMs in the concrete will control deleterious expansion. Mixture proportions that satisfy the test requirements shall be permitted to be used.

4.6.1.2.1.a *ASTM C1293 (concrete prism test)*

The use of SCMs or lithium nitrate in the concrete to control deleterious expansion with the potentially reactive aggregate can be evaluated using ASTM C1293 for a 2-year duration. The expansion shall be less than 0.04 percent at 2 years for the combination of SCMs or lithium nitrate and reactive aggregate to be suitable for use in concrete construction.

4.6.1.2.1.b *ASTM C1567 (accelerated mortar bar test)*

The use of SCMs in the concrete to control deleterious expansion with the potentially reactive aggregate can be evaluated using ASTM C1567. The expansion shall be less than 0.10 percent at 16 days after casting for the combination of SCM and reactive aggregate to be used in concrete construction.

4.6.1.2.2 *Prescriptive-based approach*

Table 4.1.3(a) provides the minimum SCM replacement percentages for mitigating ASR when the prescriptive approach is used.

4.6.2 *Alkali-carbonate reaction*

4.6.2.1 *Determining alkali reactivity of aggregates*

In regions known to have dolomitic aggregates that are susceptible to ACR, the potential alkali-carbonate reactivity of aggregates shall be determined by ASTM C586 or ASTM C1105, or both. When an aggregate is determined to be potentially reactive by ASTM C1105, the aggregate shall not be used in concrete.

4.6.2.1.1 *ASTM C586 (rock-cylinder method)*

Expansion of less than 0.10 percent in ASTM C586 at 28 days after immersion in sodium hydroxide indicates an aggregate is nonreactive. Aggregate exhibiting expansion greater than 0.10 percent shall be tested using ASTM C1105.

COMMENTARY

R4.6.1.2.1.a *ASTM C1293 (concrete prism test)*

Standardization of lithium in mortars within ASTM has not occurred; AASHTO PP65 does offer a method for determining the standard dosage of lithium. The dosage of lithium-nitrate admixture is based on the alkali content of the cement in the concrete mixture. A procedure for calculating a lithium-nitrate admixture addition rate is provided in AASHTO PP65. Additional guidance is provided in ACI 212.3R and CRD-C 662.

R4.6.1.2.2 *Prescriptive-based approach*

Alkali loading is the total amount of alkalis in the concrete mixture expressed in lb/yd^3. This is calculated by multiplying the portland cement content of the concrete in lb/yd^3 by the alkali content of the cement divided by 100.

R4.6.2.1.1 *ASTM C586 (rock-cylinder method)*

Reactions that occur between cement alkalis and carbonate aggregates are sensitive to aggregate lithology. The measurement results should be interpreted by qualified personnel with recognition of these variables. The acceptance or rejection of aggregate sources solely on the results of this test is not recommended because, in commercial production, expansive and non-expansive materials may occur in close proximity and the securing of samples adequately representative of the variability of the production of the source is a difficult task and requires the efforts of an individual trained to distinguish differences in lithology.

In the case of alkali-carbonate reaction, neither the use of low-alkali cement nor the use of a supplementary cementitious material are effective in preventing deleterious expansion. The only means of preventing the reaction of a reactive

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aggregate is to not use the aggregate. Preventive measures that are often used to prevent ASR are ineffective for ACR and the aggregate should not be used.

Table 4.1.3(a) shows the minimum SCM replacement amounts to be used in concrete when the prescriptive-based approach is used to control the potential for alkali-silica reaction.

Slag cement, silica fume with alkali contents greater than 1 percent Na_2O_e, and fly ashes with alkali contents greater than 4 percent Na_2O_e and/or with CaO contents greater than 15 percent, may be used when their effectiveness in reducing expansion due to ASR is demonstrated in accordance with Section 4.6.1.2.1.b.

The chemical composition of silica fume shall be a minimum of 85 percent SiO_2 for use in concrete. For more information refer to ASTM C1240.

LBA is calculated by multiplying the portland cement content of the concrete in lb/yd^3 by the alkali content of the portland cement divided by 100. For example, for concrete containing 500 lb/yd^3 with an alkali content of 0.81 percent Na_2O_e, the value of LBA = 500 × 0.81/100 = 4.05 lb/yd^3. For this concrete, the minimum replacement level of silica fume is 1.8 × 4.05 = 7.3 percent. Regardless of the calculated value, the minimum level of silica fume shall not be less than 7 percent when it is the only method of prevention.

4.6.2.1.2 *ASTM C1105 (ACR concrete prism test)*

Expansion of less than 0.015 percent at 3 months, 0.025 percent at 6 months, or 0.030 percent at 1 year in ASTM C1105 indicates an aggregate is nonreactive and the aggregate shall be permitted to be used in concrete. If the expansion is greater than or equal to 0.015 percent at 3 months, 0.025 percent at 6 months, or 0.030 percent at 1 year, the aggregate is potentially reactive.

4.7—Additional requirements for corrosion protection of reinforcement and other metal embedments

4.7.1 Where conditions exist that reduce the corrosion passivation layer around, or directly corrodes, the reinforcement, the reinforcement shall be protected.

4.7.2 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.

4.7.3 Where contact between dissimilar metals will result in galvanic corrosion, isolators shall be used to prevent such contact.

R4.7—Additional requirements for corrosion protection of reinforcement and other metal embedments

R4.7.1 The corrosive conditions that require protection depend on the expected chemical exposure. Refer to 4.8.1. Chemicals that cause carbonation and resulting loss of passivation of the concrete around the reinforcing steel are included at the end of R4.8.1.4.

R4.7.2 For additional precautions against corrosion of unbonded tendons, refer to 19.16. Refer to R19.21.4 for optional additional protection of unbonded tendon anchorages.

R4.7.3 Metallic embedments including pipe spools, conduits, anchor bolts, and other potential dissimilar metals should not be tied to the reinforcement.

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

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

4.8—Additional requirements for protection against chemical attack

4.8.1 *Concrete protection against chemicals*

4.8.1.1 Concrete subject to attack by corrosive chemicals or corrosive gases shall be protected in accordance with 4.7.1.2, 4.7.1.3, and 4.7.1.4.

4.8.1.2 Proportion concrete with the appropriate types of cement and supplementary cementitious materials as applicable. Batch mixture, place, consolidate, finish, and cure the concrete to account for the expected chemical exposure.

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

4.8.1.4 Concrete exposed to chemical attack by sulfate or corrosive chemicals 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 4.10.

(b) Concrete shall be protected against corrosive chemicals with a protective coating in accordance with 4.10.

COMMENTARY

R4.7.4 Guidance is given in the CRSI Maintenance Guide, "Field Handling Techniques for Epoxy-Coated Rebar at the Job Site" (CRSI 1996).

R4.8—Additional requirements for protection against chemical attack

The type of protection used 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.

R4.8.1 *Concrete protection against chemicals*

R4.8.1.1 Facilities and structures such as, but not limited to, the following are to be protected when exposed to corrosive chemicals:

(a) Water treatment plants
(b) Domestic and industrial wastewater treatment plants
(c) Storage containment structures and reservoirs
(d) Water and wastewater pump stations
(e) Conduits, sewers, manholes, and junction chambers

R4.8.1.2 Supplementary cementitious materials 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.

R4.8.1.4 Some of the most common chemicals that may be found in liquids contained by or in direct contact with environmental engineering 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 listed even though they are not considered harmful to concrete. In some instances, treatment of the concrete 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
Calcium hydroxide
Calcium oxide

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COMMENTARY

Carbon dioxide (gas)*
Chlorine, gas (dry)
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 10 percent†
Sodium silicate
Sulfur dioxide (gas)
Tetrasodium pyrophosphate
Trisodium phosphate

*Carbonation may occur, breaking down the passive layer on the surface of the reinforcement, which may allow corrosion of reinforcement.

†Caution is recommended with respect to alkali-reactive aggregates. Refer to Sections 3.3.3/R3.3.3 of this Code.

Group 2

These chemicals will stain concrete and, sometimes where appearance will be a concern, protection of the concrete should be required to prevent the staining.

Activated carbon (except when agitated, then place in Group 3)
Sodium hydroxide when more than 15 percent
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 chemicals needs to be given a protective lining or corrosion-resistant topping in accordance with 4.9. 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 (chemical attack) of concrete

Acetic acid
Ammonia silicofluoride
Calcium hypochlorite
Carbon dioxide
Chlorine (solution) at 0.35 percent
Chlorine dioxide (solution)
Cyanide
Disodium phosphate
Ferrous chloride at up to 35 percent

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Hydrogen sulfide
Iodine
Phosphoric acid at 85 percent
Potassium hydroxide when more than 15 percent
Sodium chloride (dry)*
Sodium fluoride
Sodium hexametaphosphate
Sodium thiosulfate
Sulfur dioxide at 1 percent (solution)

*In porous or cracked concrete, sodium chloride attacks steel reinforcement.

Group 3B—Moderate 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
Hydrogen peroxide at 50 percent
Manganese sulfate
Potassium aluminum sulfate
Potassium hydroxide when greater than 20 percent
Potassium sulfate
Sodium aluminate at 40 percent
Sodium bisulfate
Sodium bisulfite at 38 percent
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
Potassium acetate
Potassium hydroxide
Sodium hydroxide more than 20 percent and temperatures exceeding 150°F
Sulfuric acid up to 98 percent and 150°F.

Table 2.5.2 of ACI 515.2R provides additional information on the effect of chemicals on concrete. PCA IS001.08T (2007) is also a reference.

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COMMENTARY

Specific attention should be given when chemicals in solution under high temperature, pressure, or both, will be in contact with concrete structures because these conditions may accelerate the corrosion process. In these types of service conditions, a liquid or gas under pressure will be forced through permeable or slightly cracked concrete and will come in contact with reinforcement 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 concrete, thereby reducing the protective alkaline environment around the reinforcing steel or metals, thus allowing the corrosion process to proceed. Refer to ASCE (1989).

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 greater than 7. Design and specifying information may be obtained from manufacturers and ACI publications such as ACI 515.1R.

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 8.0 or lower. 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 to approximately 8.0, the passive layer breaks down, and reinforcement corrosion may occur; refer to ACI 201.2R and 222R.

4.8.2 Jointing materials, including waterstops, preformed joint filler, and sealants, shall be resistant to deterioration due to anticipated exposure to chemicals and shall accommodate the anticipated service temperature range for the design life of the facility. Materials shall be tested in accordance with ASTM C920 and Federal Specification TT-S-00227E for sealants; ASTM D570, ASTM D746, and CRD-C572 for PVC waterstops (refer to 4.11.2); and ASTM D471 and ASTM D1171 for thermoplastic elastomeric rubber (TPE-R) waterstops.

4.8.3 *Testing for chemical effects*

4.8.3.1 The adequacy of protection against chemical effects shall be certified in writing by the product manufacturer.

R4.8.3.1 The manufacturer should certify their products provide protection against chemicals to which the product

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for the intended exposures, and confirmed by independent testing, or shall be selected based on documented experience records with similar exposures. The tests shall determine the need for and the effectiveness of special cements, liners, coatings, and other protective measures.

4.8.3.2 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.

4.8.3.3 When concrete is to be exposed to potentially corrosive chemicals, it shall be tested for reactions to chemical attack in accordance with ASTM C856, and aggregates shall be tested for reactions to chemical attack in accordance with ASTM C295.

4.9—Additional requirements for protection against erosion

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

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

(a) Reduce flow velocity and dissipate energy/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 4.8.3

4.9.3 Where a structure will be subjected to abrasion erosion, aggregates shall meet requirements of ASTM C33, concrete shall be tested in accordance with ASTM C1138M, and the concrete shall comply with the following additional requirements:

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

(b) Maximum w/cm = 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 materials per cubic yard of concrete

(e) Hard, dense, clean aggregates, with a MOH hardness of 5 or greater (no carbonates)

COMMENTARY

will be exposed. A licensed design professional may request test results from an independent laboratory to confirm certification.

R4.8.3.3 Where concrete is exposed to a more severe chemical attack, such as by acids, acid-resistant aggregates, a protection system, or both should be used; refer to ACI 221R. Concrete chemical attack can be determined by petrographic examination of concrete samples in accordance with ASTM C856. Acid resistance of aggregate should be tested in accordance with ASTM C295.

R4.9—Additional requirements for protection against erosion

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

R4.9.1 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 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.

R4.9.2 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.

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. Refer to Section 5.3 of ACI 210R.

R4.9.3 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 w/cm. The abrasion-erosion resistance of concrete containing quartzite aggregates has been shown to be approximately twice that of concrete containing limestone aggregates (refer to Fig. 6.1 in ACI 210R). Given a quality

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Lower cementitious materials contents than those indicated may be permitted by the licensed design professional when testing and documentation of the proposed concrete mixture verifies compliance with the Code requirements for abrasion, erosion, durability, and workability, in addition to the required concrete strength.

Where additional protection is needed, use coatings and liners in accordance with 4.10.1.

4.9.4 Structures exposed to cavitation erosion shall be constructed with high-strength, low-***w/cm*** concrete containing silica fume and no carbonate aggregates, and shall receive surface finishes that are smooth with slight changes of slope in the direction of flow.

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

4.9.6 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 C1138M.

4.10—Protection systems

4.10.1 Protection systems shall include coatings and linings that are attached or applied to concrete surfaces to provide protection.

4.10.2 When concrete is in contact with corrosive chemicals or corrosive gases that attack the cement mortar matrix,

COMMENTARY

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 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 (Holland 1983). 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.

R4.9.4 Surface imperfections have been determined to have caused cavitation damage at low flow velocity. Tolerances meeting ACI 117 may be satisfactory. Under certain conditions, however, it may be necessary to require more stringent 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. Properly designed high-strength concrete with a low ***w/cm*** has greater resistance to cavitation erosion. The use of silica fume, 1-1/2 in. maximum size aggregate, and water-reducing admixtures have proven effective in increasing resistance to erosion.

Application of dry-mix and wet-mix shotcrete using proper equipment and placement techniques will help to provide abrasion-resistant shotcrete with properly encased reinforcement and in-place shotcrete material that is well-consolidated and free of imperfections. Delivery equipment for dry-mix shotcrete should be checked to ensure material can be applied with a smooth, steady flow of material and with controlled mixing water and proper impact velocity. Wet-mix shotcrete should also be delivered to the nozzle at the proper slump and applied at the proper velocity. Refer to ACI 506R.

R4.9.5 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.

R4.9.6 When testing concrete in accordance with ASTM C1138M, a 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.

R4.10—Protection systems

R4.10.1 Two guides for determining when a particular substance attacks concrete and what are acceptable protection treatments are available in ACI 515.2R and PCA IS001 (2007).

R4.10.2 Consideration should be given to attack by corrosive chemicals or gases and biologically induced corrosion

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or embedded reinforcement, protection systems shall be used.

4.10.3 Surface protection systems shall not be used when in contact with ozone gas without a successful history of previous installations demonstrating compatibility with ozone.

4.10.4 Protection systems shall be provided for the expected exposure. The effectiveness of the protection systems shall have been confirmed by independent testing or shall be based on documented experience records. Protection systems' thickness shall be measured using film thickness gauges.

4.10.5 *Vapor transmission of liners and coatings*

Where water vapor transmission (WVT) is detrimental or where dangerous gases need to be contained, coatings and liners shall have a WVT less than 1×10^{-6} cm/s when tested in accordance with ASTM E96.

COMMENTARY

from bacteria, fungi, and algae on the concrete elements in selecting a compatible protection system. In addition, each project should be considered individually because various materials and techniques used from time to time can cause new problems of chemical attack. Detailed recommendations are given in ACI 515.1R.

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

It is recommended that form oils, form-release agents, and curing components be checked for compatibility with surface protection systems where surface protection systems are specified. For example, where a coating is to be applied to the concrete surface, the use of curing compound should be avoided unless it can be removed or demonstrated the curing compound will not affect the required bond of the coating to the concrete surface.

Follow procedures outlined in ACI 515.1R and International Concrete Repair Institute (ICRI) Guideline No. 310.2R, **"Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings and Polymer Overlays."**

R4.10.3 Special attention is called to PVC vulnerability in contact with ozone gas.

R4.10.4 For additional information on protection systems, refer to ACI 350.2R.

When cavitation or grit in the fluids of the process stream have been shown to cause damage to protection systems, testing in accordance with the following ASTM standards should be conducted: ASTM D660, D661, D662, and D4214. A loss in thickness of the protection system less than 50 percent over the estimated service life of the structure has been shown to be an acceptable limit for protection systems. ACI 210R Section 9.2.8 refers to high-head erosion tests conducted on polyurethane and neoprene protection systems. Testing has indicated problems with similar flexible protection systems when a portion of the protection system is torn from the edge of the concrete surface and much of the remaining protection system peeled away due to the hydraulic force. For a description of this testing, refer to Houghton et al. (1978).

ASTM D5162 can be adopted for concrete surfaces and is one method for determining the effectiveness of the protection system.

R4.10.5 *Vapor transmission of liners and coatings*

While it is an advantage to have breathable protection systems, 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.

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COMMENTARY

Hazardous and dangerous gases include ozone, methane, hydrogen sulfide, and chlorine or any gas that may be explosive or a health issue.

4.10.6 *Selection of protection systems*

4.10.6.1 Protection systems including coatings, liners, linings, and sealants shall be resistant to chemicals that are expected to be found in environmental engineering concrete structures, and in contact with them.

4.10.6.2 Protection systems shall be able to bridge existing open cracks, moving cracks that open and close due to thermal and moisture changes, or joints in the substrate.

R4.10.6 *Selection of protection systems*

R4.10.6.1 Liners are sheets that are attached to concrete and linings are applied protection systems to concrete surface.

R4.10.6.2 Cracks or joints may require special treatment before the application of a protection system.

4.11—Tightness testing of structures

When required by the licensed design professional, concrete containment structures that are designed to be liquid- or gas-tight shall be tightness tested in accordance with the procedures in ACI 350.1. Test procedures used shall be consistent with the design conditions of the containment structure.

R4.11—Tightness testing of structures

ACI 350.1 covers various tightness testing procedures and criteria for environmental engineering concrete structures. The Specification includes hydrostatic, surcharged hydrostatic, pneumatic, and combination hydrostatic-pneumatic tests. The procedure and criteria used should be based on the design requirements for the structure. Refer to ACI 350.1 for liquid-tightness and gas-tightness requirements.

Tightness testing of structures applies to structures that are required to contain liquid, or gas, or both.

The tightness testing requirements in ACI 350.1 provide performance requirements that measure both the cracking and permeability performance of the completed structure. The licensed design professional should establish an acceptable amount of leakage for the tightness testing. This should be done in consultation with the owner.

4.12—Joints

4.12.1 *General*

Joints shall be designed to prevent cracking, spalling, reinforcement corrosion, and chemical attack. The number, spacings, and details of movement joints (expansion and contraction) 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. Refer to Chapter 7 for additional design requirements.

4.12.2 *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.

4.12.2.1 Waterstops shall be of materials that have demonstrated performance under conditions and applications similar to those anticipated. Materials used for waterstops shall be able to sustain movement and deformations (elongation and contraction) without permanent deformation or failure and shall be resistant to freezing-and-thawing cycles,

R4.12—Joints

R4.12.1 *General*

Refer to Chapter 7 and ACI 504R for the design and details of joints.

R4.12.2 *Waterstops*

R4.12.2.1 Some waterstops do not offer suitable resistance to high temperatures, fuels, oils, solvents, hydrocarbons, and high concentrations of some acids. These exposures may require the use of a thermoplastic elastomeric robber (TPE-R) or stainless steel waterstop material. Refer to 3.6 for the proper testing requirements for validating the acceptability of the

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and temperature and chemical effects for the expected exposure conditions.

4.12.2.2 For ozone contact structures, waterstop shall be a low-carbon-grade of stainless steel or a fully vulcanized thermoplastic elastomeric rubber (TPE-R) material that has been tested as required by 4.11.4.

4.12.3 *Sealants*

Sealants shall not debond or degrade under the expected exposure and shall be resistant to the required pressures, temperatures, and movements. Refer to 4.8.2.

4.12.4 *Ozone exposure*

Sealants, joint fillers, and waterstops to be used shall be tested for compatibility with ozone.

COMMENTARY

selected waterstop material. Refer to Chapter 4 for chemical and temperature resistance conditions.

R4.12.2.2 PVC waterstops do not offer suitable resistance to the ozone concentrations found in many ozone contact structures. In ozone environments, it may be required to specify stainless steel or TPE-R waterstops. Independent testing and history of performance has shown TPE-R waterstops to be suitable for use in many ozone environments. TPE-R offers several advantages over stainless steel waterstops such as ease of installation, elimination of numerous waterstop splices, availability, and cost. To allow for the expected movement, metal waterstops at expansion joints need to be crimped at the center of the waterstop. Recent testing has shown that TPE-R waterstops are suitable for ozone exposure up to 7000 ppm.

R4.12.3 *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. Two-component polyurethane sealants are recommended.

When a polysulfide sealant will be exposed to wastewater conditions, the sealant manufacturer should provide a written recommendation and independent test results that verify the sealant will be suitable for wastewater exposure.

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 inspect, repair, maintain, and reseal joints with proper joint sealants at proper intervals.

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CHAPTER 5—CONCRETE QUALITY, MIXING, AND PLACING

5.1—General

Although shotcrete is a specific type of concrete, with respect to quality, mixing, and placing, the provisions for shotcrete are frequently different from those for other concrete. Where the provisions in this chapter are indicated specifically for shotcrete, they shall apply in lieu of the similar provisions for concrete. Where the provisions within this chapter apply to concrete, and include shotcrete, the words "concrete and shotcrete" or "concrete and wet-mix shotcrete" are used for enhanced clarity. Where the provisions in this chapter are indicated only to apply to concrete, they shall not apply to shotcrete. In this chapter, the notations f_c', f_{cr}', and s_s shall apply to concrete, excluding shotcrete.

5.1.1 *Concrete*

5.1.1.1 Concrete shall be proportioned to provide an average compressive strength f_{cr}' as prescribed in 5.3.2 and shall satisfy the durability requirements of Chapter 4. Concrete shall be produced to minimize frequency of strength tests below f_c' as prescribed in 5.7.10.3.

5.1.1.2 The requirements for f_c' shall be based on tests of cylinders made and tested as prescribed in 5.3.3.2 for trial mixtures, and 5.7.10.1 and 5.7.10.2 for field test results.

5.1.1.3 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 the contract documents.

5.1.1.4 Where design criteria in 8.6.1 and 12.8.2.4(d) provide for use of a splitting tensile strength value of concrete, f_{ct}, laboratory tests shall be made in accordance with ASTM C330 to establish a value of f_{ct} corresponding to f_c'.

COMMENTARY

CHAPTER R5—CONCRETE QUALITY, MIXING, AND PLACING

R5.1—General

The requirements for proportioning of concrete and shotcrete mixtures are based on the philosophy that they should provide both adequate durability (Chapter 4) and strength. The criteria for acceptance of concrete and shotcrete 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 and shotcrete of adequate strength can be obtained, and provides procedures for checking the quality of the concrete and shotcrete during and after its placement in the work.

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

The purpose of 5.3 and 5.4, together with Chapter 4, is to establish the required mixture proportions, and not to constitute a basis for confirming the adequacy of concrete or shotcrete strength, which is covered in 5.7 (evaluation and acceptance of concrete and shotcrete).

The basic premises governing the designation and evaluation of concrete and shotcrete strength are presented. It is emphasized that the average strength produced shall always exceed the specified value of f_c' for concrete, and f_g' for shotcrete, used in the structural design computations. This is based on probabilistic concepts and is intended to ensure that adequate strength will be developed in the structure. The durability requirements prescribed in Chapter 4 shall be satisfied in addition to attaining the average strength in accordance with 5.3.2 and 5.4.2 for concrete and shotcrete, respectively.

R5.1.1.4 Equations throughout the Code that contain the $\sqrt{f_c'}$ term have been modified, as necessary, for use with lightweight concrete. Two alternative modification procedures are provided. One alternative is based on laboratory tests to determine the relationship between average splitting tensile strength f_{ct} and specified compressive strength f_c' for the lightweight concrete. For a lightweight aggregate from a

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given source, it is intended that appropriate values of f_{ct} be obtained in advance of the design.

5.1.1.5 Splitting tensile strength tests shall not be used as a basis for field acceptance of concrete.

R5.1.1.5 Tests for splitting tensile strength of concrete (as required by 5.1.1.4) are not intended for control of, or acceptance of, the strength of concrete in the field. Indirect control will be maintained through normal compressive strength test requirements provided by 5.7.

5.1.2 *Shotcrete*

5.1.2.1 Shotcrete shall be proportioned to provide an average compressive strength as prescribed in 5.4.2 and shall satisfy the durability requirements of Chapter 4. Shotcrete shall be produced to minimize frequency of compressive strengths below f_g' as prescribed in 5.7.11.3.

5.1.2.2 For wet-mix shotcrete:

(a) When using trial mixtures the requirements for f_g' shall be based on either tests of cylinders that are made and tested as prescribed in 5.4.3.2.d, or cores or cubes taken from test panels that are made and tested as prescribed in 5.4.3.2.e.

(b) When using field test results the requirements for f_g' shall be based on cores or cubes taken from test panels that are made and tested as prescribed in 5.7.11.1 and 5.7.11.2.

5.1.2.3 For dry-mix shotcrete, the requirements for f_g' shall be based on cores or cubes taken from test panels that are made and tested as prescribed in 5.4.3.2.e for trial mixtures, and 5.7.11.1 and 5.7.11.2 for field test results.

5.1.2.4 Unless otherwise specified, f_g' shall be based on 28-day tests. If other than 28 days, test age for f_g' shall be as indicated in the contract documents.

5.2—Selection of proportions

5.2.1 *Concrete*

5.2.1.1 Proportions of materials for concrete shall be established to:

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

(b) Meet applicable durability requirements in Chapter 4

(c) Conform to strength test requirements of 5.7

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

5.2.1.3 Concrete proportions, including water-cementitious materials ratio (***w/cm***), shall be established in accordance with 5.3.

5.2.2 *Shotcrete*

R5.2—Selection of proportions

Recommendations for selecting proportions for concrete are given in detail in ACI 211.1. The Standard provides two methods for selecting and adjusting proportions for normalweight concrete: the estimated weight and absolute volume methods. Example computations 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 ACI 211.2. The Standard provides a method of proportioning and adjusting structural grade concrete containing lightweight aggregates.

Recommendations for selecting proportions for shotcrete are given in detail in ACI 506R.

The selected water-cementitious materials ratio (***w/cm***) shall be low enough, and the compressive strength high enough, to satisfy both the strength criteria and the durability requirements of Chapter 4 of the Code. ACI 350 includes provisions for especially severe exposures, such as acids or high temperatures, but aesthetic considerations, such as surface finishes, are beyond the scope of the Code and

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5.2.2.1 Proportions of materials for shotcrete shall be established to:

(a) Provide workability and consistency to permit shotcrete to be built-up onto receiving surfaces and around reinforcement under conditions of placement to be employed, without sloughing or excessive rebound

(b) Meet applicable durability requirements in Chapter 4

(c) Conform to strength test requirements of 5.7

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

5.2.2.3 Shotcrete proportions, including ***w/cm***, shall be established in accordance with 5.4.

5.2.2.4 Shotcrete in contact with prestressed reinforcement shall consist of one part portland cement and not more than three parts fine aggregate by mass.

5.2.2.5 Shotcrete not in contact with prestressed reinforcement shall consist of one part portland cement and not more than four parts fine aggregate by mass.

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

COMMENTARY

should be covered specifically in the contract documents. Concrete and shotcrete ingredients and proportions should be selected to meet the minimum requirements prescribed in the Code and the additional requirements of the contract documents.

ACI 350 restricts the methods for selecting concrete and shotcrete mixture proportions to field experience or trial mixtures. Proportioning on the basis of ***w/cm*** as described in 5.4 of ACI 318 is not permitted by ACI 350.

R5.3—Proportioning concrete on the basis of field experience or 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 is 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 flowchart 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 water-cementitious materials ratio (***w/cm***).

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

The ACI 350 limit of 12 months for test records and trial batches differs from the ACI 318-11 requirement of 24 months. Due to the variation of materials over time and special durability concerns, a 12-month period is more appropriate for environmental engineering concrete structures.

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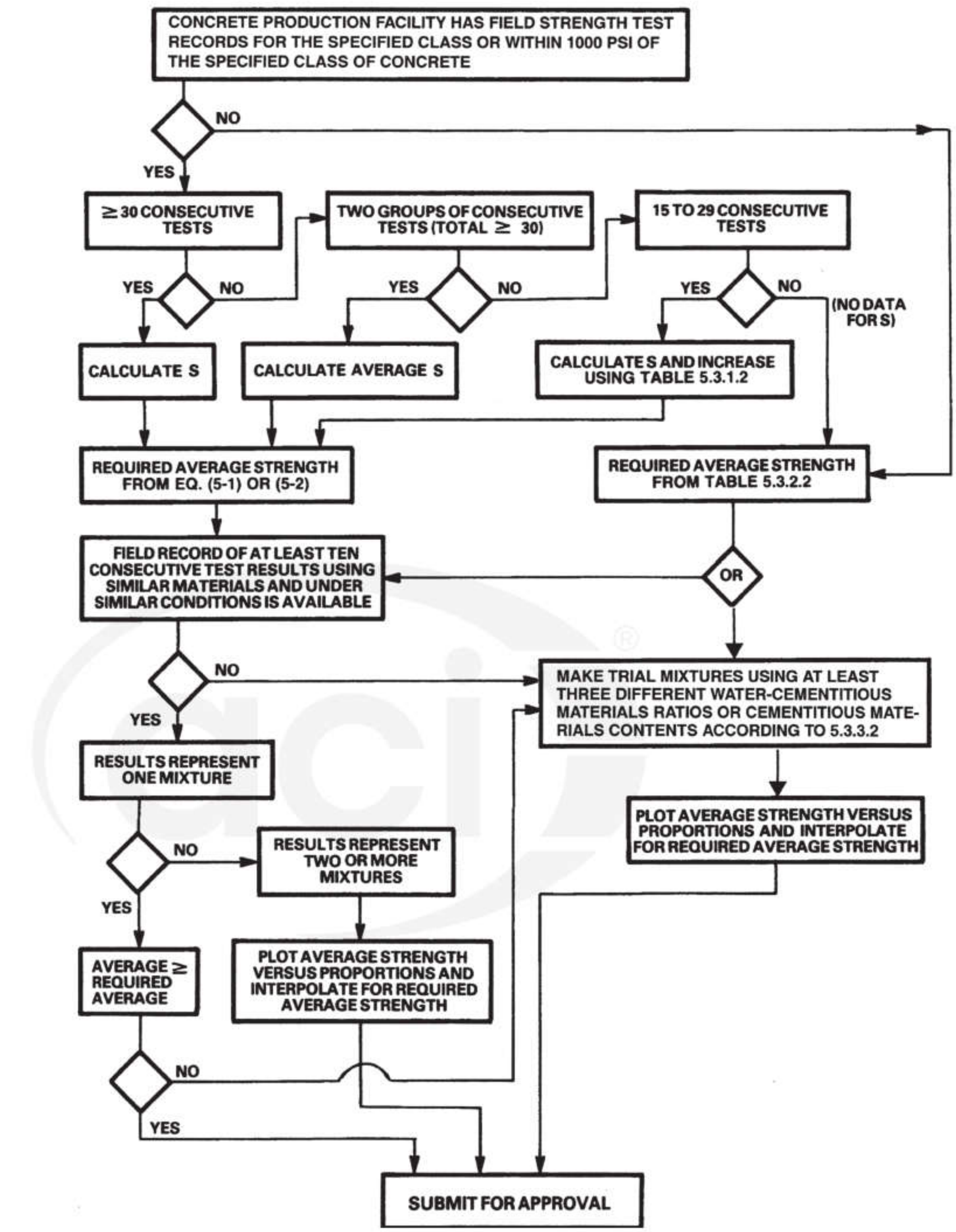

Fig. R5.3—Flowchart for selection and documentation of concrete proportions.

5.3.1 *Sample standard deviation for concrete*

Where the concrete production facility to be used for the work has field strength test records no more than 12 months old, and spanning a period of not less than 45 calendar days, a sample standard deviation s_s shall be established. Test records from which s_s is computed:

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

R5.3.1 *Sample standard deviation for concrete*

The standard deviation established from test records is a measure of the concrete supplier's ability to manage variability of materials, production, and testing of concrete. A test record obtained less than 12 months before a submittal is acceptable.

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

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

where s is sample standard deviation, psi; X_i is individual strength tests as defined in 5.7.4; $\bar{X}$ is average of n strength test results; and n is number of consecutive strength tests.

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

If less than 30 but at least 15 tests are available, the computed sample standard deviation is increased by the factor given in Table 5.3.1. This procedure results in a more conservative (increased) required average strength. The factors in Table 5.3.1 are based on the sampling distribution of the sample 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 sample standard deviation.

The sample standard deviation used in the computation of required average strength shall be developed under conditions "similar to those expected" (refer to 5.3.1(a)). This requirement is important to ensure acceptable concrete.

Concrete for background tests to determine sample 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' (refer to 5.3.1(b)). A change in the type of concrete may increase the sample standard deviation. Such a situation might occur with a change in type of aggregate (that is, from normalweight 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 sample standard deviation when the average strength level is raised by a significant amount, although the increment of increase in sample standard deviation should be somewhat less than directly proportional to the strength increase. When there is reasonable doubt, any estimated sample standard deviation used to compute the required average strength should always be on the conservative (high) side.

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Note that the Code uses the sample 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 shall exceed the design strength by an amount per Table 5.3.2.2.

Even when the average strength and sample standard deviation are of the levels assumed, there will be occasional tests that fail to meet the acceptance criteria prescribed in 5.7.10.3 (perhaps 1 test in 100).

(b) Shall represent concrete produced to meet a specified compressive strength or strengths within 1000 psi of f_c' when used only to determine s_s

R5.3.1(b) Field strength test records within 1000 psi are suitable for determination of a sample standard deviation but should not be used to select proposed concrete proportions for the project. If test records are available that meet or exceed the specified compressive strength, they may be used to determine sample standard deviation and the proposed concrete proportions.

(c) Shall consist of at least 30 consecutive tests, or two groups of consecutive tests totaling at least 30 tests, each group not less than 10 tests, except as provided in (d). A test is defined in 5.7.4. When two groups of tests are used, s_s shall be computed as the statistical average of both groups

(d) Or, where the concrete production facility does not have test records meeting requirements of (c), but does have a single record comprising 15 to 29 consecutive tests, s_s shall be established as the product of the computed sample standard deviation and modification factor of Table 5.3.1

Table 5.3.1—Modification factor for sample standard deviation for concrete

No. of tests*	Modification factor for sample 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 sample standard deviation, s_s, to be used to determine required average strength f_{cr}' from 5.3.2.1.

5.3.2 *Required average compressive strength for concrete*

R5.3.2 *Required average compressive strength for concrete*

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 the sample standard deviation s_s, computed in accordance with 5.3.1.

R5.3.2.1 Once the sample 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

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Table 5.3.2.1—Required average compressive strength when data are available to establish a sample standard deviation for concrete

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) $f_{cr}' = f_c' + 1.34s_s$ (5.1) $f_{cr}' = f_c' + 2.33s_s - 500$ (5.2)
$f_c' > 5000$	Use the larger value computed from Eq. (5.1) and (5.3) $f_{cr}' = f_c' + 1.34s_s$ (5.1) $f_{cr}' = 0.90f_c' + 2.33s_s$ (5.3)

5.3.2.2 When the concrete production facility to be used for the work does not have field strength test records for computation of s_s meeting requirements of 5.3.1, 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 sample standard deviation for concrete

Specified compressive strength, f_c', psi	Required average compressive strength, f_{cr}', psi
$4000 \leq f_c' \leq 5000$	$f_c' + 1200$
$f_c' > 5000$	$1.10f_c' + 700$

5.3.3 *Documentation of average compressive strength for concrete*

Documentation that proposed concrete proportions will produce an average compressive strength equal to or greater than required average compressive strength, f_{cr}' (refer to 5.3.2), shall consist of field strength test records or trial mixtures. The field strength test records or trial mixtures shall not be more than 12 months old and shall conform to 5.3.3.1 and 5.3.3.2, respectively.

COMMENTARY

may be less than $0.90f_c'$. These equations assume that the sample standard deviation used is equal to the population value appropriate for an infinite or very large number of tests. For this reason, use of sample 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.

R5.3.3 *Documentation of average compressive strength for concrete*

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 requirements for applicable exposure categories and classes of Chapter 4. The documentation may consist of strength test records in accordance with 5.3.3.1, or suitable laboratory trial mixtures in accordance with 5.3.3.2. Generally, if a test record is used, it will be the same one that was used for computation of the standard deviation. However, if this test record shows either lower or higher average strength than f_{cr}', 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 field test records for establishing proportions necessary to produce the average strength shall meet the requirements of 5.3.1 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}' computed in Table 5.3.2.2 to allow for a reduction in strength from laboratory trials to actual concrete production.

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5.3.3.1 Test records used to demonstrate that the concrete proportions will achieve f_{cr}' shall conform to 5.3.1 requirements, with the following exceptions:

(a) Tests shall represent concrete produced to meet or exceed f_c'

(b) A minimum of 10 consecutive tests is required

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 the other requirements of this section.

5.3.3.2 When an acceptable record of field test results to document the required average strength is not available, concrete proportions established from trial mixtures meeting the following requirements shall be permitted:

(a) Materials shall be those proposed for the work

(b) Trial mixtures (at least two) shall have a range of proportions that will meet the applicable durability requirements of Chapter 4 and produce a range of compressive strengths encompassing f_{cr}'

(c) Trial mixtures shall have slumps within the range specified for the proposed work, and for air-entrained concrete, air content shall be within the tolerance specified for the proposed work

(d) For each trial mixture, at least three 4 x 8 in. or two 6 x 12 in. test cylinders for each test age shall be made and cured in accordance with ASTM C192. Cylinders shall be tested at 28 days or at a test age specified for designated f_c'.

(e) The compressive strength results shall be used to establish the composition of the concrete mixture proposed for the work. The proposed mixture shall achieve an average compressive strength as required in 5.3.2 and satisfy the applicable durability requirements of Chapter 4.

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

5.4.1 *Sample standard deviation for shotcrete*

COMMENTARY

R5.3.3.1 Field strength test records used to select proposed concrete proportions should represent concrete produced to meet or exceed the specified compressive strength. Strength test records used to document strength should represent concrete mixtures comprising materials to be used in the proposed mixture also, as acceptable to the licensed design professional.

R5.3.3.2(b) For concrete made with more than one type of cementitious material, the concrete supplier shall establish not only the water-cementitious materials ratio (***w/cm***), but also the relative proportions of cementitious materials and admixtures, if any, that will produce the required average compressive strength and satisfy the durability requirements of Chapter 4. This will require multiple trial batches with different mixture proportions. The exact number of batches will depend on the number of cementitious materials and the range of their relative proportions.

R5.3.3.2(d) The Code permits either of two cylinder sizes for preparing test specimens for both field acceptance and trial mixture testing.

R5.4—Proportioning of shotcrete on the basis of field experience or trial mixtures, or both

Many principles of concrete and shotcrete proportioning are similar, particularly in the wet-mix process; however, differences should be recognized before proportioning mixtures. In-place shotcrete has a higher cement factor than the proportioned mixture because of the loss of coarse aggregate in the shooting process. This loss of aggregate, called "rebound," eliminates a certain percentage of the coarse aggregate in the in-place material, resulting in a finer aggregate gradation. This effect can increase the potential for shrinkage cracking and should be considered in the proportioning of the mixture.

R5.4.1 *Sample standard deviation for shotcrete*

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Where the shotcrete production facility to be used for the work has field strength test records no more than 12 months old and spanning a period of not less than 45 calendar days, a sample standard deviation s_{sg} shall be established. Test records from which s_{sg} is computed:

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

COMMENTARY

The standard deviation established from test records is a measure of the shotcrete supplier's ability to manage variability of materials, production, and testing of shotcrete. A test record obtained less than 12 months before a submittal is acceptable.

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

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

where s_g is sample standard deviation, psi; X_i is individual strength tests as defined in 5.7.5; $\bar{X}$ is average of n strength test results; and n is number of consecutive strength tests.

The sample standard deviation is used to determine the average strength required in 5.4.2.1.

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

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

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

If less than 30 but at least 15 tests are available, the computed standard deviation is increased by the factor given in Table 5.4.1. This procedure results in a more conservative (increased) required average strength. The factors in Table 5.4.1 are based on the sampling distribution of the sample 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 sample standard deviation.

The sample standard deviation used in the computation of required average strength shall be developed under conditions "similar to those expected" (refer to 5.4.1(a)). This requirement is important to ensure acceptable shotcrete.

Shotcrete for background tests to determine sample 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_g' (refer to 5.4.1(b)). A change in the type of shotcrete may increase the sample standard deviation. Such a situation might occur with a change in type of aggregate (that is, from

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normalweight aggregate to lightweight aggregate or vice versa) or a change from non-air-entrained shotcrete to air-entrained concrete. Also, there may be an increase in sample standard deviation when the average strength level is raised by a significant amount, although the increment of increase in sample standard deviation should be somewhat less than directly proportional to the strength increase. When there is reasonable doubt, any estimated sample standard deviation used to compute the required average strength should always be on the conservative (high) side.

Note that the Code uses the sample 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 shall exceed the design strength by an amount per Table 5.4.2.2.

Even when the average strength and sample standard deviation are of the levels assumed, there will be occasional tests that fail to meet the acceptance criteria prescribed in 5.7.11.3 (perhaps 1 test in 100).

(b) Shall represent shotcrete produced to meet a specified compressive strength or strengths of shotcrete within 1000 psi of f_g' when used only to determine s_{sg}

R5.4.1(b) Field strength test records within 1000 psi are suitable for determination of a sample standard deviation but should not be used to select proposed shotcrete proportions for the project. If test records are available that meet or exceed the specified compressive strength, they may be used to determine sample standard deviation and the proposed shotcrete proportions.

(c) Shall consist of at least 30 consecutive tests, or two groups of consecutive tests totaling at least 30 tests, each group not less than 10 tests, except as provided in (d). A test is defined in 5.7.5. When two groups of tests are used, s_{sg} shall be computed as the statistical average of both groups

(d) Or, where the shotcrete production facility does not have test records meeting requirements of (c), but does have a single record comprising 15 to 29 consecutive tests, s_{sg} shall be established as the product of the computed sample standard deviation and modification factor of Table 5.4.1

Table 5.4.1—Modification factor for sample standard deviation for shotcrete

No. of tests*	Modification factor for sample standard deviation for shotcrete†
Less than 15	Use Table 5.4.2.2
15	1.16
20	1.08
25	1.03
30 or more	1.00

*Interpolate for intermediate numbers of tests.

†Modified sample standard deviation s_{sg} to be used to determine required average strength f_{gr}' from 5.4.2.1.

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5.4.2 *Required average compressive strength for shotcrete*

5.4.2.1 Required average compressive strength of shotcrete f_{gr}', used as the basis for selection of shotcrete proportions, shall be determined from Table 5.4.2.1 using the sample standard deviation s_{sg} computed in accordance with 5.4.1.

Table 5.4.2.1—Required average compressive strength when data are available to establish a sample standard deviation for shotcrete

Specified compressive strength f_g', psi	Required average compressive strength of shotcrete, psi
$f_g' \leq 5000$	Use the larger value computed from Eq. (5.4) and (5.5) $f_{gr}' = f_g' + 1.34s_{sg}$ (5.4) $f_{gr}' = f_g' + 2.33s_{sg} - 500$ (5.5)
$f_g' > 5000$	Use the larger value computed from Eq. (5.4) and (5.6) $f_{gr}' = f_g' + 1.34s_{sg}$ (5.4) $f_{gr}' = 0.90f_g' + 2.33s_{sg}$ (5.6)

5.4.2.2 When the shotcrete production facility to be used for the work does not have field strength test records for computation of s_{sg} meeting requirements of 5.4.1, f_{gr}' shall be determined from Table 5.4.2.2 and documentation of average strength shall be in accordance with requirements of 5.4.3.

Table 5.4.2.2—Required average compressive strength when data are not available to establish a sample standard deviation for shotcrete

Specified compressive strength, f_g', psi	Required average compressive strength of shotcrete, f_{gr}', psi
$4000 \leq f_g' \leq 5000$	$f_g' + 1200$
$f_g' > 5000$	$1.10f_g' + 700$

5.4.3 *Documentation of average compressive strength for shotcrete*

Documentation that proposed shotcrete proportions will produce an average compressive strength equal to or greater than required average compressive strength, f_{gr}' (refer to 5.4.2), shall consist of field strength test records or trial mixtures. The field strength test records or trial mixtures shall not be more than 12 months old and shall conform to 5.4.3.1 and 5.4.3.2, respectively.

COMMENTARY

R5.4.2 *Required average compressive strength for shotcrete*

R5.4.2.1 Once the standard deviation has been determined, the required average compressive strength is obtained from the larger value computed from Eq. (5.4) and (5.5) for f_g' of 5000 psi or less, or the larger value computed from Eq. (5.4) and (5.6) for f_g' over 5000 psi. Equation (5.4) is based on a probability of 1-in-100 that the average of three consecutive tests may be below the specified compressive strength f_g'. Equation (5.5) is based on a similar probability that an individual test may be more than 500 psi below the specified compressive strength f_g'. Equation (5.6) is based on the same 1-in-100 probability that an individual test may be less than $0.90f_g'$. 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 shotcrete will be produced.

R5.4.3 *Documentation of average compressive strength for shotcrete*

Once the required average strength f_{gr}' 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 requirements for applicable exposure categories and classes of Chapter 4. The documentation may consist of strength test records in accordance with 5.4.3.1, or suitable laboratory or field trial mixtures in accordance with 5.4.3.2. Generally, if a test record is used, it will be the same one that was used for computation of the standard deviation. However, if this test record shows either lower or higher average strength than f_{gr}', different proportions may be necessary or desirable. In such instances, the average from a record of as few as 10 tests may be used. All field test records for establishing proportions necessary to produce

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5.4.3.1 Test records used to demonstrate that the concrete proportions will achieve f_{gr}' shall conform to 5.4.2 requirements, with the following exceptions:

(a) Tests shall represent shotcrete produced to meet or exceed f_g'

(b) A minimum of 10 consecutive tests is required

Required shotcrete proportions shall be permitted to be established by interpolation between the strengths and proportions of two or more test records, each of which meets the other requirements of this section.

5.4.3.2 When an acceptable record of field test results to document the required average strength is not available, shotcrete proportions established from trial mixtures meeting the following requirements shall be permitted:

(a) Materials shall be those proposed for the work

(b) Trial mixtures (at least two) shall have a range of proportions that will meet the applicable durability requirements of Chapter 4, and produce a range of compressive strengths of shotcrete encompassing f_{gr}'

(c) For wet-mix shotcrete, either the requirements of (d) or (e) shall be followed. For dry-mix shotcrete, the requirements of (e) shall be followed

(d) Test cylinders:

(1) Trial mixtures shall have slumps within the range specified for the proposed work, and for air-entrained shotcrete, air content shall be within the tolerance specified for the proposed work

(2) For each trial mixture, at least three 4 x 8 in. or two 6 x 12 in. test cylinders for each test age shall be made and cured in accordance with ASTM C192. Cylinders shall be tested at 28 days or at a test age specified for designated f_{gr}'

(e) Test panels:

For each trial mixture, separate test panels shall be fabricated and tested in accordance with 5.7.11.1 and 5.7.11.2

(f) The compressive strength results shall be used to establish the composition of the shotcrete mixture proposed for the work. The proposed mixture shall achieve an average compressive strength as required in 5.4.2 and satisfy the applicable durability criteria of Chapter 4

5.5—Average compressive strength reduction for concrete

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

(a) Fifteen or more test results are available and the average of test results exceeds that required by 5.3.2.1 using

COMMENTARY

the average strength shall meet the requirements of 5.4.1 for "similar materials and conditions."

For strengths over 5000 psi where the average strength documentation is based on trial mixtures, it may be appropriate to increase f_{gr}' computed in Table 5.4.2.2 to allow for a reduction in strength from trials to actual shotcrete production.

R5.4.3.1 Field strength test records used to select proposed shotcrete proportions should represent shotcrete produced to meet or exceed the specified compressive strength. Strength test records used to document strength should represent shotcrete mixtures comprising materials to be used in the proposed mixture also, as acceptable to the licensed design professional.

R5.4.3.2 Test cylinders are not practical for dry-mix shotcrete, so cores of test panels of trial mixtures are required. Caution is advised when using test cylinders instead of cores from test panels for wet-mix shotcrete trial mixtures because there can be problems in duplicating cylinder test results in the field.

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a sample standard deviation computed in accordance with 5.3.1, and

(b) Requirements for applicable durability requirements in Chapter 4 are met

5.6—Average compressive strength reduction for shotcrete

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

(a) Fifteen or more test results are available and the average of the test results exceeds that required by 5.4.2.1, using a sample standard deviation computed in accordance with 5.4.1, and

(b) Requirements for applicable durability requirements in Chapter 4 are met

5.7—Evaluation and acceptance of concrete and shotcrete

5.7.1 Concrete and shotcrete shall be evaluated for compliance with all requirements of this Code. The testing agency performing acceptance testing shall comply with ASTM C1077. Qualified laboratory technicians shall perform all required laboratory tests. Compressive strength shall be only one of the criteria used for evaluation and acceptance. The results of all tests performed on the concrete or shotcrete and other data and information pertaining to handling, placing, and curing shall be used to evaluate compliance with the requirements of this Code. All reports for acceptance tests shall be provided to the licensed design professional, contractor, concrete producer, and when requested, to the owner and the building official.

5.7.2 Concrete shall be evaluated in accordance with the requirements of 5.7.9, 5.7.10, 5.7.12, and 5.7.13.

5.7.3 Shotcrete shall be evaluated in accordance with the requirements of 5.7.11 through 5.7.13.

5.7.4 A strength test for concrete shall be the average of the compressive strengths of at least two 6 x 12 in. cylinders or at least three 4 x 8 in. cylinders made from the same sample of concrete and tested at 28 days or at a test age designated for determination of compressive strength. Test cylinder size

COMMENTARY

R5.7—Evaluation and acceptance of concrete and shotcrete

Once the mixture proportions have been selected and the project started, the criteria for evaluation and acceptance of the concrete or shotcrete can be obtained from 5.7.

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

R5.7.1 ASTM C1077 identifies and defines the duties and minimum technical requirements and qualifications of testing laboratory personnel and requirements for testing concrete and concrete aggregates used in construction. Inspection and accreditation of testing laboratories is a process that ensures that they conform to ASTM C1077.

The Code requires testing reports to be distributed to the licensed design professionals, parties responsible for the construction, and parties responsible for acceptance of the work. Such distribution of test reports should be indicated in contracts for inspection and testing services. Prompt distribution of testing reports allows for timely identification of either compliance or the need for corrective action. A complete record of testing allows the concrete producer to reliably establish the required average strength f_{cr}' for future work.

R5.7.4 More than the minimum number of specimens may be desirable to allow for discarding an outlying individual cylinder strength in accordance with ACI 214R. When individual cylinder strengths are discarded in accordance with ACI 214R, a strength test is valid provided at least two individual 6 x 12 in. cylinder strengths or at least three 4 x 8 in.

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shall remain constant throughout the work for each design mixture of concrete used.

5.7.5 A strength test for shotcrete shall be the average of the compressive strengths of at least three cores or cubes taken from a single test panel and tested at 28 days or at a test age designated for determination of compressive strength. Test specimen size shall remain constant throughout the work for each design mixture of shotcrete used.

5.7.6 Qualified field testing technicians shall perform tests on fresh concrete at the job site, prepare concrete specimens, including any that may be required for curing under field conditions, and transport specimens to the laboratory. Qualified laboratory technicians shall demold the specimens, cure the specimens, and perform all required laboratory tests.

5.7.7 Qualified field testing technicians shall perform tests on fresh shotcrete at the job site. Qualified nozzlemen shall apply shotcrete to the test panels. Qualified technicians shall cure and prepare, and perform testing of the hardened specimens.

5.7.8 *Frequency of testing*

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

5.7.8.2 Samples for density tests of each design mixture of wet-mix shotcrete placed each day shall be taken each time a test panel for strength tests is prepared.

COMMENTARY

cylinder strengths are averaged. All individual strengths that are not discarded in accordance with ACI 214R are to be used to compute the average strength. The size and number of specimens representing a strength test should remain constant for each class of concrete.

Testing three 4 x 8 in. cylinders preserves the confidence level of the average strength because 4 x 8 in. cylinders tend to have approximately 20 percent higher within-test variability than 6 x 12 in. cylinders.

R5.7.6 Laboratory and field technicians can establish qualifications by becoming certified through certification programs. Field technicians in charge of sampling concrete; testing for slump, density, yield, air content, and temperature; and making and curing test specimens should be certified in accordance with the requirements of ACI Concrete Field Testing Technician—Grade 1 Certification Program, or the requirements of ASTM C1077 or an equivalent program. Concrete testing laboratory personnel should be certified in accordance with the requirements of ACI Concrete Laboratory Testing Technician, Concrete Strength Testing Technician, or the requirements of ASTM C1077.

R5.7.7 Nozzlemen should be certified in accordance with the ACI Certification program or other approved nozzlemen certification program. Technicians should be certified in accordance with the respective ACI Certification program or other approved certification program.

R5.7.8 *Frequency of testing*

R5.7.8.1 Samples for strength tests shall be taken on a strictly random basis if they are to measure properly the acceptability of the concrete and shotcrete. To be representative, the choice of times of sampling, or the batches to be sampled, shall 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 (refer to 5.7.4 and 5.7.5) should be taken from a single batch, and no adjustments, including the addition of water, may be made after the sample is taken.

ASTM D3665 describes procedures for random selection of the batches to be tested.

R5.7.8.2 The following three criteria establish the required sampling frequency for each class of concrete:

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

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

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

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 engineering concrete structures complies with the more stringent requirements of the ACI 350 Code. It is very important that a higher level of quality control be maintained.

5.7.8.3 Test panels for strength tests of each wet-mix and dry-mix shotcrete design mixture placed each day shall be taken not less than once a day, nor less than once for each 50 yd^3 of shotcrete.

5.7.8.4 On a given project, if the total volume of concrete or shotcrete is such that frequency of testing required by 5.7.8.1 or 5.7.8.3 would provide less than five strength tests for a given design mixture of concrete or shotcrete, tests shall be made from at least five randomly selected batches, or one test from each batch if fewer than five batches are used.

5.7.9 *Density (unit weight)*

R5.7.9 *Density (unit weight)*

Density testing of concrete provides additional quality assurance that the mixture received at the site is the same mixture as that submitted.

5.7.9.1 Samples for concrete density tests shall be taken in accordance with ASTM C172. Density tests shall be in accordance with ASTM C138.

5.7.9.2 The field density of concrete shall be considered satisfactory if no individual density test varies by more than ±2 percent of the density of the design mixture.

5.7.9.3 If the requirement of 5.7.9.2 is not met, steps shall be taken to adjust the density to agree with the density of the design mixture.

5.7.10 *Standard-cured cylinder specimens*

R5.7.10 *Standard-cured cylinder specimens*

5.7.10.1 Samples for concrete strength tests shall be taken in accordance with ASTM C172.

5.7.10.2 Cylinders for strength tests for concrete shall be molded, transported, and standard-cured in accordance with ASTM C31, and tested in accordance with ASTM C39.

5.7.10.3 The strength level of an individual design mixture of concrete shall be considered satisfactory if both of the following requirements are met:

R5.7.10.3 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

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(a) Every arithmetic average of any three consecutive strength tests (refer to 5.7.4) equals or exceeds f_c', and

(b) No strength test (refer to 5.7.4) falls below f_c' by more than 500 psi when f_c' is 5000 psi or less, or by more than $\mathbf{0.10}f_c'$ when f_c' is more than 5000 psi

5.7.10.4 If the requirements of 5.7.10.3 are not met, steps shall be taken to increase the average of subsequent strength test results. Requirements of 5.7.13 shall be observed if the requirement of 5.7.10.3(b) is not met.

COMMENTARY

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 a minimum individual strength test result of 500 psi less than f_c' adapts itself readily to small numbers of tests. For example, if only five strength tests are made on a small project, 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.

R5.7.10.4 When concrete fails to meet either of the strength requirements of 5.7.10.3, steps shall 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 of the mixture 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

(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 included in new submittal of proposed mixture.

Laboratories testing cylinders or cores to determine compliance with these requirements should be accredited for conformance to the requirement of ASTM C1077 by a recognized agency such as the American Association for Laboratory Accreditation (A2LA), AASHTO Materials Reference Laboratory (AMRL), National Voluntary Labora-

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CODE

COMMENTARY

tory Accreditation Program (NVLAP), Cement and Concrete Reference Laboratory (CCRL), or their equivalent.

5.7.11 *Laboratory-cured core or cube specimens for shotcrete*

R5.7.11 *Laboratory-cured core or cube specimens for shotcrete*

5.7.11.1 Sample cores or cubes for strength tests shall be taken and tested from panels prepared and laboratory-cured in accordance with ASTM C1140. The test panels shall be a minimum 24 x 3-1/2 in. and shall have flared sides unless cores are taken at least half a core diameter away from the form sides. The test panels shall be cored or sawn to obtain 3 in. diameter cores or 3 in. square cubes. The cores or cubes shall be tested for compressive strength.

5.7.11.2 A length-to-diameter correction factor shall be applied to compressive strengths of cores from test panels in accordance with ASTM C42. For sawed cubes, compressive strengths shall be multiplied by a correction factor of 0.85 to obtain the equivalent strength of drilled cores.

5.7.11.3 The strength level of an individual design mixture of shotcrete 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_g'

(b) No individual strength test (refer to 5.7.5) falls below f_g' by more than 500 psi when f_g' is 5000 psi or less; or by more than $\mathbf{0.10}f_g'$ when f_g' is more than 5000 psi

R5.7.11.3 A single set of criteria is given for acceptability of strength and is applicable to all shotcrete used in structures designed in accordance with the Code, regardless of design method used. The shotcrete strength is considered to be satisfactory as long as averages of any three consecutive strength tests remain above the specified f_g' and no individual strength test falls below the specified f_g' by more than 500 psi if f_g' is 5000 psi or less, or falls below f_g' by more than 10 percent if f_g' is over 5000 psi. Evaluation and acceptance of the shotcrete 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 (approximately 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_g' adapts itself readily to small numbers of tests. For example, if only five strength tests are made on a small project, it is apparent that if any of the strength test results (average of two cores or cubes) is more than 500 psi below f_g', the criterion is not met.

5.7.11.4 If the requirements of 5.7.11.3 are not met, steps shall be taken to increase the average of subsequent strength test results. Requirements of 5.7.13 shall be observed if requirement of 5.7.11.3(b) is not met.

R5.7.11.4 When shotcrete fails to meet either of the strength requirements of 5.7.11.3, steps shall be taken to increase the average of the shotcrete test results. If sufficient shotcrete has been produced to accumulate at least 15 tests, these should be used to establish a new target average strength as described in 5.4.

If fewer than 15 tests have been made on the class of shotcrete 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

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COMMENTARY

selection of proportions, a further increase in average level is required.

5.7.12 *Field-cured specimens*

5.7.12.1 If required by the building official or licensed design professional, results of strength tests of specimens cured under field conditions shall be provided.

5.7.12.2 Specimens shall be cured under field conditions in accordance with ASTM C31.

5.7.12.3 Field-cured test specimens shall be made at the same time and from the same concrete or shotcrete samples as laboratory-cured test specimens.

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

R5.7.12 *Field-cured specimens*

R5.7.12.1 Strength tests of specimens cured under field conditions may be required to check the adequacy of curing and protection of concrete or shotcrete in the structure.

R5.7.12.4 Positive guidance is provided in the Code concerning the interpretation of tests of field-cured specimens. Research has shown that specimens protected and cured to simulate good field practice should test not less than approximately 85 percent of standard laboratory moist-cured specimens. 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 field-cured and laboratory-cured specimens, not between field-cured specimens and the specified value of f_c' or f_g'. However, results for the field-cured specimens are considered satisfactory if the field-cured cylinders exceed the specified f_c' or f_g' by more than 500 psi, even though they fail to reach 85 percent of the strength of companion laboratory-cured specimens.

5.7.13 *Investigation of low-strength test results*

5.7.13.1 If any strength test (refer to 5.7.10 and 5.7.11) of laboratory-cured specimens falls below specified value of f_c' or f_g' by more than the values given in 5.7.10.3(b) or 5.7.11.3(b), or if tests of field-cured cylinders indicate deficiencies in protection and curing (refer to 5.7.12), steps shall be taken to assure that load-carrying capacity and durability of the structure is not jeopardized.

5.7.13.2 If the likelihood of low strength is confirmed and computations indicate that load-carrying capacity is reduced, tests of cores drilled from the area in question in accordance with ASTM C42 shall be permitted. In such cases, three cores shall be taken for each strength test that falls below the values given in 5.7.10.3(b) or 5.7.11.3(b).

5.7.13.3 Cores shall be obtained, moisture-conditioned by storage in watertight bags or containers, transported to the laboratory, and tested in accordance with ASTM C42. Cores shall be tested no earlier than 48 hours before coring and no later than 7 days after coring unless approved by the licensed design professional. The specifier of tests referenced in ASTM C42 shall be the licensed design professional.

R5.7.13 *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. These instructions are applicable only for evaluation of in-place strength at time of construction. Strength evaluation of existing structures is covered by Chapter 22. The building official or licensed design professional should apply judgment as to the 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 or shotcrete in place, such as by probe penetration, impact hammer, ultrasonic pulse velocity or pullout, may be useful in determining whether a portion of the structure actually contains low-strength concrete or shotcrete. Such tests are of value primarily for comparisons within the same project 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 (Bloem 1965, 1968; Malhotra 1976, 1977). Lower strength may, of course, be tolerated under many circumstances, but this again becomes a matter of judg-

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

5.7.13.5 If criteria of 5.7.13.4 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 22 for the questionable portion of the structure, or take other appropriate action.

5.7.13.6 If structural adequacy is confirmed, refer to Chapter 22 for assessment of durability requirements.

COMMENTARY

ment on the part of the building official and licensed design professional. 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 22). Short of load tests, if time and conditions permit, an effort may be made to improve the strength of the concrete or shotcrete in place by supplemental wet curing. Effectiveness of such a treatment shall 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 being created during drilling. This adversely affects the core's compressive strength (Bartlett and MacGregor 1994). The restriction on the commencement of core testing provides a minimum time for the moisture gradient to dissipate. The maximum time between coring and testing is intended to ensure timely testing of cores when strength of concrete is in question. Bartlett and MacGregor (1994) has also shown that procedures for soaking or drying cores that were required before ACI 318-02 affect measured compressive strength and result in conditions that are not representative of structures that are dry and wet in service. Thus, to provide reproducible moisture conditions that are representative of in-place conditions, a common moisture conditioning procedure that permits dissipation of moisture gradients is prescribed for cores. ASTM C42 permits the specifier of tests to modify the default duration of moisture conditioning before testing.

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' or f_g' is not realistic, because differences in the size of specimens, conditions of obtaining specimens, 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 5.7.13 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'.

5.8—Preparation of equipment and place of deposit

5.8.1 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 coated with release agent

(d) Reinforcement shall be thoroughly cleaned of ice and other deleterious materials

(e) All laitance and other unsound material shall be removed and the surface prepared as required by the licensed

R5.8—Preparation of equipment and place of deposit

R5.8.1 Recommendations for mixing, handling and transporting, and placing concrete are given in detail in ACI 304R. The guide 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 shall be removed. Reinforcement shall be thoroughly

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design professional before concrete is placed against hardened concrete

(f) Beams, girders, and slabs supported by columns or walls shall not be cast until concrete in the vertical support members achieve initial set

(g) If new concrete is to be placed against existing concrete that is over 7 days old, the existing concrete shall be in a saturated surface-dry (SSD) condition immediately prior to placement of the new concrete

(h) All freestanding water shall be removed from place of deposit before concrete is placed unless a tremie is to be used

5.8.2 Preparation before shotcrete placement shall include the following:

(a) All equipment for placing shotcrete shall be clean

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

(c) Forms to be removed shall be coated with release agent, and adequately braced and secured to prevent excessive vibration or deflection during the placement of shotcrete

(d) Reinforcement shall be thoroughly cleaned of ice and other deleterious materials

(e) All loose and unsound previously placed shotcrete shall be removed

(f) All previously placed shotcrete shall be roughened with laitance removed

(g) Shotcrete substrates shall be in a saturated surface-dry (SSD) condition immediately prior to application

(h) All freestanding water shall be removed

(i) Reinforcement shall be sized, spaced, and arranged to facilitate the placement of shotcrete

(j) Depth gauges or ground wires shall be placed to ensure specified cover over reinforcing bars

5.8.2.1 When shotcrete is shot against integral structural metal diaphragms, the diaphragm shall be braced and supported in a manner sufficient to eliminate vibrations that would impair the bond between the diaphragm and the shotcrete.

5.9—Mixing

5.9.1 All concrete and wet-mix shotcrete shall be mixed until there is a uniform distribution of materials and shall be discharged completely before mixer is recharged.

5.9.2 Ready mixed concrete and wet-mix shotcrete shall be mixed and delivered in accordance with requirements of ASTM C94 or ASTM C685.

5.9.3 Site-mixed concrete and wet-mix shotcrete shall be mixed in accordance with the following (a) through (e):

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

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

COMMENTARY

cleaned of ice, dirt, loose rust, mill scale, or other coatings. Water should be removed from the forms.

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

R5.8.2 Delivery and placement of wet-mix and dry-mix shotcrete are vastly different than for concrete. Recommendations for mixing and placing shotcrete are given in detail in ACI 506R. Attention is directed to the need for proper spacing of reinforcement and adequate preparation of each layer of applied shotcrete.

R5.9—Mixing

Uniform and satisfactory quality of concrete and wet-mix shotcrete 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 density, air content, slump, and coarse aggregate content. Test methods for uniformity of mixing are given in ASTM C94. The necessary time of mixing will depend on many factors, including batch size, stiffness of the batch, 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.

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(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 ASTM C94

(d) Materials handling, batching, and mixing shall conform to applicable provisions of ASTM C94

(e) For each placement, a detailed record shall be kept to identify:

(1) the number of batches produced
(2) the proportions of materials used
(3) the approximate location of final deposit in structure
(4) the time and date of mixing and placement

5.10—Conveying concrete and wet-mix shotcrete

5.10.1 Concrete and wet-mix shotcrete shall be conveyed from mixer to location of placement by methods that will prevent separation or loss of materials.

5.10.2 Conveying equipment shall be capable of providing a supply of concrete and wet-mix shotcrete at the location of placement without separation of ingredients and without interruptions of sufficient length of time to permit loss of plasticity between successive increments.

COMMENTARY

R5.10—Conveying concrete and wet-mix shotcrete

Each step in the handling and transporting of concrete and wet-mix shotcrete 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 and of water from the other ingredients.

The Code requires the equipment for handling and transporting concrete or wet-mix shotcrete to be capable of supplying the mixture to the place of deposit continuously and reliably under all conditions and for all methods of placement. The provisions of 5.10 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 (Newlon and Ozol 1969). Hydrogen gas generated by the reaction between the cement alkalis 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 or wet-mix shotcrete from a truck mixer.

5.11—Depositing of concrete

5.11.1 Concrete shall be deposited as nearly as practical in its final position to avoid segregation.

5.11.2 Concrete placement shall be such that it is at all times plastic and flows readily into spaces between reinforcement.

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

5.11.4 Retempered concrete shall not be permitted.

5.11.5 After concrete placement is started, it shall be a continuous operation until placing of a panel or section, as defined by its boundaries or predetermined joints, is completed.

R5.11—Depositing of concrete

Rehandling concrete can cause segregation of the materials. Hence, the Code cautions against this practice. This does not preclude the practice (recognized in ASTM C94) of adding water to mixed concrete to bring it up to the specified slump range, as long as prescribed limits on the maximum mixing time and water-cementitious materials ratio (***w/cm***) are not violated.

Recommendations for consolidation of concrete are given in detail in ACI 309R. The guide presents current information on the mechanism of consolidation and gives recommendations on equipment characteristics and procedures for various classes of concrete.

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5.11.6 Top surfaces of vertically formed lifts shall be generally level.

5.11.7 When construction joints are required, joints shall be made in accordance with Chapter 7.

5.11.8 All concrete shall be thoroughly consolidated by suitable means during placement, including around reinforcement and embedments, and into corners of formwork.

5.12—Application of shotcrete

5.12.1 *Dry-mix shotcrete*

5.12.1.1 Shotcrete nozzlemen shall be certified in accordance with the ACI Shotcrete Nozzleman Certification program, or other shotcrete nozzleman certification program.

5.12.1.2 Shotcrete shall be delivered and applied using shotcrete guns that ensure a smooth, steady flow of material through the hose and nozzle.

5.12.1.3 The shotcrete nozzle operator shall maintain the mixing water to achieve required encasement of reinforcement and to avoid sagging, sloughing, or dropouts in the material.

5.12.1.4 The nozzle operator shall maintain the impact velocity required to ensure encasement of reinforcement.

5.12.2 *Wet-mix shotcrete*

5.12.2.1 The shotcrete shall be delivered to the nozzle at the rate and velocity needed for the required pneumatic application and encasement of reinforcement.

5.12.2.2 The nozzle operator shall ensure that the material is pneumatically applied at the correct slump and velocity to avoid sagging, sloughing, or dropouts in the material.

5.12.3 *Shotcrete on vertical surfaces*

Shotcrete on vertical surfaces shall be built up of individual layers of shotcrete, 3 in. or less in thickness.

5.12.4 *Shotcrete nozzle technique*

The nozzle for shotcrete application shall be held at a small upward angle not exceeding 5 degrees and shall be constantly moving, without shaking, and always pointing perpendicular to the surface receiving the application. The nozzle distance from the surface shall be such that shotcrete does not build up over or cover the front faces of the reinforcement, wires, and strands until the spaces between or behind them are filled.

5.13—Curing

5.13.1 Concrete and shotcrete shall be maintained above 50°F and 40°F, respectively, and in a moist condition for at least the first 7 days after placement, except:

COMMENTARY

R5.12—Application of shotcrete

The successful application of shotcrete depends on proper equipment and placement techniques. The dry-mix and wet-mix processes use different types of delivery equipment with different operating characteristics that can affect the choice of the shotcreting process, application, and quality of the shotcrete. Recommendations for proper application of dry-mix and wet-mix shotcrete are given in detail in ACI 506R.

R5.13—Curing

Recommendations for curing concrete are given in detail in ACI 308R. The guide presents basic principles of proper curing and describes the various methods, procedures, and

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a) as indicated in 5.13.2, or
b) when cured in accordance with 5.13.3, or
c) when cured in cold weather per 5.14, or
d) as otherwise specified by the licensed design professional

5.13.2 High-early-strength concrete and shotcrete shall be maintained above 50°F and 40°F, respectively, and in a moist condition for at least the first 3 days after placement, except:
a) when cured in accordance with 5.13.3, or
b) when cured in cold weather per 5.14, or
c) as otherwise specified by the licensed design professional

5.13.3 *Accelerated curing*

5.13.3.1 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.

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

5.13.3.3 Curing process shall be such as to produce concrete and shotcrete with durability at least equivalent to that produced by the curing method of 5.13.1 or 5.13.2.

5.13.4 When required by the licensed design professional, supplementary strength tests in accordance with 5.7.13 shall be performed to assure that curing is satisfactory.

COMMENTARY

materials for curing of concrete. Recommendations for curing shotcrete are given in detail in ACI 506R.

R5.13.3 *Accelerated curing*

The provisions of this section apply whenever an accelerated curing method is used, whether for precast, cast-in-place, or shotcrete elements. The compressive strength of steam-cured concrete or shotcrete is not as high as that of similar material 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 mixture proportions on steam-cured test specimens.

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

R5.13.4 In addition to requiring a minimum curing temperature and time for normal- and high-early-strength concrete and shotcrete, the Code provides a specific criterion in 5.7.12 for judging the adequacy of field curing. At the test age for which the strength is specified (usually 28 days), field-cured specimens should produce strength not less than 85 percent of that of the standard, laboratory-cured specimens. For a reasonably valid comparison to be made, field-cured specimens and companion laboratory-cured specimens shall come from the same sample. Field-cured specimens shall be cured under conditions identical to those of the structure. If the structure is protected from the elements, the specimens should be similarly protected, and specimens representing 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 specimens should not be treated more favorably than the elements they represent (refer to 5.7.12 for additional information).

If the field-cured specimens 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 in the structure, core tests may be required, with or without supplemental wet curing, to check the structural adequacy as provided in 5.7.13.

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5.14—Cold weather requirements

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

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

5.14.3 Frozen materials or materials containing ice shall not be used.

5.15—Hot weather requirements

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

COMMENTARY

R5.14—Cold weather requirements

Recommendations for cold weather concreting are given in detail in ACI 306R. ACI 306.1 is the specification for cold weather concreting.

Shotcrete has a greater heat of hydration than conventional cast-in-place concrete because of its higher cement factor. Although this aids in resisting freezing, the placement of shotcrete in thin layers counterbalances the heat of hydration benefits. When shotcrete will be placed under cold conditions, a plan should be developed outlining procedures for surface preparation, shotcrete placement, curing, and protection. Refer to ACI 506R and ACI 506.2.

R5.15—Hot weather requirements

Recommendations for hot weather concreting are given in detail in ACI 305R. The guide defines the hot weather factors that affect concrete properties and construction practices and recommends measures to eliminate or minimize the undesirable effects. ACI 305.1 is the specification for hot weather concreting.

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CHAPTER 6—FORMWORK AND EMBEDMENTS

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 contract documents.

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 the previously placed structure.

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

(a) Rate and method of placing concrete or shotcrete

(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 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.

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*

COMMENTARY

CHAPTER R6—FORMWORK AND EMBEDMENTS

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 ACI 347R, "Guide to Formwork for Concrete," and *Formwork for Concrete* (Hurd 2005) reported by ACI Committee 347. ACI 347R 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 in preparing project specifications. *Formwork for Concrete* is a how-to-do-it handbook for contractors, engineers, and architects following the guidelines established in ACI 347R. Planning, building, and using formwork are discussed, including tables, diagrams, and formulas for formwork 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.

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 (Liu et al. 1989). The construction loads are frequently at least as much 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.

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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 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 the remaining forming and shoring system has sufficient strength to safely support its weight and loads placed thereon.

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

COMMENTARY

Other methods to evaluate compressive strength during construction may include:

(a) Tests of cast-in-place cylinders in accordance with ASTM C873. (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 ASTM C803

(c) Pullout strength in accordance with ASTM C900

(d) Maturity factor measurements and correlation in accordance with ASTM C1074

Procedures (b), (c), and (d) require sufficient data, using project 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 C150) or ASTM C595 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 multi-story construction should be a part of a planned procedure considering the temporary support of the entire structure as well as that of each individual member. Such a procedure should be worked out prior to 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 multi-story construction, the strength of the concrete during the various stages of construction should be substantiated by field-cured test specimens or other approved methods.

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6.3—Embedments in concrete and shotcrete

6.3.1 Embedments of any material not harmful to concrete or shotcrete or within limitations of 6.3 shall be permitted in concrete and shotcrete with the acceptance of the licensed design professional, provided they are not considered to structurally replace the displaced concrete and shotcrete, except as provided in 6.3.6.

6.3.2 Aluminum embedments in concrete and shotcrete shall be isolated from the concrete by coating or other form of protective covering to prevent aluminum-concrete reaction or electrolytic action between aluminum and steel.

6.3.3 Embedments passing through a slab, wall, or beam shall not impair the integrity of the construction.

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 otherwise approved by the licensed design professional, conduits and pipes embedded within a slab, wall, or beam (other than those merely passing through) shall satisfy 6.3.5.1 through 6.3.5.3:

6.3.5.1 They shall not be larger in outside dimension than one-third 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 the integrity of the structure.

6.3.6 Conduits, pipes, and sleeves shall be permitted to be considered as replacing the displaced compression zone concrete or shotcrete provided requirements of 6.3.6.1 through 6.3.6.3 are met.

6.3.6.1 They are not exposed to corrosion or other deterioration.

COMMENTARY

R6.3—Embedments in concrete and shotcrete

R6.3.1 Any embedments—for example, conduits, pipes, and sleeves—not harmful to concrete or shotcrete can be placed in the concrete and shotcrete, but this 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 B31.1 piping codes for power piping and ANSI/ASME B31.3 for chemical and petroleum piping. The licensed design professional 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 embedments that are not indicated on the contract documents or not approved by the licensed design professional.

For the integrity of the structure, it is important that all conduit and pipe fittings embedded within the concrete be carefully assembled as indicated on the contract documents.

R6.3.2 The Code prohibits the use of aluminum in concrete and shotcrete unless it is effectively coated. Aluminum reacts with concrete and shotcrete and, in the presence of chloride ions, can also react electrolytically with steel, causing cracking and spalling of the concrete and shotcrete.

R6.3.6 There have been some reported cases of extensive deterioration of 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 that may also cause

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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 Except where designed to intersect the surface, concrete cover over pipes, conduits, and fittings shall meet the cover requirements for reinforcement.

6.3.11 Reinforcement with an area not less than 0.002 times the area of the concrete or shotcrete section nor less than the area required by 12.13 of this Code shall be provided perpendicular to piping.

6.3.12 Design and detail structural elements to account for the placement of embedments.

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

6.3.14 Dissimilar metallic embedments shall be electrically isolated (not tied with electrically conductive materials) from each other and from dissimilar metallic reinforcement.

COMMENTARY

"hydrogen embrittlement" of certain steels (Yamaoka et al. 1988; Gibala and Haherman 1984).

R6.3.7 ACI 318-83 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 member.

R6.3.14 Care should be taken to avoid setting up destructive corrosion currents in the new structure. Any dissimilar metals that are electrically connected with tie wire or any conductive element will create an electrochemical or galvanic corrosion cell (galvanic action). Thus, all dissimilar metals should be electrically isolated. This is executed by tying dissimilar elements (such as galvanized piping, or steel piping) to forms and not to the reinforcement). If these elements are tied to reinforcement, tying with plastic ties and use of epoxy coatings or thick (10 mil) polymer tape to electrically isolate the materials in the area of contact may mitigate the galvanic corrosion cell. This is true for most dissimilar metals except for coupled stainless and carbon steel. Research has shown that galvanic action between coupled carbon and stainless steel embedded in concrete does not increase corrosion rates significantly. Indeed, corrosion rates occurring when carbon and stainless steels are coupled in a concrete member have been found to be less than that of corroding carbon steel when coupled with noncorroding carbon steel (Qian et al. 2005; Hope 2001).

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CHAPTER 7—JOINTS

7.1—Jointing

7.1.1 *General*

7.1.1.1 Joints shall be designed and located to reduce the potential for concrete cracking due to drying shrinkage, temperature changes, creep, differential subgrade settlement, and movement under loading conditions, as well as to accommodate scheduled interruptions in concrete placement or the positioning of abutting precast concrete elements.

7.1.1.2 Joints shall be designed to comply with the requirements of Chapter 4.

7.1.1.3 Joints shall be provided only where indicated and detailed on the contract documents or permitted by the licensed design professional.

7.1.2 *Joint definitions and details*

7.1.2.1 construction joint—an intentionally created interface between concrete placements.

7.1.2.2 monolithic construction joint—a construction joint in which the surface between concrete placements is prepared to enhance bond and shear transfer, and all reinforcement is continuous.

COMMENTARY

CHAPTER R7—JOINTS

R7.1—Jointing

R7.1.1 *General*

R7.1.1.1 Joints should be designed to prevent leakage, spalling, reinforcement corrosion, and to reduce cracking due to restrained shrinkage and temperature strains, as appropriate. The number, spacing, and details of joints should be designed taking full account of the physical properties and ability of the filler, sealant, and waterstop to withstand cyclic deformations where necessary.

R7.1.1.3 Joints at locations not intended by the licensed design professional may affect the performance and integrity of the structure and its ability to be liquid- or gas-tight. Movement joints should be located only where indicated in contract documents. Joint locations not indicated on contract documents should be approved by the licensed design professional prior to concrete placement. For work that is properly planned and performed under the contractor's means and methods, unplanned joints should not occur. However, the licensed design professional may consider such a possibility and provide written procedures that detail the actions to be taken if an unintended interruption of concrete placement results in an unplanned joint. What should be done will depend on the licensed design professional's judgement, the specific structure, and the location and serviceability requirements of the unplanned joint.

R7.1.2 *Joint definitions and details*

R7.1.2.1 Construction joints may also be contraction, expansion, and isolation joints, as they are typically located at terminations in concrete placement. When a construction joint is not a contraction, expansion, or isolation joint, it should be considered a monolithic construction joint (refer to R7.1.2.2).

R7.1.2.2 Historically, in the design of environmental engineering concrete structures, it has been common to consider the term "construction joint" as defined as a "monolithic construction joint" in this Code.

The joint surface should be prepared by roughening, wetting, application of bonding agent, or a combination of these procedures, as required by the licensed design professional. Care should be exercised with bonding agents to ensure the condition of the bonding agent, at all times during casting, will increase and not decrease bond of the fresh concrete to the hardened concrete surface. Refer to Fig. R7.1.2.2 for an example of a monolithic construction joint detail.

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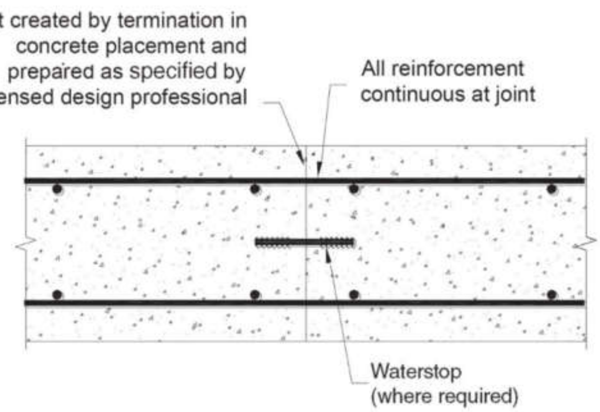

Fig. R7.1.2.2—Monolithic construction joint detail example.

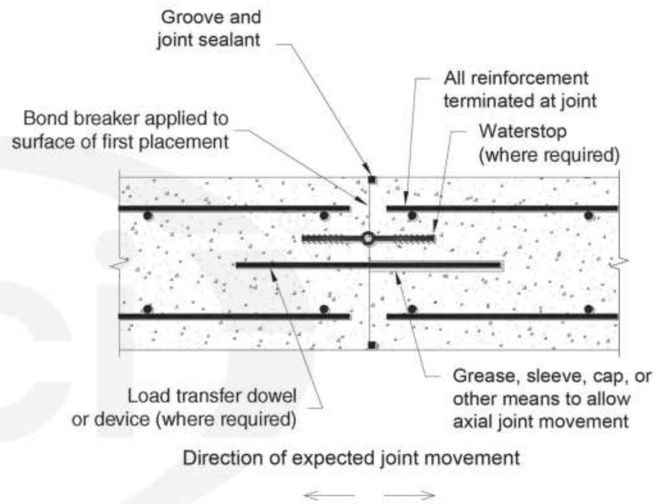

Fig. R7.1.2.3—Full contraction joint detail example.

7.1.2.3 full contraction joint—a formed interface provided between concrete placements to allow movement due to dimensional reduction of the adjacent sections, and through which all of the bonded reinforcement is interrupted.

R7.1.2.3 A full contraction joint is a movement joint that permits dimensional reduction of the concrete members adjacent to the joint surface. A bond breaker should be applied to the surface of the first concrete placement to prevent bond of the subsequent adjacent concrete section. Potential joint movement will create a gap between adjacent concrete sections. For structural concrete, sufficiently sized and spaced load transfer dowels are typically used to allow unrestrained axial joint movement. This is typically accomplished by the use of a greased dowel or the use of a hollow plastic dowel sleeve designed to allow for axial movement. Alternative load transfer devices may be used, provided that the manufacturer of such materials can provide sufficient test data showing that the product can accommodate the required load transfer and expected joint movement. To perform as intended, full contraction joints should experience negligible restraint from bonded reinforcement, requiring all such reinforcement to be terminated at the joint. Refer to Fig. R7.1.2.3 for an example of a full contraction joint detail.

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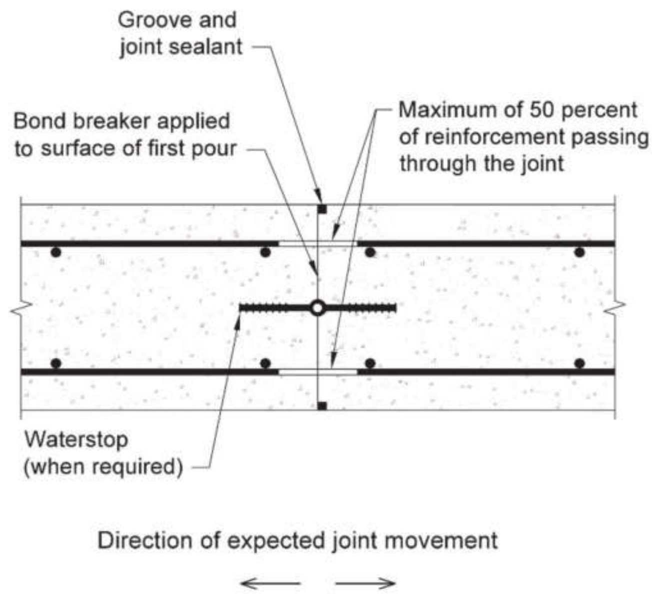

Fig. R7.1.2.4 —Partial contraction joint detail example.

7.1.2.4 partial contraction joint—a formed interface provided between concrete placements to allow movement due to dimensional reduction of the adjacent sections, and through which no more than 50 percent of the bonded reinforcement is continuous through the joint.

R7.1.2.4 A partial contraction joint is a movement joint that permits dimensional reduction of the concrete members adjacent to the joint surface. A bond breaker should be applied to the surface of the first concrete placement to prevent bond of the subsequent adjacent concrete section. Potential joint movement will create a gap between adjacent concrete sections. Due to the reinforcement passing through a partial contraction joint, these joints are less effective than full contraction joints for dissipating concrete volume changes. Thus, partial contraction joints, when used, need to be spaced closer together than full contraction joints to help control cracking from concrete volume changes. Refer to Fig. R7.1.2.4 for an example of a partial contraction joint detail.

7.1.2.5 crack-inducing joint—formed, sawed, or tooled groove in a concrete structure to create a weakened plane to regulate the location of cracking resulting from the dimensional reduction of adjacent sections of the structure.

R7.1.2.5 To further weaken the joint and induce cracking, it is desirable to terminate 50 percent of bonded reinforcement through the joint. A crack-inducing joint may be created by forming a groove into one or both faces of the concrete placement, sawcutting or hand-tooling a groove into the surface of the joint, inserting a crack-inducing joint forming strip into the plastic concrete, or using a combination of these methods. All these methods will create a weakened plane at the desired joint location, increasing the likelihood that a crack will form at the crack-inducing joint location. Refer to Fig. R7.1.2.5a, R7.1.2.5b, and R7.1.2.5c for examples of crack-inducing joint details.

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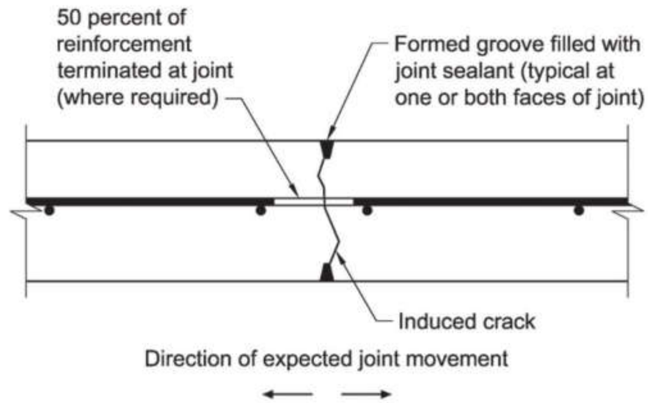

Fig. R7.1.2.5a—Crack-inducing joint detail example (with formed groove).

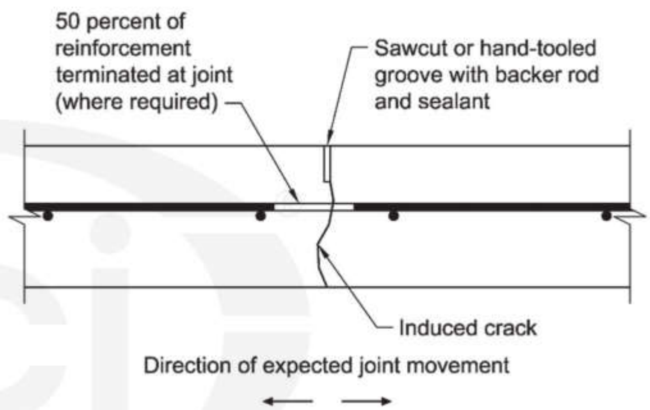

Fig. R7.1.2.5b—Crack-inducing joint detail example (with sawcut or hand-tooled groove).

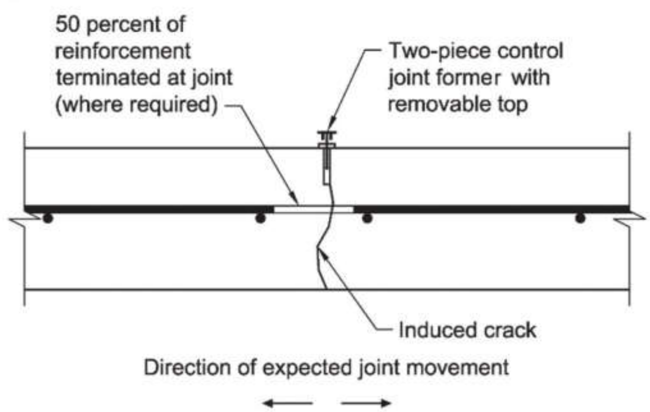

Fig. R7.1.2.5c—Crack-inducing joint detail example (with two-piece crack-inducing joint former).

7.1.2.6 expansion joint—a formed separation provided between concrete placements to allow movement due to dimensional increases and reductions of the adjacent sections, and through which all of the bonded reinforcement is interrupted.

R7.1.2.6 An expansion joint is considered a movement joint that permits dimensional reduction or expansion of the concrete members adjacent to the joint surface. All bonded reinforcement should be terminated at the joint. Joint movement as a result of dimensional reduction of adjacent concrete sections will increase the width of the gap between

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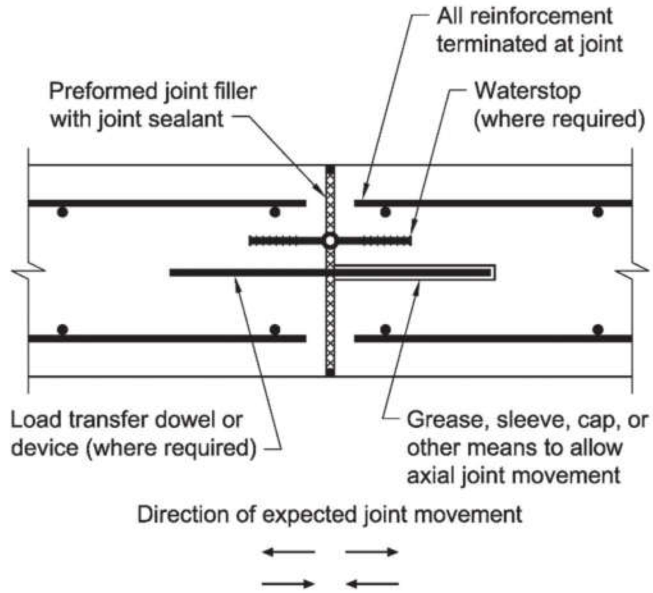

Fig. R7.1.2.6—Expansion joint detail example.

adjacent concrete sections. Joint movement as a result of expansion of adjacent concrete sections will decrease the width of the initial gap between adjacent concrete sections. Where required, sufficiently sized and spaced load transfer dowels may be used to allow unrestrained axial joint movement, typically accommodated by the use of a greased dowel with an end cap or the use of a hollow plastic dowel sleeve, both designed to allow for axial movement. Alternative load transfer devices may be used, provided that the manufacturer of such materials can provide sufficient test data showing that the product can accommodate the required load transfer and expected joint movement. Where load transfer devices are used, the use of corrosion-resistant material should be considered. Refer to Fig. R7.1.2.6 for an example of an expansion joint detail.

7.1.2.7 isolation joint—a formed separation between adjacent concrete placements to allow relative movement in all directions and, through which all the bonded reinforcement is interrupted.

R7.1.2.7 An isolation joint is a movement joint that permits dimensional reduction, dimensional expansion, or differential movement of the concrete members adjacent to the joint surface. An isolation joint will accommodate joint movement in any direction. All normal bonded reinforcement should be terminated at the joint to prevent the transfer of any loads across the joint. Where required, the waterstop should be a flexible elastomeric material to prevent the transfer of any loads across the joint. Joint movement as a result of dimensional reduction of adjacent concrete sections will increase the width of the gap between adjacent concrete sections. Joint movement as a result of expansion of adjacent concrete sections will decrease the width of the initial gap between adjacent concrete sections. Differential joint movement will cause adjacent concrete sections to slide parallel to the joint surface. Refer to Fig. R7.1.2.7 for an example of an isolation joint detail.

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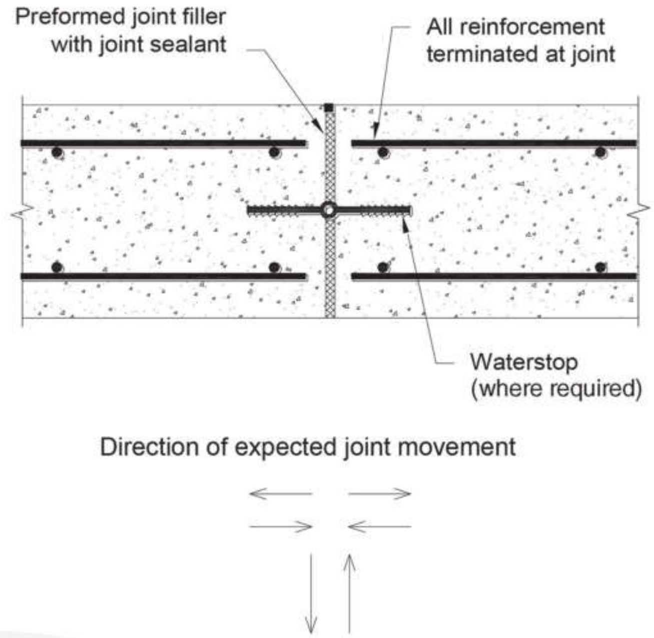

Fig. R7.1.2.7—Example isolation joint detail.

7.1.3 *Joint requirements—general*

R7.1.3 *Joint requirements—general*

7.1.3.1 Beams, girders, haunches, drop panels, and shear caps shall be cast with slabs in the same concrete placement, unless otherwise permitted by the licensed design professional.

R7.1.3.1 Separate placement of slabs and beams, haunches, and similar elements may be permitted when indicated on the contract documents and where provision has been made to transfer forces.

7.1.3.2 Joints that contain waterstops shall be configured such that the waterstop does not interfere with the reinforcement.

R7.1.3.2 Joint geometry and the position of the waterstop should be indicated in the contract documents. Consideration should be given to the clear distance between the waterstop and formwork and reinforcement, and the specified maximum aggregate size in order to allow concrete to be properly placed and consolidated.

7.2—Construction joints

7.2.1 Monolithic construction joints shall be detailed and located so as not to impair the integrity of the structure.

R7.2—Construction joints

R7.2.1 When shear due to gravity load is not significant, as is usually the case in the middle of the span of flexural members, a vertical joint may be adequate. Design for lateral forces may require special treatment of monolithic construction joints. Shear keys, intermittent shear keys, diagonal dowels, or the shear transfer method of 11.6 may be used whenever a force transfer is required.

7.2.2 Construction joints in girders shall be offset a minimum distance of two times the width of intersecting beams.

7.2.3 Joint preparation prior to subsequent concrete placement shall be specified by the licensed design professional.

R7.2.3 The surfaces of concrete at construction joints should be properly prepared.

The licensed design professional may specify sandblasting, hydroblasting, application of surface retarders, or additional procedures if conditions warrant. One common method to promote bonding of adjacent concrete placements is preparation of the surface of monolithic construction

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joints to saturated surface-dry condition prior to concrete placement.

When mortar batches are needed at the start of concrete placement, mortar proportions, congestion of reinforcement, vibrator access, and other factors should be considered.

When bonding agents are used, care should be exercised to ensure the condition of the bonding agent during concrete placement will increase and not decrease bond of the fresh concrete to the hardened concrete surface.

7.2.4 In wrapped prestressed structures, horizontal construction joints shall not be permitted in the core wall between the base and the top.

R7.2.4 The wall base joint, top joint, and vertical joints will be the only construction joints in wrapped prestressed walls.

7.2.5 Construction joints shall include a waterstop where intended to contain gases or liquids.

R7.2.5 A construction joint between the base of a wall and top of supporting slab or foundation may require the use of a starter wall at the joint interface to eliminate interference between the integral waterstop and top layer of reinforcement in the supporting slab or foundation. Alternatively, concrete cover on the top layer of reinforcement in the slab or foundation may be increased to achieve the same results. For positioning of a waterstop at the base of a wall, an upturned keyway is not recommended because of the potential of a crack forming through the keyway width or emanating from the top edge of the waterstop.

7.3—Crack-inducing joints

7.3.1 Crack-inducing joints shall be created with the use of one or more of the following methods:

(a) Formed groove
(b) Sawcut or hand-tooled groove
(c) Inserted T-shaped joint former with removable top

R7.3—Crack-inducing joints

R7.3.1 Figures R7.1.2.5a through R7.1.2.5c illustrate examples of each crack-inducing joint forming method.

Sawcut joints should be made as soon as possible without damaging concrete, to minimize random cracking.

7.3.2 Total reduction in concrete member thickness to create a crack inducing joint shall be specified by the licensed design professional.

R7.3.2 To create a crack-inducing joint at the desired location, a sufficient reduction in the thickness of the concrete element is required at the location of the crack-inducing joint. Experience has shown that a 25 percent reduction in concrete thickness will effectively create an induced crack. Crack-inducing joints are generally used in concrete elements containing only one layer of reinforcement. It can be difficult to consistently create a crack-inducing joint in concrete elements greater than 12 in. thick at the intended location with traditional crack-inducing joint forming methods.

7.3.3 Crack-inducing joint groove shall be provided with joint sealant.

R7.3.3 To prevent the intrusion of foreign materials into the crack-inducing joint, and to protect reinforcement, a joint sealant is required. A backer rod may be necessary to control the required depth of the joint sealant.

7.3.4 Crack-inducing joints shall be prohibited where liquid or gas tightness is required.

R7.3.4 Experience has shown that cracks formed at crack-inducing joints do not always propagate to the desired location on the waterstop, which may cause leaks. Proper centering of the waterstop and accurate crack-inducing joint forming are difficult to execute consistently and the waterstop can be difficult to secure in place so that it is not displaced during concrete placement. Proper concrete consolidation is

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often overlooked at crack-inducing joints because concrete is placed continuously through the joint and not stopped at a bulkhead. In shotcrete applications, it can be difficult to apply shotcrete behind the waterstop.

7.4—Movement joints

7.4.1 Expansion joints, full and partial contraction joints, isolation joints, and crack-inducing joints shall all be considered movement joints.

7.4.2 The licensed design professional shall consider and provide for volume changes of the structure.

7.4.3 Expansion and isolation joints shall include a compressible preformed joint filler and, where intended to contain gases or liquids, a waterstop. A joint sealant shall also be provided at horizontal expansion and isolation joints exposed to liquids.

7.4.4 Full and partial contraction joints shall include a waterstop where intended to contain gases or liquids. A joint sealant shall also be provided at horizontal contraction joints designed to contain liquids. The joint sealant shall be placed in a preformed groove or recess.

7.4.5 The concrete interface of full and partial contraction joint surfaces shall be provided with a bond breaker or other means to prevent bond between initial and subsequent concrete placements.

7.4.6 Shear keys shall be prohibited in isolation joints.

R7.4—Movement joints

R7.4.1 Expansion joints, full and partial contraction joints, crack-inducing joints, and isolation joints are designed to allow for joint movement in one or more directions, as indicated in the figures in R7.1.2. Information on the design and detailing of movement joints is included in ACI 350.4R and ACI 224.3R.

R7.4.2 Volume changes in concrete are generally caused by changes in temperature, moisture content, or both. Movement joints (expansion joints, isolation joints, crack-inducing joints, and full and partial contraction joints) may be used to accommodate the resultant displacement(s), particularly at changes in configuration or locations of restraint.

R7.4.3 Joint sealants are required at horizontal expansion and isolation joints exposed to liquids to prevent the intrusion of foreign materials into the joint that may restrict anticipated joint movement, damage the waterstop, or impair the overall performance of the joint. Joint sealants may also be used at vertical expansion and isolation joints exposed to liquids for similar reasons, although exposure of such joints is not as critical because foreign materials are less likely to settle into the joint. Vertical joint sealants may be more prone to debonding and dislodging with the potential to foul process equipment or flow control devices.

R7.4.4 Joint sealants are required at horizontal contraction joints exposed to liquids to prevent the intrusion of foreign materials into the joint that may restrict anticipated joint movement, damage the waterstop, or impair the overall performance of the joint. Joint sealants may also be used at vertical contraction joints exposed to liquids for similar reasons, although such exposure of such joints is not as critical because foreign materials are less likely to settle into the joint. Vertical joint sealants may be more prone to debonding and dislodging with the potential to foul process equipment or flow control devices. Joint sealant may also be advisable at the buried or unexposed surface of vertical contraction joints to prevent the intrusion of foreign materials into the joint.

R7.4.6 Isolation joints require that no load is transferred across the joint.

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7.4.7 Shear keys shall be prohibited in expansion and full contraction joints.

7.4.8 For expansion and full contraction joints, shear load transfer shall be achieved with smooth, corrosion-resistant load transfer dowels or devices, with one-half of the dowel or device lubricated or inserted into a sleeve to allow for axial movement. For expansion joints, the load-transfer dowel or device must be able to allow expansion and contraction demand on the joint.

R7.4.8 Load transfer dowels for expansion and full contraction joints should be smooth and have sawcut ends. Sheared ends often distort the geometry of the dowel, potentially preventing the dowel from accommodating axial movement. Load transfer dowels should be aligned perpendicular to the joint to prevent movement restraint. Load transfer dowels for expansion joints should be able to accommodate expansion and contraction, which typically requires the use of a dowel sleeve or hollow cap with a void between the end of the dowel and the sleeve/cap no smaller than the maximum amount of joint movement anticipated. Load transfer for contraction joints should simply allow for one-half of the dowel to slide axially, typically accomplished with a dowel sleeve, bond breaker, or grease.

7.4.9 Circular prestressed structures using a sliding wall base joint shall include a bearing pad and flexible waterstop. Sponge filler shall be used when pads are not continuous.

7.5—Joint accessories

7.5.1 *Waterstop*

7.5.1.1 Waterstops shall be of materials that have demonstrated acceptable performance under conditions and applications similar to those anticipated.

R7.5—Joint accessories

R7.5.1 *Waterstop*

To maintain the integrity of a liquid- or gas-tight joint, a waterstop is required to prevent the passage of the liquid or gas.

When applicable to the waterstop type (such as PVC, TPE-R, or rubber), the waterstop should be tied to adjacent reinforcement with light gauge tie wire looped through metal rings, eyelets, or holes pre-punched into the waterstop. It is common to tie the waterstop at a 12 in. spacing. Concrete placement directly onto a waterstop can displace the waterstop from its intended position, preventing proper alignment within the joint. It may also cause the waterstop to fold over, compromising the waterstop performance.

Concrete placed adjacent to joints should be thoroughly consolidated to ensure intimate concrete contact with the entire waterstop surface. Effective waterstop performance is highly dependent on proper consolidation of adjacent concrete. Honeycombing between the waterstop and concrete can provide a path for fluids to bypass the waterstop as well as increase the possibility of waterstop pullout from adjacent concrete in movement joints.

When a waterstop is required between an existing concrete structure and a new concrete placement, mechanically anchored "T"- or "L"-shaped waterstops designed specifically for retrofit applications may be used. When no movement is expected at the joint, and the joint will be continuously exposed to moisture, swellable strip-applied waterstops may also be acceptable. Injectable hose waterstops have also been used in joints between new and existing concrete. When using swellable strip-applied waterstop materials, injectable hose waterstops, or other alternative waterstop materials, the minimum concrete coverage around the waterstop should comply with the manufacturer's

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CODE

7.5.1.2 Waterstop size shall be selected based on head pressure resistance, construction tolerances, and concrete element thickness.

7.5.1.3 Waterstops for movement joints shall be of materials and geometry that are able to accommodate the maximum amount of expected joint movement.

COMMENTARY

requirements to prevent concrete spalling when expansive pressures are exerted by the waterstop.

R7.5.1.2 A variety of waterstop materials have been used in construction joints with success. These materials include flexible PVC, rubber, carbon steel, stainless steel, injectable hoses, and swellable strip materials. The adequacy of the waterstop material is dependent on many factors such as duration of moisture exposure, compatibility with liquids being contained, reinforcement interference, service temperature requirements, and installation experience of the installer. No single waterstop material is the most suitable material for every application.

R7.5.1.3 When selecting traditional waterstop materials such as PVC, rubber, steel, or fully vulcanized thermoplastic elastomeric rubber (TPE-R), the following should be considered:

(a) Although head pressure ratings would suggest a 4 in. wide waterstop is acceptable for many environmental engineering concrete structure applications, it is good design practice to use a waterstop of minimum 6 in. width for monolithic construction joints and contraction joints to allow installation tolerances.

(b) For expansion or isolation joints, a minimum 9 in. width waterstop is typically used because the effective waterstop width is decreased by the thickness of the preformed expansion or isolation joint material.

(c) The licensed design professional should verify that the selected waterstop will withstand the maximum head pressure to which the waterstop will be exposed.

(d) For concrete elements less than 6 in. thick at monolithic construction joints and contraction joints, a 4 in. wide waterstop may be necessary to accommodate minimum concrete coverage requirements around the waterstop. For concrete elements less than 9 in. thick at expansion and isolation joints, a 4 or 6 in. wide waterstop may be necessary for the same reason. The waterstop should not interfere with reinforcement or dowels within the joint, potentially dictating a decreased waterstop width in joints for thinner concrete elements.

(e) Minimum concrete coverage around the waterstop is typically no less than one-half of the waterstop width completely confined in concrete. However, for wrapped prestressed concrete tanks, history has shown that concrete coverage can be less than one-half of the embedded width of the waterstop because of special construction practices and concrete placement and consolidation techniques.

(f) When considering PVC waterstop materials, it is also good design practice to select profiles with a minimum thickness of 3/8 in. at the center of the waterstop profile for waterstop widths 6 in. or greater. For 4 in. PVC waterstop or TPE-R waterstop of any width, a minimum waterstop thickness of 3/16 in. at the center of the waterstop profile is common. TPE-R waterstop is more rigid than PVC, allowing the use of thinner profiles that are not likely to fold over

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during concrete placement. For stainless steel waterstop, history has shown successful performance with 20-gauge thickness.

(g) When considering a PVC or TPE-R waterstop, ribbed waterstop profiles are more desirable than dumbbell profiles, as the ribs provide a greater wetted waterstop perimeter and improved interlock with adjacent concrete.

When selecting swellable strip-applied waterstop materials, injectable hose waterstops, or other alternative waterstop materials, waterstop size should be selected based on the manufacturer's published head pressure ratings.

For PVC, rubber, and TPE-R waterstops, it is common to select a profile with a hollow centerbulb or tearweb bulb to accommodate joint movement. For centerbulb or tearweb bulb profiles, the licensed design professional should verify that the waterstop can accommodate the expected maximum joint movement. For rigid materials such as stainless steel waterstop, the center of the waterstop should include a bent "V" shape that will accommodate the maximum joint movement. Verify with the waterstop manufacturer before selecting a swellable or strip-style or injectable hose waterstop for movement joints. These waterstop materials are not typically suitable for movement joints.

CODE

7.5.1.4 Split-flange waterstops shall not be used in joints required to be liquid- or gas-tight.

COMMENTARY

R7.5.1.4 Due to the inability to use split-flange waterstops where directional changes of joints or intersections between two or more joints occur, split-flange waterstops should not be used.

CODE

7.5.1.5 Cast-in, swellable strip-applied waterstops shall not be used in secondary containment applications. In other applications, where they cannot be maintained in a moist condition, they shall be used only where some initial leakage as the waterstop swells is acceptable.

COMMENTARY

R7.5.1.5 Swellable strip-applied waterstops perform best when used in joints continuously exposed to moisture, allowing the waterstop to remain in a swollen state. Joints may leak initially when suddenly exposed to moisture after a long period with no moisture exposure—a common occurrence with secondary containment applications. The swell rate and maximum swell capacity should also be considered if the waterstop will be subjected to fluids other than water.

CODE

7.5.1.6 For prestressed wire-wrapped structures, waterstops shall be used in vertical wall construction joints in core walls without a metal diaphragm to prevent leakage at adjacent wall sections.

7.5.2 *Sealantss*

COMMENTARY

R7.5.2 *Sealants*

The geometry of the installed joint sealant is critical. Improper geometry can cause the sealant to fail prematurely. Refer to ACI 504R for guidance in the correct design of joint sealant.

CODE

7.5.2.1 Sealants shall be selected based on the required pressures, temperatures, movements, and chemical resistance.

COMMENTARY

R7.5.2.1 Joint sealant is required to prevent the intrusion of foreign materials into the joint that may restrict anticipated joint movement, damage the waterstop, or impair the overall performance of the joint.

Sealant should be able to accommodate the maximum amount of joint movement expected in all directions relative to the joint.

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CODE

COMMENTARY

7.5.2.2 Sealants shall be of materials that have demonstrated acceptable performance under conditions and applications similar to those anticipated.

R7.5.2.2 Sealant material and properties such as non-sag versus sag, and one-component versus multiple-component, should be selected for the applicable exposure and installation conditions. Traditionally, polyurethane sealants have been selected for environmental engineering concrete structures. While polysulfide sealants have not traditionally been used for wastewater structures, some polysulfide sealants may be acceptable for such exposure.

7.5.2.3 Joint preparation for sealant application shall be specified by licensed design professional.

R7.5.2.3 The use of solvents to clean a joint can impair the bond between the sealant and the concrete surfaces when the sealant is applied and may also have an adverse effect on the integrity of the sealant. Proper preparation of the joint before sealant application should be in accordance with the sealant manufacturer's recommendations.

A primer is usually recommended prior to sealant application to enhance the bond between the sealant and the concrete surfaces to which the sealant is applied. Requirements for primer application should be determined by the sealant manufacturer.

7.5.2.4 When applied either to filler materials that may bond to the sealant, or to a concrete surface at the bottom of the joint, a bond breaker or tape shall be used between the sealant and filler or concrete surface at the bottom of the joint.

R7.5.2.4 For the surface sealant to perform as intended and accommodate anticipated joint movement, it should not bond to supporting materials, such as preformed joint filler or backer rod. The sealant should bond only to the concrete surfaces between which it provides the sealing.

7.5.3 *Preformed joint filler and backer rod*

R7.5.3 *Preformed joint filler and backer rod*

7.5.3.1 Preformed joint filler in expansion and isolation joints shall be designed and detailed to accommodate maximum joint movement, and secured in a manner to prevent restraint of the joint.

R7.5.3.1 Joint filler should be made from a highly compressible material that will accommodate a decrease in joint width without restricting joint movement or displacing the sealant or backer rod it is intended to support. Initial joint gap should be wide enough to accommodate maximum joint movement without compressing the joint filler material more than recommended by the joint filler manufacturer.

Mechanical anchors used to secure preformed joint filler material to hardened concrete may prevent or restrain intended joint movement in expansion and isolation joints.

7.5.3.2 Bituminous joint filler materials shall not be used with thermosetting, chemical curing sealants such as polyurethane.

R7.5.3.2 Bituminous joint filler may leach chemicals that impair the ability for the surface sealant to cure properly or compromise the bond between the sealant and adjacent concrete.

7.5.3.3 Backer rod shall be used to support sealant where required to provide proper sealant geometry.

7.5.3.4 Backer rod shall be of compressible closed-cell polyethylene or other suitable closed-cell material that is compatible with, but will not bond to, joint sealant.

7.5.4 *Bearing pads*

R7.5.4 *Bearing pads*

7.5.4.1 Bearing pads for sliding joints shall be attached to the hardened concrete surface to prevent uplift during

R7.5.4.1 Bearing pads are normally attached to the hardened concrete with a moisture-insensitive adhesive to

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CODE

subsequent placement of concrete. Nailing of pads shall not be permitted unless pads are specifically designed for such anchorage.

7.5.4.2 Voids between adjacent bearing pads shall be filled with joint sealant or other compressible material compatible with the bearing pad, filler, and waterstop.

COMMENTARY

prevent uplift during subsequent concreting. 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 ends of the nonprestressed reinforcement.

R7.5.4.2 Sealant should be applied between abutting ends of adjacent bearing pads to prevent the intrusion of foreign materials into potential voids between bearing pads.

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CHAPTER 8—ANALYSIS AND DESIGN – GENERAL CONSIDERATIONS

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 specified in Chapter 9.

8.1.2 Design of nonprestressed reinforced concrete members using Appendix A, Alternate Design Method, shall be permitted.

8.1.3 Anchors within the scope of Appendix E, installed in concrete to transfer loads between connected elements, shall be designed using Appendix E.

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.

COMMENTARY

CHAPTER 8—ANALYSIS AND DESIGN – GENERAL CONSIDERATIONS

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, this 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.

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 should 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.

R8.1.2 The alternate method of design, outlined in Appendix A, is similar to the working stress design method of ACI 318-63. The general serviceability requirements of the Code, such as the requirements for deflection and crack control, should be met whether the strength design method of the Code or the alternate design method of Appendix A is used.

Although prestressed members may not be designed under the provisions of the alternate design method, Chapter 19 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 (refer to 19.7).

R8.1.3 This Code has included specific provisions for anchoring to concrete for the first time in the previous edition. The provisions have 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 E in the main part of this Code to make it a legal part of this Code.

R8.2—Loading

The provisions in this Code are for live, wind, and earthquake loads such as those recommended in ASCE/SEI 7, formerly ANSI A58.1. If the service loads specified by the general building code (of which ACI 350 may form a part)

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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.

8.2.3 In design for earth pressure, hydrostatic, hydrodynamic, wind, and earthquake loads, integral structural parts shall be designed to resist the total lateral loads.

8.2.4 Consideration shall be given to effects of forces due to prestressing, crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete, 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.

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 8.7 through 8.11.

8.3.2 Except for prestressed concrete, approximate methods of frame analysis shall be permitted for engineering

COMMENTARY

differ from those of ASCE/SEI 7, the general building code governs. However, if the nature of the loads contained in a general building code differ considerably from ASCE/SEI 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 should 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.

R8.2.3 Any reinforced concrete wall that is monolithic with other structural elements is considered 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 the required resistance without contribution of the removable partition; however, the effects of all partition walls attached to the structure are to be considered in the analysis of the structure because they may lead to increased design forces in some or all elements. Provisions for earthquake-resistant design are given in Chapter 13.

R8.2.4 A concrete superstructure built over 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.

As described in R12.13.1.2, restraint of shrinkage and temperature movements can cause significant tension in slabs, as well as displacements, shear forces, and flexural moments in columns and walls. In cases of restraint, shrinkage and temperature reinforcement requirements may exceed flexural reinforcement requirements.

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 A 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, and reactions.

Moment and shear coefficients generally used for analysis of plates and shells used in environmental structures are contained in PCA (1942, 1973, 1981) and Moody (1963).

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mental engineering concrete 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) through (e) are satisfied:

(a) There are two or more spans

(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) Unfactored live load $\boldsymbol{L}$ does not exceed three times unfactored dead load $\boldsymbol{D}$

(e) Members are prismatic

For computing negative moments, ℓ_n is taken as the average of the adjacent clear span lengths.

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: $1.15w_u\ell_n/2$

Shear at face of all other supports: $w_u\ell_n/2$

8.3.4 Strut-and-tie models shall be permitted to be used in the design of structural concrete. Refer to Appendix B.

8.4—Redistribution of moments in continuous flexural members

8.4.1 Except where approximate values for moments are used, it shall be permitted to decrease factored moments computed by elastic theory at sections of maximum negative or maximum positive moment in any span of

COMMENTARY

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 should be evaluated separately.

R8.3.4 The strut-and-tie model in Appendix B is based on the assumption that portions of concrete structures can be analyzed and designed using hypothetical pin-jointed trusses consisting of struts and ties connected at nodes. This design method can be used in the design of regions where the basic assumptions of flexural theory are not applicable, such as regions near force discontinuities arising from concentrated forces or reactions, and regions near geometric discontinuities such as abrupt changes in cross section.

R8.4—Redistribution of moments in continuous flexural members

Moment redistribution is dependent on adequate ductility in plastic hinge regions. These plastic hinge regions develop at sections of maximum positive or negative moment and cause a shift in the elastic moment diagram. The usual result

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flexural members for any assumed loading arrangement by not more than $1000\varepsilon_t$ percent, with a maximum of 20 percent.

8.4.2 Redistribution of moments shall be made only when ε_t is equal to or greater than 0.0075 at the section at which moment is reduced.

8.4.3 The reduced moment shall be used for computing redistributed moments at all other sections within the spans. Static equilibrium shall be maintained after redistribution of moments for each loading arrangement.

COMMENTARY

is a reduction in the values of maximum negative moments in the support regions and an increase in the values of positive moments between supports from those computed by elastic analysis. However, because negative moments are determined for one loading arrangement and positive moments for another (refer to 14.7.6 for an exception), economies in reinforcement can sometimes be realized by reducing maximum elastic positive moments and increasing negative moments, thus narrowing the envelope of maximum negative and positive moments at any section within the span (Bondy 2003). The plastic hinges permit the use of the full capacity of more cross sections of a flexural member at ultimate loads.

Before 2020, this Code addressed moment redistribution by permitting an increase or decrease of factored negative moments above or below elastically computed values, within specified limits. A decrease in negative moment strength implies inelastic behavior in the negative moment region at the support. By increasing the negative moment strength, the positive moments can be reduced, but the result is that inelastic behavior will occur in the positive moment region of the member, and the percentage change in the positive moment section could be much larger than the 20 percent permitted for negative moment sections (Bondy 2003). The 2020 change places the same percentage limitations on both the positive and negative moment.

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 varying from 10 to 20 percent, depending on the reinforcement ratio. The results were found to be conservative (refer to Fig. R8.4). Studies by Cohn (1965) and Mattock (1959) 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 standard is available if the members satisfy the code requirements.

Moment redistribution does not apply to members designed by the alternate design method of Appendix A.

Moment redistribution as permitted by 8.4 is not intended for moments in two-way slab systems that are analyzed using the pattern loadings given in 14.7.6.3, or for use where approximate values of bending moments are used. For the Direct Design Method, 10 percent modification is allowed by 14.6.7.

ACI 350-01 Section 8.4 specified the permissible redistribution percentage in terms of reinforcement indexes. ACI 350-01 specified the permissible redistribution percentage in terms of the net tensile strain in the extreme tension steel at nominal strength, ε_t. Refer to Mast (1992) for a comparison of these moment redistribution provisions.

Consideration should be given to effects of moment distribution at service load levels. Because the 20 percent modification is no greater than the minimum factored load increase

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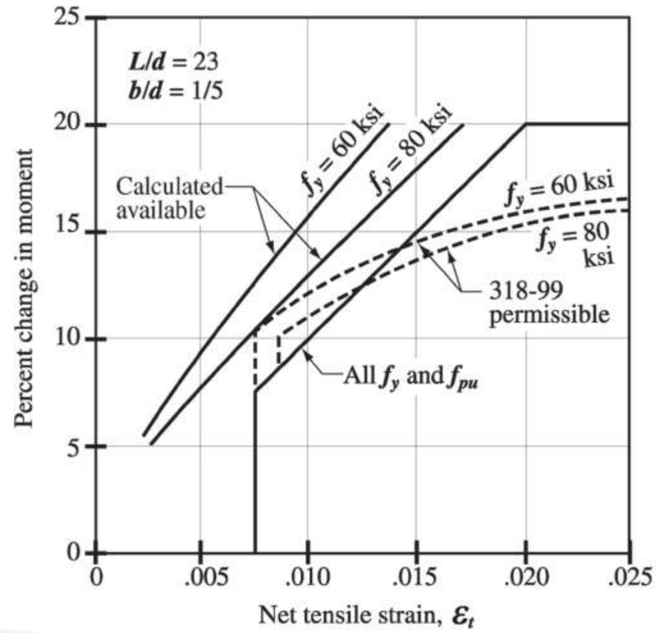

Fig. R8.4—Permissible moment redistribution for minimum rotation capacity.

based on a load factor of 1.2, the reinforcement should not yield at design service load levels. However, stresses in the reinforcement will increase, and an increased level of cracking may occur in regions where moments have been reduced for design.

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 160 lb/ft^3. For normalweight concrete, E_c shall be permitted to be taken as $\mathbf{57{,}000}\sqrt{f_c'}$.

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_p, for prestressing steel shall be determined by tests or reported by the manufacturer.

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 Pauw (1960), where E_c is defined as the slope of the line drawn from a stress of zero to a compressive stress of $\mathbf{0.45}f_c'$. The modulus of elasticity for concrete is sensitive to the modulus of elasticity 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 ASTM C469.

8.6—Lightweight concrete

8.6.1 To account for the use of lightweight concrete, unless specifically noted otherwise, a modification factor λ appears as a multiplier of $\sqrt{f_c'}$ in all applicable equations and sections of this Code, where λ = **0.85** for sand-lightweight concrete and 0.75 for all-lightweight concrete. Linear interpolation between 0.75 and 0.85 shall be permitted, on the basis of volumetric fractions, when a portion of the lightweight fine aggregate is replaced with normalweight fine aggregate. Linear interpolation between 0.85 and 1.0 shall be permitted, on the basis of volumetric fractions, for concrete containing normalweight fine aggregate and a blend of lightweight and normalweight coarse aggregates. For normalweight concrete, λ = **1.0**. If average splitting tensile strength of lightweight concrete, f_{ct}, is specified

R8.6—Lightweight concrete

R8.6.1 Factor λ reflects the lower tensile strength of lightweight concrete, which can reduce shear strength, friction properties, splitting resistance, bond between concrete and reinforcement, and increase development length, compared with normalweight concrete of the same compressive strength.

Two alternative procedures are provided to determine λ. The first alternative is based on the assumption that the tensile strength of lightweight concrete is a fixed fraction of the tensile strength of normalweight concrete (Ivey and Buth 1967). The multipliers are based on data from tests on many types of structural lightweight aggregate.

The second alternative is based on laboratory tests to determine the relationship between average splitting tensile

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$$\lambda = \frac{f_{ct}}{6.7\sqrt{f_c'}} \leq 1.0$$

8.7—Stiffness

8.7.1 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 the analysis.

8.7.2 Effect of haunches shall be considered both in determining moments and in design of members.

8.8—Effective stiffness to determine lateral deflections

8.8.1 Lateral deflections of reinforced concrete structural systems resulting from service lateral loads shall be computed by either a linear analysis with member stiffness determined using 1.4 times the flexural stiffness defined in 8.8.2 and 8.8.3, or by a more detailed analysis. Member properties shall not be taken greater than the gross section properties.

8.8.2 Lateral deflections of reinforced concrete structural systems resulting from factored lateral loads shall be computed either by linear analysis with member stiffness defined by (a) or (b), or by a more detailed analysis considering the reduced stiffness of all members under the loading conditions:

COMMENTARY

strength f_{ct} and the specified compressive strength f_c' for the lightweight concrete being used. For normalweight concrete, the average splitting tensile strength f_{ct} is approximately equal to $6.7\sqrt{f_c'}$ (Ivey and Buth 1967; Hanson 1961).

R8.7—Stiffness

R8.7.1 Ideally, the member stiffnesses E_cI and GJ should reflect the degree of cracking and inelastic action which 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 E_cI values for all members or to use half the gross E_cI of the beam stem for beams and the gross E_cI for the columns.

For frames that are free to sway, a realistic estimate of E_cI is desirable and should be used if second-order analyses are carried out. Guidance for the choice of E_cI for this case is given in R10.10.4.

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.

R8.7.2 Stiffness and fixed-end moment coefficients for haunched members may be obtained from PCA (1972).

R8.8—Effective stiffness to determine lateral deflections

R8.8.1 The selection of appropriate effective stiffness values depends on the intended performance of the structure. For wind loading, it is desirable to maintain elastic behavior in members at service load conditions. When analyzing a structure subjected to earthquake events at short recurrence intervals, some yielding without significant damage to the members may be a tolerable performance objective. As with lateral stability analysis of concrete structures (R10.10.4), a factor of 1.4 times the stiffness used for analysis under factored lateral loads is adequate to model effective section properties for lateral deflection analysis under service loads. Alternatively, a more accurate level of stiffness based on the expected element performance can be determined.

R8.8.2 The lateral deflection a structure sustains under factored lateral loads can be substantially different from that computed using linear analysis, in part because of the inelastic response of the members and the decrease in effective stiffness. The selection of appropriate effective stiffness for reinforced concrete frame members has dual purposes: to

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(a) By section properties defined in 10.10.4.1 (a) through (c), or

(b) 50 percent of stiffness values based on gross section properties

8.8.3 Where two-way slabs without beams are designated as part of the lateral-force-resisting system, lateral deflections resulting from factored lateral loads shall be permitted to be computed by using linear analysis. The stiffness of slab members shall be defined by a model that is in substantial agreement with results of comprehensive tests and analysis and the stiffness of other frame members shall be as defined in 8.8.2.

8.9—Span length

8.9.1 Span length of members not built integrally with supports shall be considered as the clear span plus the depth of the member but need not exceed distance between centers of supports.

8.9.2 In analysis of frames or continuous construction for determination of moments, span length shall be taken as the distance center-to-center of supports.

COMMENTARY

provide realistic estimates of lateral deflection and to determine deflection-imposed actions on the gravity system of the structure. A detailed nonlinear analysis of the structure would adequately capture these two effects. A simple way to estimate an equivalent nonlinear lateral deflection ($\boldsymbol{\delta}_{em}$ at the top story in IBC [2006]) using linear analysis is to reduce the modeled stiffness of the concrete members in the structure. The type of lateral load analysis affects the selection of appropriate effective stiffness values. For analyses with wind loading, where it is desirable to prevent nonlinear action in the structure, effective stiffness representative of pre-yield behavior may be appropriate. For earthquake loading, a level of nonlinear behavior is tolerable depending on the intended structural performance and earthquake recurrence interval.

Varying degrees of confidence can be obtained from a simple linear analysis based on the calculation rigor used to define the effective stiffness of each member. One option that considers the reduced stiffness of the elements is to compute the secant stiffness value to the point of yielding of reinforcement for the member, or the secant value to a point before yielding of the reinforcement if analysis demonstrates yielding is not expected for the given loading condition. The alternative options presented in 8.8.2 use values that approximate stiffness for reinforced concrete structural systems loaded to near or beyond the yield level and have been shown to produce reasonable correlation with both experimental and detailed analytical results (Moehle 1992; Lepage 1998). The effective stiffnesses in Option (a) were developed to represent lower-bound values for stability analysis of concrete structural systems subjected to gravity and wind loads. Option (a) is provided so that the model used to compute slenderness effects may be used to compute lateral deflections due to factored wind and earthquake loading. In general, for effective section properties, $\boldsymbol{E_c}$ may be defined as in 8.5.1, $\boldsymbol{A}$ as in 10.10.4.1(c), and the shear modulus may be taken as $\mathbf{0.4}\boldsymbol{E_c}$.

R8.8.3 Analysis of structures with two-way slab systems without beams requires that the model represent the transfer of lateral loads between vertical members. The model should result in prediction of stiffness in substantial agreement with results of comprehensive tests and analysis. Several acceptable models have been proposed to accomplish this action (Vanderbilt and Corley 1983; Hwang and Moehle 2000; Dovich and Wight 2005).

R8.9—Span length

Beam moments computed at support centers may be reduced to the moments at support faces for design of beams. PCA (1959) provides an acceptable method of reducing moments at support centers to those at support faces.

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8.9.3 For beams built integrally with supports, design on the basis of moments at faces of support shall be permitted.

8.9.4 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 the width of beams otherwise neglected.

8.10—Columns

8.10.1 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.

8.10.2 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.

8.10.3 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.

8.10.4 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.

8.11—Arrangement of live load

8.11.1 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 fixed

8.11.2 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

R8.10—Columns

Section 8.10 has been developed with the intent of making certain that the most demanding combinations of axial load and moments be identified for design.

Section 8.10.4 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 8.10.4 refers to the difference between the moments in a given vertical plane, exerted at column centerline by members framing into that column.

R8.11—Arrangement of live load

For determining column, wall, and beam moments and shears caused by gravity loads, this 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 fixed for the purpose of analysis under gravity loads. This assumption does not apply to lateral load analysis; however, in analysis for lateral loads, 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 most demanding sets of design forces should be established by investigating the effects of live load placed in various critical patterns.

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 10.10.

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8.12—T-beam construction

8.12.1 In T-beam construction, the flange and web shall be built integrally or otherwise effectively bonded together.

8.12.2 Width of slab effective as a T-beam flange shall not exceed one-fourth 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

8.12.3 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

8.12.4 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.

8.12.5 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:

8.12.5.1 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.

8.12.5.2 Transverse reinforcement shall be spaced not farther apart than 12 in.

R8.12—T-beam construction

This section contains provisions identical to those of previous Codes for limiting dimensions related to stiffness and flexural calculations. Provisions related to T-beams and other flanged members are stated in 11.5.1 with regard to torsion.

8.13—Joist construction

8.13.1 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.

8.13.2 Ribs shall be not less than 4 in. in width and shall have a depth of not more than 3.5 times the minimum width of rib.

8.13.3 Clear spacing between ribs shall not exceed 30 in.

8.13.4 Joist construction not meeting the limitations of 8.13.1 through 8.13.3 shall be designed as slabs and beams.

R8.13—Joist construction

The size and spacing limitations for concrete joist construction meeting the limitations of 8.13.1 through 8.13.3 are based on successful performance in the past.

R8.13.3 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.

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8.13.5 When permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to f_c' in the joists are used:

8.13.5.1 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.

8.13.5.2 Slab thickness over permanent fillers shall not be less than one-twelfth the clear distance between ribs, nor less than 1-1/2 in.

8.13.5.3 In one-way joists, reinforcement normal to the ribs shall be provided in the slab as required by 12.13.

8.13.6 When removable forms or fillers not complying with 8.13.5 are used:

8.13.6.1 Slab thickness shall not be less than one-twelfth the clear distance between ribs, nor less than 2 in.

8.13.6.2 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 12.13.

8.13.7 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 significantly impair the strength of the construction.

8.13.8 For joist construction, 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.

8.14—Separate floor finish

8.14.1 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 18.

8.14.2 It shall be permitted to consider all concrete floor finishes as part of required cover or total thickness for nonstructural considerations.

COMMENTARY

R8.13.8 The increase in shear strength permitted by 8.13.8 is justified on the basis of: 1) satisfactory performance of joist construction with higher shear strengths, designed under previous Codes, which allowed comparable shear stresses; and 2) redistribution of local overloads to adjacent joists.

R8.14—Separate floor finish

This 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 licensed design professional.

As in previous editions of this 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 18.

All floor finishes may be considered for nonstructural purposes such as cover for reinforcement and fire protection. Provisions should be made, however, to ensure that the finish will not spall off, thus causing decreased cover. Furthermore, development of reinforcement considerations requires minimum monolithic concrete cover according to 12.7.

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CHAPTER 9—STRENGTH AND SERVICEABILITY REQUIREMENTS

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 D shall be permitted. Use of load factor combinations from this chapter in conjunction with strength reduction factors of Appendix D shall not be permitted.

9.2—Required strength

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.

$$U = 1.4(D + F) \quad (9\text{-}1)$$

$$U = 1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \quad (9\text{-}2)$$

$$U = 1.2(D + F) + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.5W) \quad (9\text{-}3)$$

$$U = 1.2(D + F) + 1.0(W + L) + 0.5(L_r \text{ or } S \text{ or } R) \quad (9\text{-}4)$$

$$U = 1.2(D + F) + 1.0(E + L) + 1.6\text{H} + 0.2S \quad (9\text{-}5)$$

COMMENTARY

CHAPTER R9—STRENGTH AND SERVICEABILITY REQUIREMENTS

R9.1—General

In ACI 350-06, the load factor combinations and strength reduction factors of ACI 350-01 were revised and moved to Appendix C. The 2001 combinations were replaced based on those of ASCE 7-02. The strength reduction factors were replaced with those of ACI 318-99 Appendix C, except that the factor for flexure was increased. In this edition of the Code, the factored load combinations were revised based on ASCE/SEI 7-10.

The changes were made to further unify the design profession on one set of load factors and combinations, and to facilitate the proportioning of environmental engineering concrete 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:

$$\text{design strength} \geq \text{required strength}$$

$$\phi\,(\text{nominal strength}) \geq U$$

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 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, it should not be assumed that all cases are covered.

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$$U = 0.9D + 1.2F + 1.0W + 1.6H \quad (9\text{-}6)$$

$$U = 0.9D + 1.2F + 1.0E + 1.6H \quad (9\text{-}7)$$

$$U = 1.4(D + F) + 0.6H \quad (9\text{-}8)$$

except as follows:

(a) The load factor on $\boldsymbol{L}$ in Eq. (9-3) to (9-5) shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and areas where the live load $\boldsymbol{L}$ is greater than 100 lb/ft^2.

(b) Where $\boldsymbol{W}$ is based on service-level wind loads, $\mathbf{1.6}\boldsymbol{W}$ shall be used in place of $\mathbf{1.0}\boldsymbol{W}$ in Eq. (9-4) and (9-6), and $\mathbf{0.8}\boldsymbol{W}$ shall be used in place of $\mathbf{0.5}\boldsymbol{W}$ in Eq. (9-3).

(c) Where earthquake effects $\boldsymbol{E}$ is based on service-level seismic forces, $\mathbf{1.4}\boldsymbol{E}$ shall be used in place of $\mathbf{1.0}\boldsymbol{E}$ in Eq. (9-5) and (9-7).

(d) The load factor on $\boldsymbol{H}$ shall be reduced to 0.6 where $\boldsymbol{H}$ reduces the effect of $\boldsymbol{D}$, $\boldsymbol{L}$, or $\boldsymbol{F}$. $\boldsymbol{H}$ shall be permitted to be used to reduce other load effects only if investigations and

COMMENTARY

Due regard is to be given to sign in determining $\boldsymbol{U}$ 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 $\mathbf{0.9}\boldsymbol{D}$ 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 unusual circumstances require greater reliance on the strength of particular members than encountered in usual practice, some reduction in the stipulated strength reduction factor $\boldsymbol{\phi}$ or increase in the stipulated load factors $\boldsymbol{U}$ may be appropriate for such members.

In this edition, the Code removed the weight of soil and other fill materials as part of the definition of $\boldsymbol{H}$. Consistent with ASCE/SEI 7, the weight of these materials is part of dead load $\boldsymbol{D}$. The load factors for $\boldsymbol{D}$ are appropriate provided the unit weight and thickness of earth or other fill materials are well controlled. If the weight of earth stabilizes the structure, a load factor of zero may be appropriate.

R9.2.1(a) The load factor modification of 9.2.1(a) is different than the live load reductions based on the loaded area that may be allowed in the legally adopted general building code. The live load reduction adjusts the nominal load $\boldsymbol{L}$. The smaller load factor reflects the reduced probability of the joint occurrence of maximum values of multiple transient loads at the same time. The reduced live loads as specified in the legally adopted general building code can be used in combination with the 0.5 load factor specified in 9.2.1(a).

R9.2.1(b) ASCE/SEI 7 has converted wind loads to strength level and reduced the wind load factor to 1.0. This Code requires use of the previous load factor for wind loads, 1.6, when service-level wind loads are used. For serviceability checks, the commentary to Appendix D of ASCE/SEI 7-10 provides service-level wind loads $\boldsymbol{W_a}$.

R9.2.1(c) In 1993, ASCE 7 converted earthquake forces to strength level and reduced the earthquake load factor to 1.0. Model building codes (BOCA National Building Code, SBCCI's Standard Building Code, ICBO's Uniform Building Code) followed. This Code requires use of the previous load factor for earthquake effects, approximately 1.4, when service-level earthquake effects are used.

R9.2.1(d) 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

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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.

(f) Where approved by the Authority Having Jurisdiction, the Responsible Design Professional is permitted to use ***F*** and ***H*** simultaneously in accordance with Eq. (9-8) in place of Eq. (9-1). For Eq. (9-8), ***H*** shall not include groundwater. All limitations on tightness testing, future adjacent construction, operation, and maintenance required to ensure that ***H*** will continue to act simultaneously with ***F*** shall be included on the contract drawings per 1.2.

9.2.2 *Impact effects*

If resistance to impact effects is taken into account in design, such effects shall be included with live load ***L***.

9.2.3 *Self-restraining effects*

Where applicable, the structural effects of ***T*** shall be considered in combination with other loads. The load factor on ***T*** shall be established considering the uncertainty associated with the likely magnitude of ***T***, the probability that the maximum effect of ***T*** will occur simultaneously with other applied loads, and the potential adverse consequences if the effect of ***T*** is greater than assumed. The load factor on ***T*** shall not have a value less than 1.0.

COMMENTARY

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 be applied to ***H*** as noted, and the magnitude of earth pressure used should be developed conservatively by a geotechnical engineer. This reduced load factor for environmental structures may be different than the load factor used in other codes, such as ACI 318 and ASCE 7.

R9.2.1(e) Both ***L*** and ***F*** are considered transient loads, so designs should consider the effects for such loads being present or absent.

R9.2.1(f) The load case shown in Eq. (9-8) is not intended to be used for environmental engineering concrete structures where tightness testing is possible or desired prior to backfilling. However, there are conditions where the walls will not be exposed to internal fluid pressures ***F*** without the presence of some external lateral soil pressure ***H***. For example, when the sinking caisson method of construction is used, the soil on the outside of the caisson remains in place during construction, and performance of a tightness test without soil in place is impractical. Another example is a tank wall that requires backfill to be placed against it prior to construction of an integral shallower structure, both of which are to be constructed prior to tightness testing the tank. A third example is when the liquid level exceeds normal high-level operating conditions due to equipment malfunction or operator error. In such cases, ***H*** may be considered to act simultaneously with ***F*** in accordance with Eq. (9-8). Because groundwater is uncertain by nature, it is not included with ***H*** in Eq. (9-8). Consideration should be given in the design to some minimum depth of potential excavation adjacent to the structure to facilitate future testing, operation, maintenance, future adjacent construction, retrofit, or rehabilitation. Where there is a potential for soils such as clays or silts to dry out and form a vertical crack adjacent to the structure which may reduce the soil pressure, ***H*** should be adjusted as determined by a geotechnical engineer.

R9.2.2 *Impact effects*

If the live load is applied rapidly, as may be the case for vehicle loads and cranes, impact effects should be considered. In all equations, substitute (***L*** + impact) for ***L*** when impact should be considered.

R9.2.3 *Self-restraining effects*

Several strategies can be used to accommodate movements due to differential settlement and volume change. Forces due to ***T*** effects are not commonly calculated and combined with other load effects. Rather, designs rely on successful past practices, using compliant structural members and ductile connections to accommodate differential settlement and volume change movement while providing the needed resistance to gravity and lateral loads. Expansion joints and construction closure strips are used to limit volume change

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9.2.4 *Flood and ice loads*

If a structure is in a flood zone, or is subjected to forces from atmospheric ice loads, the flood or ice loads and the appropriate load combinations of ASCE/SEI 7 shall be used.

9.2.5 *Prestressing steel jacking force*

For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum prestressing steel jacking force.

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 shall be applied to the required shear to be carried by shear reinforcement as shown in Chapter 11 and is not applied to the total required shear strength U. This durability factor shall not be used for designs using service loads and permissible service load stresses.

$$S_d = \frac{\phi f_y}{\gamma f_{s,max}} \geq 1.0 \tag{9-8a}$$

where

$$\gamma = \frac{\text{factored load}}{\text{unfactored load}}$$

and where $f_{s,max}$ is the maximum allowable tensile stress in reinforcement as given below:

COMMENTARY

movements based on performance of similar structures. Shrinkage and temperature reinforcement is commonly proportioned based on gross concrete area rather than calculated force.

However, where structural movements can lead to damage of nonductile elements, calculation of the predicted force should consider the inherent variability of the expected movement and structure response. A long-term study of the volume change behavior of precast concrete buildings (PCA 1993), completed in 2009, recommends procedures to account for connection stiffness, thermal exposure, member softening due to creep, and other factors that influence T forces.

R9.2.4 *Flood and ice loads*

Areas subject to flooding are defined by flood hazard maps, usually maintained by local governmental jurisdictions.

R9.2.5 *Prestressing steel jacking force*

The load factor of 1.2 applied to the maximum tendon jacking force results in a design load of approximately 113 percent of the specified prestressing steel yield strength, but 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.

R9.2.6 In environmental engineering concrete structures, durability and long-term service life are paramount. The resulting stresses in nonprestressed reinforcement using general building code load factors could be 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 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 that S_d be no less than 1.0 is to ensure that the strength requirements of ACI 318 are always met as a minimum regardless of crack control considerations. This limitation will likely control where reinforcement of relatively low yield strength, or closely spaced bars, 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 $f_y/f_{s,max}$ with ϕ factors set equal 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 $f_y/f_{s,max}$ (≥1.0).

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COMMENTARY

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	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$

Notes: Values given shall be used directly for members with normalweight concrete and Grade 60 reinforcement. For other conditions, the values shall be modified as follows:

a) For lightweight concrete having equilibrium density, w_c, in the range 90 to 115 lb/ft^3, the values shall be multiplied by $(1.65 - 0.005w_c)$ but not less than 1.09.

b) For f_y other than 60,000 psi, the values shall be multiplied by $(0.4 + f_y/100{,}000)$.

2. Use a value of 1.0 for applicable design ϕ factors.

The general building code 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 tension: Refer to 10.6.4.

R9.2.6.1 Required flexural strength $\geq S_dU$.

9.2.6.2 Direct and hoop tension in normal environmental exposures: $f_{s,max}$ **= 20,000 psi**

9.2.6.3 Direct and hoop tension in severe environmental exposures: $f_{s,max}$ **= 17,000 psi**

R9.2.6.2 and R9.2.6.3 Required strength in direct and hoop tension $\geq S_dU$.

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" (PCA 1942).

9.2.6.4 Shear carried by shear reinforcement in normal environmental exposures: $f_{s,max}$ **= 24,000 psi**

9.2.6.5 Shear carried by shear reinforcement in severe environmental exposures: $f_{s,max}$ **= 20,000 psi**

R9.2.6.4 and R9.2.6.5 Shear stress carried by the shear reinforcement is the excess shear strength required in addition to the design shear strength provided by the concrete ϕV_c

$$\phi V_s \geq S_d(V_u - \phi V_c)$$

9.2.7 S_d need not be greater than 1.0, regardless of exposure, for any of the following:

9.2.7.1 The design of compression-controlled sections

9.2.7.2 Load combinations Eq. (9-4), (9-5), (9-6), and (9-7)

9.2.7.3 Flooding with a 100-year or longer return period

9.2.7.4 Liquid level that exceeds normal operation conditions due to equipment malfunction or operator error

9.2.7.5 All prestressed reinforcement and post-tensioned anchorage zone reinforcement

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 0.002 per 10.3.3, and therefore have less concern for cracking.

The Code does not require the environmental durability factor to be applied for unusual or unlikely loading conditions after which crack repairs can be made if needed. The minimum code provisions for strength and stability should be applied for such loading conditions. However, the facility owner may choose to include the environmental durability factor for some such conditions to minimize repairs after such an event.

9.2.8 *Tensile hoop stress in concrete*

R9.2.8 Equation (9-8b) accounts for the combined effects of concrete shrinkage and concrete tensile stresses due to T_{hoop}. This practice is based on PCA (1993).

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The service level tensile hoop stress in the transformed section of concrete in walls of nonprestressed circular tanks shall be limited to

$$f_{c,tension} = \frac{C_s E_s A_s + T_{hoop}}{A_g + \frac{E_s}{E_c} A_s} \leq 0.1 f_c' \quad (9\text{-}8b)$$

where C_s, the average coefficient of shrinkage for the reinforced concrete, is equal to 0.0003.

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, 9.3.4 and 9.3.5.

9.3.2 Strength reduction factor ϕ shall be as given in 9.3.2.1 through 9.3.2.7:

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: 0.75

(b) Other reinforced members: 0.65

For sections in which the net tensile strain in the extreme tension steel at nominal strength, ε_t, is between the limits for compression-controlled and tension-controlled sections, ϕ shall be permitted to be linearly increased from that for

COMMENTARY

Equation (9-8b) is the theoretical service level hoop tensile stress, which is based on an assumed coefficient of shrinkage for reinforced concrete of 0.0003 and a transformed section that assumes uncracked concrete. The service level hoop stress, $f_{c,tension}$, is limited to a maximum value that is equal to the historically assumed tensile strength of the concrete, $0.10f_c'$.

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 1.

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 (MacGregor 1976; Winter 1979).

In ACI 318-02, the strength reduction factors were adjusted to be compatible with the ASCE 7-02 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 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 (MacGregor 1976) and current reliability analyses (Nowak et al. 2005), statistical study of material properties, as well as the opinion of the committee that the historical performance of concrete structures supports ϕ = **0.90**. In ACI 318-08, ϕ for spirally reinforced compression-controlled sections was revised based on the reliability analyses reported in Nowak et al. (2005) and the superior performance of such members when subjected to excessive demand as documented in Mlakar (2005).

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 ϕ for cases of axial load or flexure, or both, in terms of the type of loading. For these cases, ϕ is now determined by the strain conditions at a cross section, at nominal strength.

A lower ϕ 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

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CODE | COMMENTARY

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 9.5.4.3 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.

Table 9.5(c)—Minimum thickness of slabs without interior beams

	Without drop panels‡			With drop panels‡		
	Exterior panels			Exterior panels		
f_y, psi†	Without edge beams	With edge beams§	Interior panels	Without edge beams	With edge beams§	Interior panels
40,000	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
60,000	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
75,000	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$	$\ell_n/31$	$\ell_n/34$	$\ell_n/33$

*For two-way construction, ℓ_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.

†For f_y between the values given in the table, minimum thickness shall be determined by linear interpolation.

‡Drop panels as defined in 13.2.5.

§Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall not be less than 0.8.

compression-controlled sections to 0.90 as ε_t 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 $(d - d')/h$ not 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.

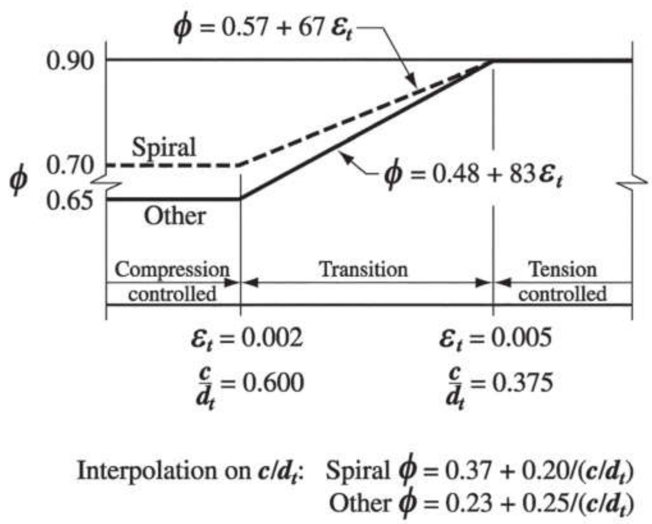

Fig. R9.3.2.2—Variation of ϕ with net tensile ε_t and c/d_t for Grade 60 reinforcement and for prestressing steel.

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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. R9.3.2.2. 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 reinforcement. 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 with Grade 60 reinforcement and to prestressed sections. Figure R9.3.2.2 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 ACI 318-02. 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 Section 9.3, refer to Mast (1992).

CODE

9.3.2.3 Shear and torsion: 0.75

9.3.2.4 Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models): 0.65

9.3.2.5 Post-tensioned anchorage zones: 0.85

R9.3.2.5 The ϕ of 0.85 reflects the wide scatter of results of experimental anchorage zone studies. Because 19.13.4.2 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 $0.85 \times 0.7\lambda f_{ci}' \approx 0.6\lambda f_{ci}'$.

9.3.2.6 Strut-and-tie models (Appendix B), and struts, ties, nodal zones, and bearing areas in such models: 0.75

R9.3.2.6 The ϕ used in strut-and-tie models is taken equal to the ϕ for shear. The value of ϕ for strut-and-tie models is applied to struts, ties, and bearing areas in such models.

9.3.2.7 Flexural sections in pretensioned members where strand embedment is less than the development length as provided in 12.9.1.1:

(a) From the end of the member to the end of the transfer length: 0.75

R9.3.2.7 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. For sections between the end of the transfer length and the end of the development length, the value of ϕ may be determined by linear interpolation, as shown in Fig. R9.3.2.7a and R9.3.2.7b.

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(b) From the end of the transfer length to the end of the development length ϕ shall be permitted to be linearly increased from: 0.75 to 0.9

Where bonding of a strand does not extend to the end of the member, strand embedment shall be assumed to begin at the end of the debonded length. Refer also to 12.9.3.

9.3.3 Development lengths specified in Chapter 12 do not require a ϕ.

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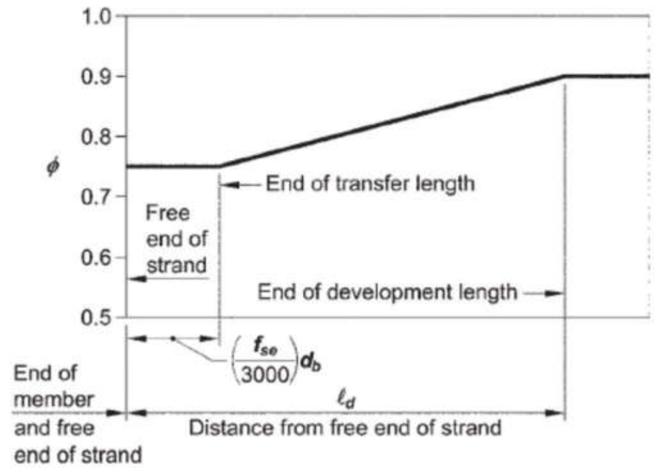

Fig. R9.3.2.7a—Variation of ϕ with distance from the free end of strand in pretensioned members with fully bonded strands.

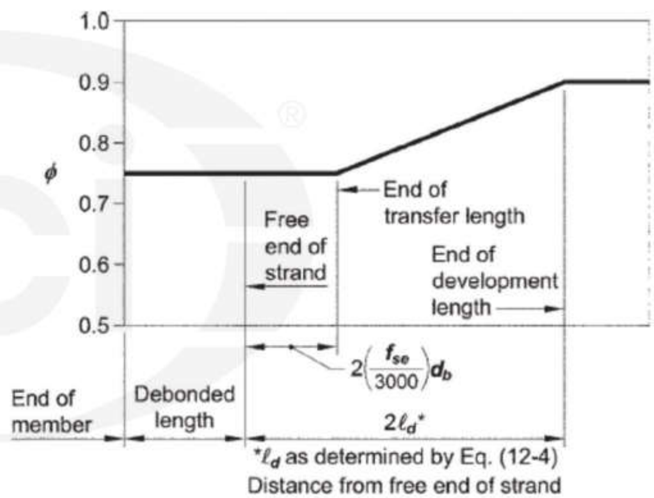

Fig. R9.3.2.7b—Variation of ϕ with distance from the free end of strand in pretensioned members with debonded strands where 12.9.3 applies.

Where bonding of one or more strands does not extend to the end of the member, instead of a more rigorous analysis, ϕ may be conservatively taken as 0.75 from the end of the member to the end of the transfer length of the strand with the longest debonded length. Beyond this point, ϕ may be varied linearly to 0.90 at the location where all strands are developed, as shown in Fig. R9.3.2.7b. Alternatively, the contribution of the debonded strands may be ignored until they are fully developed. Embedment of debonded strand is considered to begin at the termination of the debonding sleeves. Beyond this point, the provisions of 12.9.3 are applicable.

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9.3.4 For structures that rely on intermediate precast structural walls in Seismic Design Category D, E, or F, special moment frames, or special structural walls to resist earthquake effects $\boldsymbol{E}$, ϕ shall be modified as given in (a) through (c):

(a) For any structural member that is designed to resist $\boldsymbol{E}$, ϕ for shear shall be 0.60 if the nominal shear strength of the member 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 $\boldsymbol{E}$.

(b) For diaphragms, ϕ for shear shall not exceed the minimum ϕ for shear used for the vertical components of the primary seismic-force-resisting system.

(c) For joints and diagonally reinforced coupling beams, ϕ for shear shall be 0.85.

COMMENTARY

R9.3.4 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.

9.4—Design strength for reinforcement

The values of f_y and f_{yt} used in design calculations shall not exceed 80,000 psi, except for prestressing steel and for transverse reinforcement in 10.9.3 and 13.1.5.

R9.4—Design strength for reinforcement

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, 11.7.6, and 18.9.3.2, the maximum f_y or f_{yt} that may be used in design for shear and torsion reinforcement is 60,000 psi, except that f_{yt} up to 80,000 psi may be used for shear reinforcement meeting the requirements of ASTM A1064.

In 19.3.2 and 21.2.5, the maximum specified f_y is 60,000 psi in shells, folded plates, special moment frames, and special structural walls.

The deflection provisions of 9.5 and the limitations on distribution of flexural reinforcement of 10.6 become increasingly critical as f_y increases.

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.

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 dead load and that portion of the live load that is sustained need be considered.

Two methods are given for controlling deflections (*Deflections of Concrete Structures* 1974). 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 should be calculated by the procedures described or referred to in the appropriate

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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.

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.

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

$$f_r = 7.5\lambda\sqrt{f_c'} \tag{9-11}$$

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 (refer to 9.5.2), and for composite members (refer to 9.5.5).

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 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 ACI 213R-87. No correction is specified for concretes with w_c greater than 115 lb/ft^3 because the correction term would be close to unity in this range.

The modification for f_y 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.

R9.5.2.3 The effective moment of inertia procedure described in the Code and developed in Branson (1965) was selected as being sufficiently accurate for use to control deflections (ACI Committee 435 1966, 1968; ACI 209R). 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.

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9.5.2.4 For continuous members, I_e 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 λ_Δ

$$\lambda_\Delta = \frac{\xi}{1+50\rho'} \tag{9-12}$$

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
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).

COMMENTARY

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 (1973, 1978) and *Deflections of Concrete Structures* (1974).

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 by ACI Committee 435 (1978). In Eq. (9-12), the multiplier on ξ accounts for the effect of compression reinforcement in reducing long-term deflections, and ξ = **2.0** 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 *Deflections of Concrete Structures* (1974) and Branson (1965, 1971, 1977) may be used.

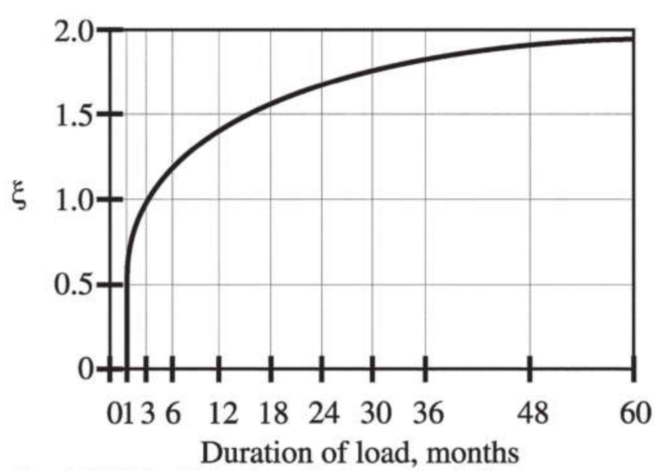

Fig. R9.5.2.5—Multipliers for long-term deflections.

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

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COMMENTARY

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 (refer to ACI Committee 435 [1968]).

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.

9.5.3 *Two-way construction (nonprestressed)*

R9.5.3 *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 14 and conforming with the requirements of 14.6.1.2. 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 14.2.5.2 and 14.2.5.1: 5 in.

(b) Slabs with drop panels as defined in 14.2.5.2 and 14.2.5.1: 4 in.

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.

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_{fm} 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

$$h = \frac{\ell_n\left(0.8 + \dfrac{f_y}{200{,}000}\right)}{36 + 5\beta(\alpha_{fm} - 0.2)} \qquad \text{(9-13)}$$

and not less than 5 in.

(c) For α_{fm} greater than 2.0, the thickness shall not be less than

$$h = \frac{\ell_n\left(0.8 + \dfrac{f_y}{200{,}000}\right)}{36 + 9\beta} \qquad \text{(9-14)}$$

and not less than 3.5 in.

(d) At discontinuous edges, an edge beam shall be provided with a stiffness ratio α_f not less than 0.80 or the minimum

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_{fm} 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_{fm} equal to 0.2.

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thickness required by Eq. (9-13) or (9-14) shall be increased by at least 10 percent in the panel with a discontinuous edge.

Term ℓ_n in (b) and (c) is length of clear span in long direction measured face-to-face of beams. Term β in (b) and (c) is ratio of clear spans in long to short direction of slab.

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 where computed deflections do 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, I_e, 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, I_g, 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, I_e, as defined by Eq. (9-9).

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 (ACI Committee 435 1968). Other procedures and other values of the stiffness E_cI 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. PCI (2004) gives information on deflection calculations using a bilinear moment-deflection relationship and using an effective moment of inertia. Mast (1998) gives additional information on deflection of cracked prestressed concrete members.

Shaikh and Branson (1970) 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 Shaikh and Branson (1970), with approximate forms given in ACI Committee 435 (1968) and Branson (1970).

R9.5.4.3 Calculation of long-term deflections of prestressed concrete flexural members is complicated. The calculations should consider not only the increased deflections due to flexural stresses, but also the additional long-term deflections resulting from time-dependent 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

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significant shortening of the flexural members that continues for several years after construction and should be considered in design. The shortening tends to reduce the tension in the prestressing reinforcement, 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 ACI Committee 435 (1963, 1968), Branson et al. (1970), and Ghali and Favre (1986).

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), 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).

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 shall 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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CHAPTER 10—FLEXURE AND AXIAL LOADS

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 that, for deep beams as defined in 10.7.1, an analysis that considers a nonlinear distribution of strain shall be used. Alternatively, it shall be permitted to use a strut-and-tie model. Refer to 10.7, 11.7, and Appendix B.

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 f_y 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.

COMMENTARY

CHAPTER 10—FLEXURE AND AXIAL LOADS

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.

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 (refer to 8.5.2).

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10.2.5 Tensile strength of concrete shall be neglected in axial and flexural calculations of reinforced concrete, except when meeting requirements of 19.4.

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.

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 $\mathbf{0.85}f_c'$ 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 from the fiber of maximum strain to the neutral axis, c, shall be measured in a direction perpendicular to the neutral axis.

10.2.7.3 For f_c' of 4000 psi, β_1 shall be taken as 0.85. For f_c' above 4000 psi, β_1 shall be reduced linearly 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.

COMMENTARY

R10.2.5 The tensile strength of concrete in flexure (modulus of rupture) is a more variable property than the compressive strength and is approximately 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 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 design, the Code allows the use of an equivalent rectangular compressive stress distribution (stress block) to replace the more exact concrete stress distributions. In the equivalent rectangular stress block, an average stress of $\mathbf{0.85}f_c'$ 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 (Leslie et al. 1976; Karr et al. 1978) 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

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zone at ultimate but does provide essentially the same results as those obtained in tests (Mattock et al. 1961).

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.

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 Mattock et al. (1961). Mattock et al. (1961) and previous editions of this Commentary also give the derivations of strength equations for cross sections other than rectangular.

10.3.2 Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to f_y just as concrete in compression reaches its assumed ultimate strain of 0.003.

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.

10.3.3 Sections are compression-controlled if the net tensile strain in the extreme tension steel, ε_t, is equal to or less than the compression-controlled strain limit when 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.

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.

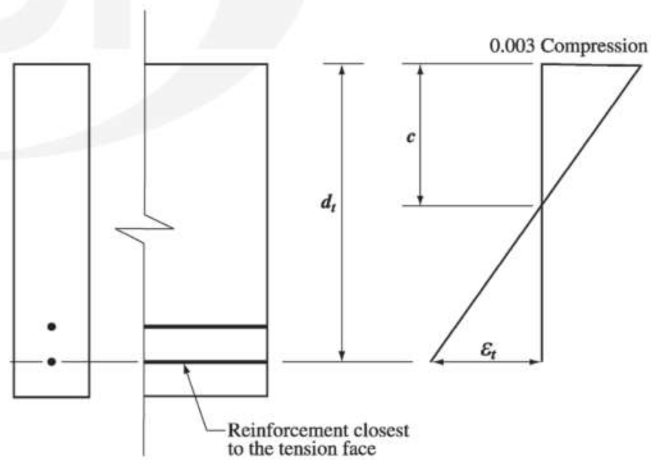

Fig. R10.3.3—Strain distribution and net tensile strain.

10.3.4 Sections are tension-controlled if the net tensile strain in the extreme tension steel, ε_t, is equal to or greater than 0.005 when the concrete in compression reaches its assumed strain limit of 0.003. Sections with ε_t between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.

R10.3.4 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 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

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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 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.

10.3.5 For nonprestressed flexural members and nonprestressed members with factored axial compressive load less than $0.10f_c'A_g$, ε_t at nominal strength shall not be less than 0.004.

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.

10.3.5.1 Use of compression reinforcement shall be permitted in conjunction with additional tension reinforcement to increase the strength of flexural members.

10.3.6 Design axial strength ϕP_n of compression members shall not be taken greater than $\phi P_{n,max}$, computed by Eq. (10-1) or (10-2).

10.3.6.1 For nonprestressed members with spiral reinforcement conforming to 12.10.1.4 or composite members conforming to 10.13

$$\phi P_{n,max} = 0.85\phi[0.85f_c'(A_g - A_{st}) + f_yA_{st}] \quad (10\text{-}1)$$

10.3.6.2 For nonprestressed members with tie reinforcement conforming to 12.10.1.5

$$\phi P_{n,max} = 0.80\phi[0.85f_c'(A_g - A_{st}) + f_yA_{st}] \quad (10\text{-}2)$$

10.3.6.3 For prestressed members, design axial strength ϕP_n shall not be taken greater than 0.85 (for members with

R10.3.6 and R10.3.7 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 (refer to 10.10.6.5). The specified minimum eccentricities were originally intended to serve as a means of reducing the axial 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 strength of a compression member. This is now accomplished directly in 10.3.6 by limiting the design axial 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 of 0.05 and 0.10, specified in the earlier Codes for the spirally rein-

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spiral reinforcement) or 0.80 (for members with tie reinforcement) of the design axial strength at zero eccentricity, ϕP_o.

10.3.7 Members subject to compressive axial load shall be designed for the maximum moment that can accompany the axial load. The factored axial force P_u at given eccentricity shall not exceed that given in 10.3.6. The maximum factored moment M_u shall be magnified for slenderness effects in accordance with 10.10.

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.

COMMENTARY

forced 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 ACI 318-63 and ACI 318-71 are equally applicable.

For prestressed members, the design axial 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 10.10.7.1.

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* (ACI SP-17(09)) and the *CRSI Handbook* (CRSI 1984). The reciprocal load method (Bresler 1960) and the load contour method (Parme et al. 1966) are the methods used in those two handbooks. Research (Heimdahl and Bianchini 1975; Furlong 1979) 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 (Bresler 1960)

$$\frac{1}{P_{ni}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_o}$$

where P_{ni} is nominal axial load strength at given eccentricity along both axes; P_o is nominal axial load strength at zero eccentricity; P_{nx} is nominal axial load strength at given eccentricity along x-axis; and P_{ny} is 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.

R10.4—Distance between lateral supports of flexural members

Tests (Hansell and Winter 1959; Sant and Bletzacker 1961) 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.

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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 and 10.5.3, A_s provided shall not be less than that given by

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \tag{10-3}$$

and not less than $200b_wd/f_y$.

10.5.2 For statically determinate members with a flange in tension, $A_{s,min}$ shall not be less than the value given by Eq. (10-3) except that b_w is 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, A_s provided is at least one-third greater than that required by analysis for required strength U, not including the environmental durability factor S_d.

10.5.4 In addition to the provisions of 10.5.1 and 10.5.3, walls, structural slabs, mats, and footings of uniform thickness, shall meet the requirements for reinforcement in 12.13.2. Walls with axial loading shall also meet the minimum areas of reinforcement as required by 15.4.

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

COMMENTARY

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.

To prevent such a failure, a minimum amount of tensile reinforcement is required by 10.5.1 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 $200b_wd/f_y$ 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.

R10.5.3 The minimum reinforcement required by Eq. (10-3) is to 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.

R10.5.4 The minimum reinforcement required for slabs should be equal to the same amount as that required by 12.13.2 for shrinkage and temperature reinforcement.

Slabs-on-ground 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 in slabs-on-ground 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.

The provisions of 10.5.4 differ from those in ACI 318 because structural slabs, walls, and other flexural members of uniform thickness in environmental engineering concrete structures are commonly loaded over their full surface areas. This is unlike similar members in non-environmental structures designed in accordance with ACI 318 that are commonly partially loaded, which allows for the redistribution of stresses.

R10.6—Distribution of flexural reinforcement

R10.6.1 Many structures designed by the working stress method and with low steel stress served their intended functions with very limited flexural cracking. When high-

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controlled sections. The requirements of this section do not apply to load combinations that include earthquake effects.

10.6.2 Distribution of flexural reinforcement in two-way slabs shall also meet the requirements of 14.3. 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.

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.

10.6.4 The maximum allowable tensile stress $f_{s,max}$ in flexural reinforcement at service loads shall not exceed that given by Eq. (10-4) and Eq. (10-5):

10.6.4.1 In normal environmental exposure areas as defined in 10.6.4.5

$$f_{s,max} = \frac{320}{\beta\sqrt{s^2 + 4(2 + d_b/2)^2}} \tag{10-4}$$

COMMENTARY

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. 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 with a yield strength of 60,000 psi is used.

Extensive laboratory work (Gergely and Lutz 1968; Kaar 1966; Base et al. 1966) 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 most-effective crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

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 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 engineering concrete structures is to control stress levels to limits shown to control corrosion effectively rather than predict crack widths with any precision.

R10.6.3 Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

R10.6.4 This section replaces the z factor requirements of the ACI 350-01. The maximum allowable stresses are now specified directly as a function of bar spacing (Beeby 1979; Frosch 1999). Figures R10.6.4a through R10.6.4d 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.

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but need not be less than 20,000 psi for one-way and 24,000 psi for two-way members and shall not exceed $0.6f_y$ but no greater than 36,000 psi.

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 and shall not exceed $0.6f_y$ but no greater than 36,000 psi.

10.6.4.3 In Eq. (10-4) and Eq. (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 calculated by

$$\beta = \frac{h - c}{d - c} \quad (10\text{-}6)$$

where c is calculated at service loads. Instead 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.

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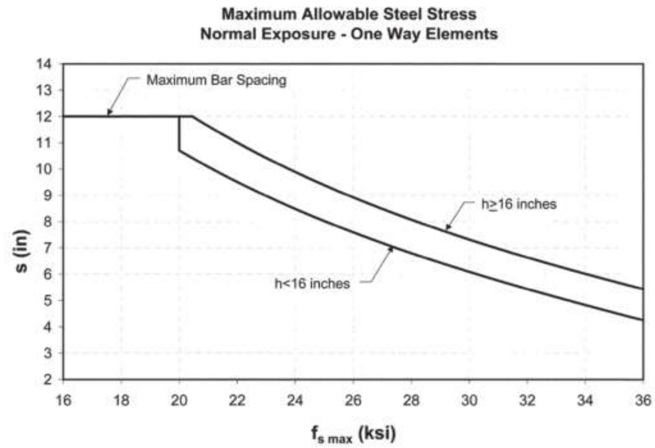

Fig. R10.6.4a—Maximum allowable steel stress, normal exposure—one-way elements.

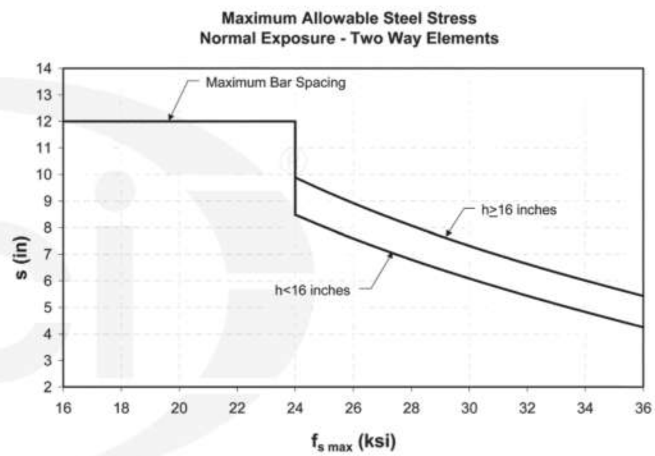

Fig. R10.6.4b—Maximum allowable steel stress, normal exposure—two-way elements.

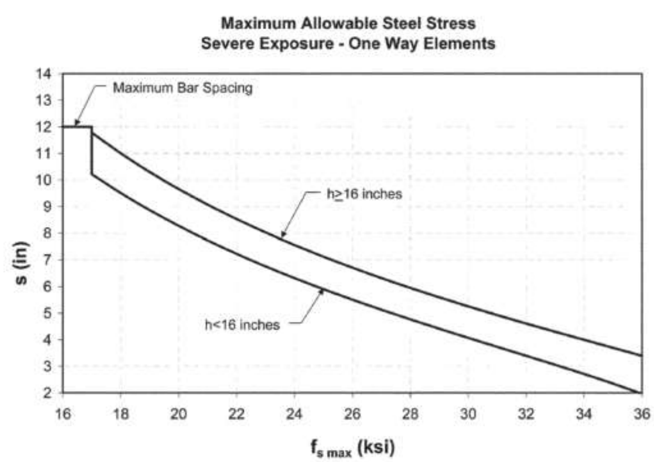

Fig. R10.6.4c—Maximum allowable steel stress, severe exposure—one-way elements.

Crack widths in environmental engineering concrete structures are highly variable. In previous codes, provisions were given for distribution of reinforcement that were based on empirical equations using a calculated maximum crack

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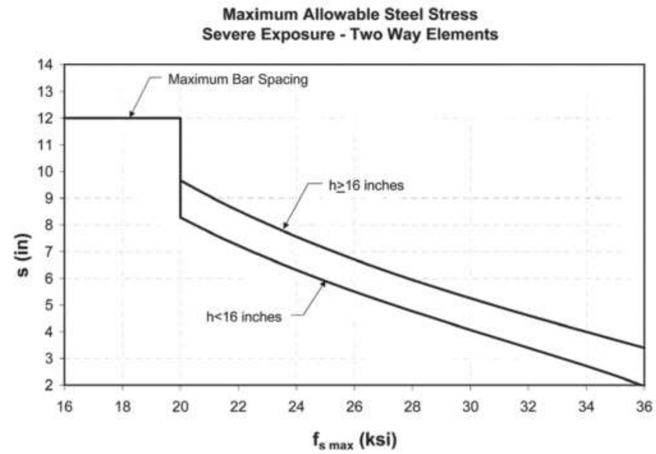

Fig. R10.6.4d—Maximum allowable steel stress, severe exposure—two-way elements.

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 (Darwin et al. 1985; Oesterle 1997) shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels. Although numerous 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 consolidation, limiting maximum bar stresses, and equally distributing more smaller bars rather than few larger bars on tension faces.

Testing has shown that the inclusion of epoxy-coated reinforcement will cause an increase in the crack width of flexural members by approximately 30 percent (Frantz and Breen 1980). While crack control aimed at minimizing corrosion of reinforcement may not be of concern when epoxy-coated reinforcement is used, the effects of larger crack widths may be a concern for aesthetic reasons, durability, and liquid tightness. Crack widths may be decreased by reducing the reinforcement stress.

The calculated stress f_{ess} in reinforcement closest to the tension face at service load should be computed based on the unfactored moment.

10.6.5 Where appearance of the concrete surface is of concern and c_c exceeds 3 in., the service load flexural tensile stress shall 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

$$s = 15\left(\frac{36{,}000}{f_s}\right) - 2.5c_c \qquad (10\text{-}7)$$

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 Eq. (10-5). Equation (10-6) is taken from ACI 318-08 Section 10.6.4 and is intended to limit surface cracks to a width that is generally acceptable in practice.

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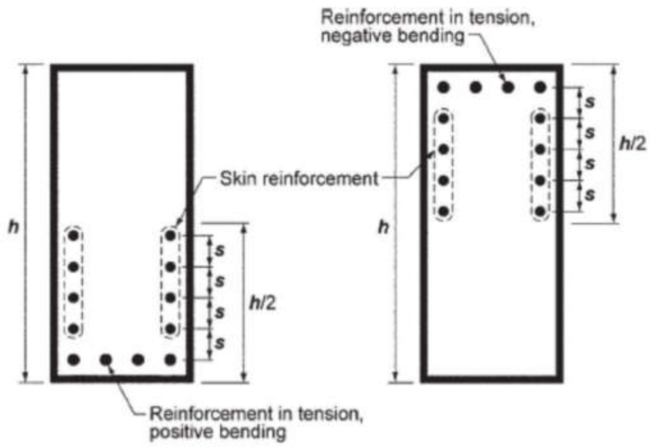

Fig. R10.6.7—Skin reinforcement for beams and joists with **d** *> 36 in.*

This section was updated in the 2005 edition of ACI 318 to reflect the higher service stresses that occur in flexural reinforcement with the use of the load combinations introduced in the 2002 ACI 318 Code. The maximum bar spacing is specified directly to control cracking (Beeby 1979; Frosch 1999; Darwin et al. 1985). For the usual case of beams with Grade 60 reinforcement and 2 in. clear cover to the main reinforcement, with f_s = **40,000 psi**, the maximum bar spacing is 10 in.

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 8.12, 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 should 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.

10.6.7 Where h of a beam or joist exceeds 36 in., longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member. Skin reinforcement shall extend for a distance $h/2$ from the tension face. The spacing s shall be as provided in 10.6.4 and 10.6.5, where c_c is the least distance from the surface of the skin reinforcement or prestressing steel to the side face. 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.

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 (Frosch 2002; Chow et al. 1953) (refer to 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 Deep beams are members loaded on one face and supported on the opposite face so that compression struts can develop between the loads and the supports, and have either:

(a) clear spans ℓ_n equal to or less than four times the overall member depth h, or

(b) regions with concentrated loads within a distance $2h$ from the face of the support.

R10.7—Deep beams

The span-to-depth ratios used to define deep beams in the 1999 and earlier ACI 318 codes were based on papers published in 1946 and 1953. The definitions of deep beams given in Chapters 10 and 11 of these earlier codes were different from each other and different from the ACI 318-11 code definition that is based on D-region behavior (refer to Appendix B). Since ACI 318-02, the definitions of deep

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Deep beams shall be designed either taking into account nonlinear distribution of strain, or by Appendix B. Refer also to 11.7.1 and 12.8.10.6.

10.7.2 Deep beams shall satisfy the requirements of 11.7.

10.7.3 Minimum area of flexural tension reinforcement, $A_{s,min}$, shall conform to 10.5.

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 12.7.

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 to special moment frames or special structural walls designed in accordance with Chapter 13.

10.9—Limits for reinforcement of compression members

10.9.1 Area of longitudinal reinforcement, A_{st}, for noncomposite compression members shall not be less than $0.01A_g$ nor more than $0.08A_g$.

COMMENTARY

beams in Sections 10.7.1 and 11.7.1 are consistent with each other.

This Code does not contain detailed requirements for designing deep beams for flexure except that nonlinearity of strain distribution and lateral buckling is to be considered. Suggestions for the design of deep beams for flexure are given in PCA (1946), Park and Paulay (1975), and Chow (1953).

R10.8—Design dimensions for compression members

With ACI 318-71, 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. When small sections are used, there is a greater need for careful workmanship and shrinkage stresses have increased significance.

R10.8.2, R10.8.3, and R10.8.4 For column design (Furlong 1971), 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 shall not be considered as carrying load; however, the effects of the additional concrete on member stiffness should be included in the structural analysis. The effects of the additional concrete also should 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 (refer to R9.4), should be considered. The percentage of reinforcement in columns 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

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COMMENTARY

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 (1933) and minimum reinforcement ratios of 0.01 and 0.005 were recommended for spiral and tied columns, respectively; however, in all editions of the ACI Building Code since 1936, the minimum ratio has been 0.01 for both types of laterally reinforced columns.

Maximum reinforcement: Extensive tests of the ACI column investigation (Saatcioglu and Razvi 2002) 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 (1933) 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 ACI Building Code, the limit for tied columns with bending was raised to 0.08. Since the ACI 318-63, 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 4 for bars within rectangular or circular ties, 3 for bars within triangular ties, and 6 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 transverse 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 should be considered in design.

10.9.3 Ratio of spiral reinforcement, $\boldsymbol{\rho_s}$, shall be not less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} \qquad (10\text{-}8)$$

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 reinforce-

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where the value of f_{yt} used in Eq. (10-8) shall not exceed 100,000 psi. For f_{yt} greater than 60,000 psi, lap splices according to 12.10.1.4.5(a) shall not be used.

10.10—Slenderness effects in compression members

10.10.1 Slenderness effects shall be permitted to be neglected in the following cases:

(a) for compression members not braced against sidesway when

$$\frac{k\ell_u}{r} \le 22 \tag{10-9}$$

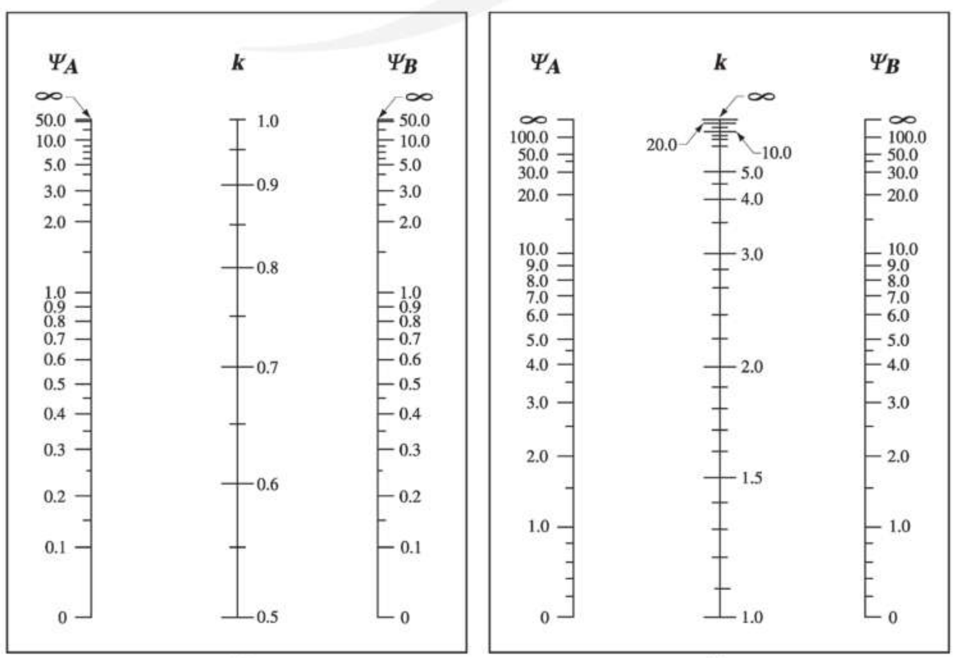
COMMENTARY

ment required by Eq. (10-8) 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 (1933) and has been a part of the ACI 318 code since 1936. The derivation of Eq. (10-8) 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. Research (Pessiki et al. 2001; Richart et al. 1929; Column Research Council 1966) has indicated that reinforcement with a 100,000 psi yield strength can be used for confinement. For this Code, the limit in yield strength for spiral reinforcement was increased from 60,000 psi to 100,000 psi.

R10.10—Slenderness effects in compression members

The slenderness provisions have been reorganized in the ACI 318-08 to reflect the evolution of current practice where second-order effects are considered primarily using computer analysis techniques.

R10.10.1 Second-order effects in many structures are negligible. In these cases, it is not necessary to consider slenderness effects and compression members can be designed based on forces determined from first-order analyses. Slenderness effects can be neglected in both braced and unbraced systems depending on the $k\ell_u/r$ of the member. In evalua-



(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. R10.10.1—Effective length factors **k**.

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(b) for compression members braced against sidesway when

$$\frac{k\ell_u}{r} \leq 34 - 12(M_1/M_2) \leq 40 \qquad (10\text{-}10)$$

where M_1/M_2 is positive if the column is bent in single curvature, and negative if the member is bent in double curvature.

It shall be permitted to consider compression members braced against sidesway when bracing elements have a total stiffness, resisting lateral movement of that story, of at least 12 times the gross stiffness of the columns within the story.

10.10.1.1 The unsupported length of a compression member, ℓ_u, shall be taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support in the direction being considered. Where column capitals or haunches are present, ℓ_u shall be measured to the lower extremity of the capital or haunch in the plane considered.

10.10.1.2 It shall be permitted to take the radius of gyration, 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 r for the gross concrete section.

10.10.2 When slenderness effects are not neglected as permitted by 10.10.1, 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 satisfying 10.10.3, 10.10.4, or 10.10.5. These members shall also satisfy 10.10.2.1 and 10.10.2.2. 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 contract documents or the analysis shall be repeated.

10.10.2.1 Total moment, including second-order effects in compression members, restraining beams, or other structural members, shall not exceed 1.4 times the moment due to first-order effects.

COMMENTARY

tion of unbraced systems, moments are based on first-order elastic analysis.

The primary design aid to estimate the effective length factor k is the Jackson and Moreland Alignment Charts (Fig. R10.10.1), which allow a graphical determination of k for a column of constant cross section in a multibay frame (SP-17(97); MacGregor et al. 1970).

Equation (10-10) is based on Eq. (10-14) assuming that a 5 percent increase in moments due to slenderness is acceptable (MacGregor 1992). As a first approximation, k may be taken equal to 1.0 in Eq. (10-10).

ACI 318R used to state that a compression member may be assumed braced if located in a story in which the bracing elements have a total stiffness, resisting lateral movement of the story, at least six times the sum of the stiffnesses of all the columns in the story. In ACI 318-95, the language was changed to: "… the bracing elements 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." The change was made because of some concern that the multiplier of six might not be conservative enough. For ACI 318-08, a more conservative multiplier of 12 was chosen. The stiffness of the lateral bracing is considered based on the principal directions of the framing system. Bracing elements in typical building structures consist of shear walls or lateral braces. Torsional eccentricity of the structural system can increase second-order effects and should be considered.

R10.10.2 Design may be based on a nonlinear second-order analysis, an elastic second-order analysis, or the moment magnifier approach (MacGregor 1992; Ford et al. 1981; Wilson 1997). The structure that is analyzed should 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.

Several methods have been developed to evaluate slenderness effects in compression members that are subject to biaxial bending. A review of some of these methods is presented in MacGregor and Hage (1977).

R10.10.2.1 If the weight of a structure is high in proportion to its lateral stiffness, excessive P-Δ effects (where secondary moments are more than 25 percent of the primary moments) may result, which will eventually introduce singularities into the solution to the equations of equilibrium, indicating physical structural instability (ASCE 7-05). Analytical research (Grossman 1990) on reinforced concrete frames showed that the probability of stability failure increases rapidly when the stability index Q exceeds 0.2, which is equivalent to a secondary-to-primary moment ratio of 1.25. According to ASCE/SEI 7-10 (Grossman 1987), the maximum value of the stability coefficient θ, which is close to the ACI stability coefficient Q, is 0.25. This value is

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equivalent to a secondary-to-primary moment ratio of 1.33. The upper limit of 1.4 on the secondary-to-primary moment ratio was chosen considering the above. By providing an upper limit on the second-order moment, it was unnecessary to retain the stability check in ACI 318-05 Section 10.13.6.

10.10.2.2 Second-order effects shall be considered along the length of compression members. It shall be permitted to account for these effects using the moment magnification procedure outlined in 10.10.6.

R10.10.2.2 The maximum moment in a compression member may occur between its ends. While second-order computer analysis programs may be used to evaluate magnification of the end moments, magnification between the ends may not be accounted for unless the member is subdivided along its length. The magnification may be evaluated using the procedure outlined in 10.10.6.

10.10.3 *Nonlinear second-order analysis*

Second-order analysis shall consider material nonlinearity, member curvature and lateral drift, duration of loads, shrinkage and creep, and interaction with the supporting foundation. The analysis procedure shall have been shown to result in prediction of strength in substantial agreement with results of comprehensive tests of columns in statically indeterminate reinforced concrete structures.

R10.10.3 *Nonlinear second-order analysis*

The nonlinear 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 1. The concept of a stiffness reduction factor ϕ_K is discussed in R10.10.4. For consistency with the second-order analysis in 10.10.4, the stiffness reduction factor ϕ_K can be taken as 0.80.

10.10.4 *Elastic second-order analysis*

Elastic second-order analysis shall consider section properties determined taking into account the influence of axial loads, the presence of cracked regions along the length of the member, and the effects of duration of loads.

R10.10.4 *Elastic second-order analysis*

The stiffnesses EI used in an analysis 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 values 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.

Design computations for slender columns and frames include both a strength reduction factor ϕ for the cross-sectional strength and a stiffness reduction factor ϕ_K for the member stiffnesses. The variability in the cross-sectional strength is accounted for by ϕ in the interaction diagrams while the variability of member stiffness is accounted for by ϕ_K in the structural analysis.

10.10.4.1 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, I

Compression members:

Columns: $0.70I_g$

Walls, uncracked: $0.70I_g$

Walls, cracked: $0.35I_g$

Flexural members:

Beams: $0.35I_g$

R10.10.4.1 The values of E_c, I, and A 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 of the concrete, E_c, is based on the specified concrete compressive strength while the sway deflections are a function of the average concrete strength, which is higher. The moments of inertia are taken from McGregor and Hage (1977), which are multiplied by the stiffness reduction factor $\phi_K = 0.875$. For example, the moment of inertia for columns is $0.875(0.80I_g) = 0.70I_g$.

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Flat plates and flat slabs: $\mathbf{0.25}\boldsymbol{I_g}$

(c) Area: $\mathbf{1.0}\boldsymbol{A_g}$

Alternatively, the moments of inertia of compression and flexural members $\boldsymbol{I}$ shall be permitted to be computed as follows:

Compression members:

$$I = \left(0.80 + 25\frac{A_{st}}{A_g}\right)\left(1 - \frac{M_u}{P_u h} - 0.5\frac{P_u}{P_o}\right) I_g \leq 0.875 I_g \qquad (10\text{-}11)$$

where $\boldsymbol{P_u}$ and $\boldsymbol{M_u}$ shall be from the particular load combination under consideration, or the combination of $\boldsymbol{P_u}$ and $\boldsymbol{M_u}$ resulting in the smallest value of $\boldsymbol{I}$. $\boldsymbol{I}$ need not be taken less than $\mathbf{0.35}\boldsymbol{I_g}$.

Flexural members:

$$I = (0.10 + 25\rho)\left(1.2 - 0.2\frac{b_w}{d}\right) I_g \leq 0.5 I_g \qquad (10\text{-}12)$$

For continuous flexural members, $\boldsymbol{I}$ shall be permitted to be taken as the average of values obtained from Eq. (10-12) for the critical positive and negative moment sections. $\boldsymbol{I}$ need not be taken less than $\mathbf{0.25}\boldsymbol{I_g}$.

The cross-sectional dimensions and reinforcement ratio used in the above formulas shall be within 10 percent of the dimensions and reinforcement ratio shown on the contract documents or the stiffness evaluation shall be repeated.

COMMENTARY

These two effects result in an overestimation of the second-order deflections on the order of 20 to 25 percent, corresponding to an implicit stiffness reduction of 0.80 to 0.85 on the stability calculation.

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 $\boldsymbol{I_g}$ of a T-beam as two times the $\boldsymbol{I_g}$ for the web, $\mathbf{2}(\boldsymbol{b_w h^3}/\mathbf{12})$.

If the factored moments and shears from an analysis based on the moment of inertia of a wall, taken equal to $\mathbf{0.70}\boldsymbol{I_g}$, indicate that the wall will crack in flexure, based on the modulus of rupture, the analysis should be repeated with $\boldsymbol{I} = \mathbf{0.35}\boldsymbol{I_g}$ in those stories where cracking is predicted using factored loads.

The values of the moments of inertia were derived for nonprestressed members. For prestressed members, the moments of inertia may differ depending on the amount, location, and type of the reinforcement and the degree of cracking prior to ultimate. The stiffness values for prestressed concrete members should include an allowance for the variability of the stiffnesses.

Section 10.10 provides requirements for strength and assumes frame analyses will be carried out using factored loads. Analyses of deflections, vibrations, and building periods are needed at various service (unfactored) load levels (Khuntia and Ghosh 2004a,b) to determine the serviceability of the structure and to estimate the wind forces in wind tunnel laboratories. The moments of inertia of the structural members in the service load analyses should 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 1.0/0.70 = 1.43 times the moments of inertia given here for service load analyses.

Equations (10-11) and (10-12) provide more refined values of $\boldsymbol{EI}$ considering axial load, eccentricity, reinforcement ratio, and concrete compressive strength as presented in Mirza et al. (1987) and Mirza (1990). The stiffnesses provided in these references are applicable for all levels of loading, including service and ultimate, and consider a stiffness reduction factor ϕ_K comparable to that included in 10.10.4.1(b). For use at load levels other than ultimate, $\boldsymbol{P_u}$ and $\boldsymbol{M_u}$ should be replaced with their appropriate values at the desired load level.

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10.10.4.2 When sustained lateral loads are present, $\boldsymbol{I}$ for compression members shall be divided by $(\mathbf{1} + \beta_{ds})$. The term β_{ds} shall be taken as the ratio of maximum factored sustained shear within a story to the maximum factored shear in that story associated with the same load combination, but shall not be taken greater than 1.0.

COMMENTARY

R10.10.4.2 The unusual case of sustained lateral loads might exist, for example, if there were permanent lateral loads resulting from unequal earth pressures on two sides of an environmental structure.

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10.10.5 *Moment magnification procedure*

Columns and stories in structures shall be designated as nonsway or sway columns or stories. The design of columns

COMMENTARY

R10.10.5 *Moment magnification procedure*

This section describes an approximate design procedure that uses the moment magnifier concept to account for slenderness effects. Moments computed using an ordinary first-

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in nonsway frames or stories shall be based on 10.10.6. The design of columns in sway frames or stories shall be based on 10.10.7.

10.10.5.1 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.

10.10.5.2 It also shall be permitted to assume a story within a structure is nonsway if

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us} \ell_c} \leq 0.05 \qquad (10\text{-}13)$$

where $\sum \boldsymbol{P_u}$ and $\boldsymbol{V_{us}}$ are the total factored vertical load and the horizontal story shear, respectively, in the story being evaluated, and Δ_o is the first-order relative lateral deflection between the top and the bottom of that story due to $\boldsymbol{V_{us}}$.

10.10.6 *Moment magnification procedure—nonsway*

Compression members shall be designed for factored axial force $\boldsymbol{P_u}$ and the factored moment amplified for the effects of member curvature $\boldsymbol{M_c}$ as follows

$$M_c = \delta M_2 \qquad (10\text{-}14)$$

where

$$\delta = \frac{C_m}{1 - \dfrac{P_u}{0.75 P_c}} \geq 1.0 \qquad (10\text{-}15)$$

and

$$P_c = \frac{\pi^2 EI}{(k \ell_u)^2} \qquad (10\text{-}16)$$

COMMENTARY

order frame analysis are multiplied by a moment magnifier that is a function of the factored axial force $\boldsymbol{P_u}$ and the critical buckling load $\boldsymbol{P_c}$ for the column. Nonsway and sway frames are treated separately. A first-order frame analysis is an elastic analysis that does not include the internal force effects resulting from deflections.

The moment magnifier design method requires the designer to distinguish between nonsway frames, which are designed according to 10.10.6, and sway frames, which are designed according to 10.10.7. 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, 10.10.5.1 and 10.10.5.2 give two possible ways of doing this. In 10.10.5.1, a story in a frame is said to be nonsway if the increase in the lateral load moments resulting from $\boldsymbol{P}$-Δ effects does not exceed 5 percent of the first-order moments (Grossman 1990). Section 10.10.5.2 gives an alternative method of determining this based on the stability index for a story $\boldsymbol{Q}$. In computing $\boldsymbol{Q}$, $\sum \boldsymbol{P_u}$ should correspond to the lateral loading case for which $\sum \boldsymbol{P_u}$ is greatest. A frame may contain both nonsway and sway stories. This test would not be suitable if $\boldsymbol{V_{us}}$ is zero.

If the lateral load deflections of the frame have been computed using service loads and the service load moments of inertia given in 10.10.4, it is permissible to compute $\boldsymbol{Q}$ in Eq. (10-13) 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.

R10.10.6 *Moment magnification procedure—nonsway*

The ϕ used in the design of slender columns represent two different sources of variability. First, the stiffness reduction ϕ_K accounts for the variability in the stiffness $\boldsymbol{EI}$ and the moment magnification analysis. Second, the strength reduction ϕ for tied and spiral columns accounts for the variability of the strength of the cross section. Studies reported in Lai and MacGregor (1983) indicate that the stiffness reduction factor ϕ_K and the cross-sectional strength reduction ϕ do not have the same values. 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 factor in Eq. (10-15) is the stiffness reduction factor ϕ_K. The factor is based on the probability of understrength of a single isolated slender column. In the case of a multistory frame, the column and frame deflections depend on the average concrete strength which is higher than the strength of the concrete in the critical single understrength column. For this reason, the value of ϕ_K in 10.10.4 is 0.875.

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10.10.6.1 EI shall be taken as

$$EI = \frac{(0.2E_cI_g + E_sI_{se})}{1+\beta_{dns}} \tag{10-17}$$

or

$$EI = \frac{0.4E_cI_g}{1+\beta_{dns}} \tag{10-18}$$

Alternatively, EI shall be permitted to be computed using the value of I from Eq. (10-11) divided by $(1 + \beta_{dns})$.

10.10.6.2 The term β_{dns} shall be taken as the ratio of maximum factored axial sustained load to maximum factored axial load associated with the same load combination but shall not be taken greater than 1.0.

10.10.6.3 The effective length factor k shall be permitted to be taken as 1.0.

10.10.6.4 For members without transverse loads between supports, C_m shall be taken as

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \tag{10-19}$$

where M_1/M_2 is positive if the column is bent in single curvature, and negative if the member is bent in double curvature. For members with transverse loads between supports, C_m shall be taken as 1.0.

10.10.6.5 Factored moment, M_2, in Eq. (10-14) shall not be taken less than

$$M_{2,min} = P_u(0.6 + 0.03h) \tag{10-20}$$

COMMENTARY

R10.10.6.1 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 nonlinearity of the concrete stress-strain curve. Either Eq. (10-17) or Eq. (10-18) may be used to compute EI. Equation (10-17) was derived for small eccentricity ratios and high levels of axial load where slenderness effects are most pronounced. Equation (10-18) is a simplified approximation to Eq. (10-17) and is less accurate (Bianchini et al. 1960). For improved accuracy, EI can be approximated using the suggested E and I values provided by Eq. (10-11) divided by $(1 + \beta_{dns})$.

R10.10.6.2 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, by dividing EI by $(1 + \beta_{dns})$. Both the concrete and steel terms in Eq. (10-17) are divided by $(1 + \beta_{dns})$ to reflect the premature yielding of steel in columns subjected to sustained load. For simplification, it can be assumed that $\beta_{dns} = 0.6$. In this case, Eq. (10-18) becomes

$$EI = 0.25E_cI_g$$

R10.10.6.3 The effective length factor for a compression member considering braced behavior ranges from 0.5 and 1.0. While lower values can be justified, it is recommended that a k value of 1.0 be used. If lower values are used, the calculation of k should be based on analysis of the frame using E_c and I values given in 10.10.4. The Jackson and Moreland Alignment Charts (Fig. R10.10.1) can be used to estimate lower values of k (SP-17(97); MacGregor et al. 1970).

R10.10.6.4 The factor C_m is a correction factor relating the actual moment diagram to an equivalent uniform moment diagram. 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 should be based on an equivalent uniform moment C_mM_2 that would lead to the same maximum moment when magnified (MacGregor 1993).

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. (10-14). C_m is to be taken as 1.0 for this case.

R10.10.6.5 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 should be based on the minimum eccentricity given

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about each axis separately, where 0.6 and h are in inches. For members in which $M_{2,min}$ exceeds M_2, the value of C_m in Eq. (10-19) shall either be taken equal to 1.0, or shall be based on the ratio of the computed end moments, M_1/M_2.

10.10.7 *Moment magnification procedure—sway*

Moments M_1 and M_2 at the ends of an individual compression member shall be taken as

$$M_1 = M_{1ns} + \delta_s M_{1s} \quad (10\text{-}21)$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \quad (10\text{-}22)$$

where δ_s is computed according to 10.10.7.3 or 10.10.7.4.

10.10.7.1 Flexural members shall be designed for the total magnified end moments of the compression members at the joint.

10.10.7.2 The effective length factor k shall be determined using the values of E_c and I given in 10.10.4 and shall not be less than 1.0.

10.10.7.3 The moment magnifier δ_s shall be calculated as

$$\delta_s = \frac{1}{1-Q} \geq 1 \quad (10\text{-}23)$$

If δ_s calculated by Eq. (10-23) exceeds 1.5, δ_s shall be calculated using second-order elastic analysis or 10.10.7.4.

COMMENTARY

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. (10-19) in determining the ratio M_1/M_2 for the column when the design should 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.

R10.10.7 *Moment magnification procedure—sway*

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.

R10.10.7.1 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. This section provides that the designer makes certain that the restraining flexural members have the capacity to resist the magnified column moments.

R10.10.7.3 The iterative P-Δ analysis for second-order moments can be represented by an infinite series. The solution of this series is given by Eq. (10-23) (Grossman 1990). Ospina and Alexander (1998) shows that Eq. (10-23) closely predicts the second-order moments in a sway frame until δ_s exceeds 1.5.

The P-Δ moment diagrams for deflected columns are curved, with Δ related to the deflected shape of the columns. Equation (10-23) and most commercially available second-order frame analyses have been derived assuming that the P-Δ 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-Δ moment diagram. The curved P-Δ moment diagrams lead to lateral displacements on the order of 15 percent larger than those from the straight-line P-Δ moment diagrams. This effect can be included in Eq. (10-23) by writing the denominator as ($\mathbf{1} - \mathbf{1.15}Q$) rather than ($\mathbf{1} - Q$). The 1.15 factor has been left out of Eq. (10-23) for simplicity.

If deflections have been calculated using service loads, Q in Eq. (10-23) should be calculated in the manner explained in R10.10.5.

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The Q factor analysis is based on deflections calculated using the values of E_c and I from 10.10.4, which include the equivalent of a stiffness reduction factor ϕ_K. These values of E_c and I 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-Δ moments. As a result, no additional ϕ is needed. Once the moments are established using Eq. (10-23), selection of the cross sections of the columns involves the strength reduction factors ϕ from 9.3.2.2.

10.10.7.4 Alternatively, it shall be permitted to calculate δ_s as

$$\delta_s = \frac{1}{1 - \frac{\Sigma P_u}{0.75 \Sigma P_c}} \geq 1 \quad (10\text{-}24)$$

where $\sum P_u$ is the summation for all the factored vertical loads in a story and $\sum P_c$ is the summation for all sway-resisting columns in a story. P_c is calculated using Eq. (10-16) with k determined from 10.10.7.2 and EI from 10.10.6.1, where β_{ds} shall be substituted for β_{dns}.

R10.10.7.4 To check the effects of story stability, δ_s is computed as an averaged value for the entire story based on use of $\sum P_u / \sum P_c$. This reflects the interaction of all sway resisting columns in the story in the P-Δ effects because the lateral deflection of all columns in the story should 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 should be checked using 10.10.6.

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. (10-24) is a stiffness reduction factor ϕ_K as explained in R10.10.6.

In the calculation of EI, β_{ds} will normally be zero for a sway frame because the lateral loads are generally of short duration. 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_{ds} in 10.10.4.2 gives $\beta_{ds} = 0$. In the unusual case of a sway frame where the lateral loads are sustained, β_{ds} 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.

10.11—Axially loaded members supporting slab system

Axially loaded members supporting a slab system included within the scope of 14.1 shall be designed as provided in Chapter 10 and in accordance with the additional requirements of Chapter 14.

10.12—Transmission of column loads through floor system

If f_c' of a column is greater than 1.4 times that of the floor system, transmission of load through the floor system shall be provided by 10.12.1, 10.12.2, or 10.12.3.

R10.12—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 (Everard and Cohen 1964). 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 10.12.1 or 10.12.2 should be used for corner

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or edge columns. Methods in 10.12.1, 10.12.2, or 10.12.3 should be used for interior columns with adequate restraint on all four sides.

10.12.1 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 7.1.3.1 and 7.1.3.2.

R10.12.1 Application of the concrete placement procedure described in 10.12.1 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 responsibility of the licensed design professional to indicate on the contract documents where the high- and low-strength concretes are to be placed.

With the ACI 318-83, 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.

10.12.2 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.

10.12.3 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 10.12.3, the ratio of column concrete strength shall not be taken greater than 2.5 for design.

R10.12.3 Research (Hawkins 1968) 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.

10.13—Composite compression members

10.13.1 Composite compression members shall include all such members reinforced longitudinally with structural steel shapes, pipe, or tubing with or without longitudinal bars.

R10.13—Composite compression members

R10.13.1 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.

10.13.2 Strength of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

R10.13.2 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 the SP-17(97) chapter on Columns, but with γ slightly greater than 1.0.

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10.13.3 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.

10.13.4 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.

10.13.5 For evaluation of slenderness effects, radius of gyration, $\boldsymbol{r}$, of a composite section shall be not greater than the value given by

$$r = \sqrt{\frac{(E_c I_g/5) + E_s I_{sx}}{(E_c A_g/5) + E_s A_{sx}}} \qquad (10\text{-}25)$$

and, as an alternative to a more accurate calculation, $\boldsymbol{EI}$ in Eq. (10-16) shall be taken either as Eq. (10-17) or

$$EI = \frac{(E_c I_g/5)}{1+\beta_{dns}} + E_s I_{sx} \qquad (10\text{-}26)$$

10.13.6 *Structural steel-encased concrete core*

10.13.6.1 For a composite member with a concrete core encased by structural steel, the thickness of the steel encasement shall be not less than

$$b\sqrt{\frac{f_y}{3E_s}} \text{ for each face of width } b$$

nor

$$h\sqrt{\frac{f_y}{8E_s}} \text{ for circular sections of diameter } h$$

10.13.6.2 Longitudinal bars located within the encased concrete core shall be permitted to be used in computing $\boldsymbol{A_{sx}}$ and $\boldsymbol{I_{sx}}$.

10.13.7 *Spiral reinforcement around structural steel core*

A composite member with spirally reinforced concrete around a structural steel core shall conform to 10.13.7.1 through 10.13.7.4.

10.13.7.1 Design yield strength of structural steel core shall be the specified minimum yield strength for the grade of structural steel used but not to exceed 50,000 psi.

10.13.7.2 Spiral reinforcement shall conform to 10.9.3.

COMMENTARY

R10.13.3 and R10.13.4 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.

R.10.13.5 Equation (10-25) is given because the rules of 10.10.1.2 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 $\boldsymbol{EI}$. Accordingly, both the concrete and steel terms in Eq. (10-17) 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. (10-26) was revised in the 1980 ACI 318 Building Code supplement so that only the $\boldsymbol{EI}$ of the concrete is reduced for sustained load effects.

R10.13.6 *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.

R10.13.7 *Spiral reinforcement around structural steel core*

Concrete that is laterally contained by a spiral has increased 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.

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10.13.7.3 Longitudinal bars located within the spiral shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.13.7.4 Longitudinal bars located within the spiral shall be permitted to be used in computing $\boldsymbol{A_{sx}}$ and $\boldsymbol{I_{sx}}$.

10.13.8 *Tie reinforcement around structural steel core*
A composite member with transversely tied concrete around a structural steel core shall conform to 10.13.8.1 through 10.13.8.7.

10.13.8.1 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.

10.13.8.2 Transverse ties shall extend completely around the structural steel core.

10.13.8.3 Transverse 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 reinforcement of equivalent area shall be permitted.

10.13.8.4 Vertical spacing of transverse ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or 0.5 times the least side dimension of the composite member.

10.13.8.5 Longitudinal bars located within the ties shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.13.8.6 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.

10.13.8.7 Longitudinal bars located within the ties shall be permitted to be used in computing $\boldsymbol{A_{sx}}$ and $\boldsymbol{I_{sx}}$.

10.14—Bearing strength

10.14.1 Design bearing strength of concrete shall not exceed $\boldsymbol{\phi(0.85f_c'A_1)}$, 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 by not more than 2.

COMMENTARY

R10.13.8 *Tie reinforcement around structural steel core*
The design yield strength of the steel core should be limited to that which would not generate 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.

Research[10.49] has shown that the required amount of tie reinforcement around the structural steel core is sufficient for the longitudinal steel bars to be included in the flexural stiffness of the composite column.

R10.14—Bearing strength

R10.14.1 This section deals with bearing strength on concrete supports. The permissible bearing stress of $\boldsymbol{0.85f_c'}$ is based on tests reported in Reference 10.50 (refer also to 16.8).

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 11.11.

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.

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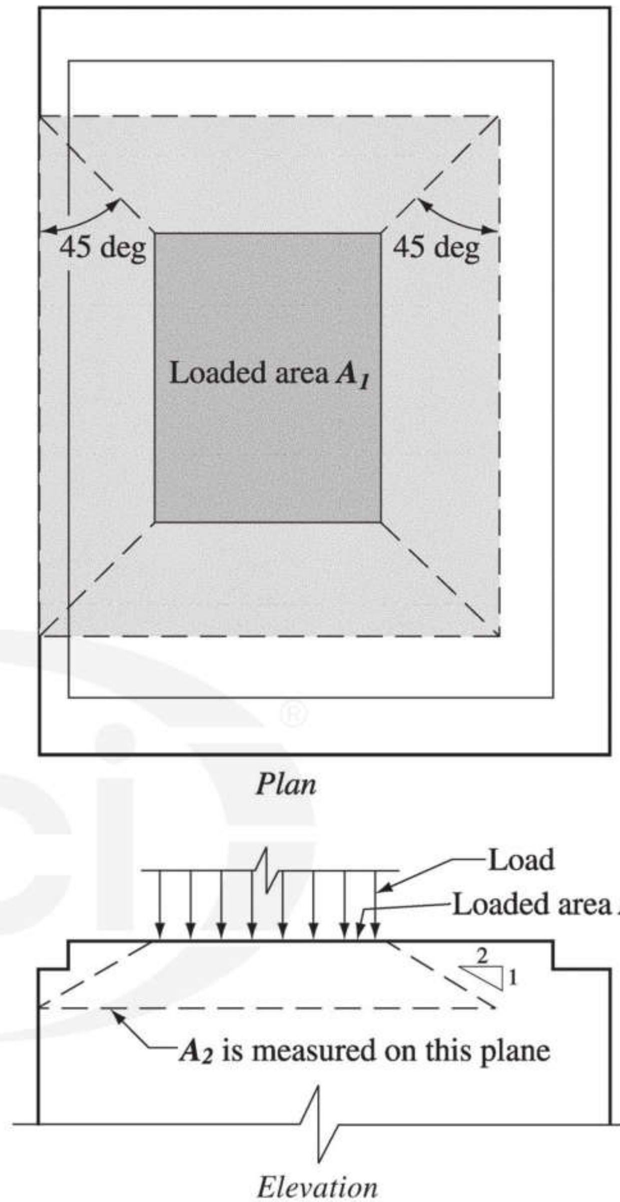

Fig. R10.14.1—Application of frustum to find A_2 *in stepped or sloped supports.*

Figure R10.14.1 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.

10.14.2 Section 10.14 does not apply to post-tensioning anchorages.

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CHAPTER 11—SHEAR AND TORSION

11.1—Shear strength

11.1.1 Except for members designed in accordance with Appendix B, design of cross sections subject to shear shall be based on

$$\phi V_n \geq V_u \tag{11-1}$$

where V_u is the factored shear force at the 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 calculated in accordance with 11.2, 11.3, or 11.11, and V_s is nominal shear strength provided by shear reinforcement calculated in accordance with 11.4, 11.9.9, or 11.11.

11.1.1.1 In determining V_n, effect of any openings in members shall be considered.

11.1.1.2 In determining 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

COMMENTARY

CHAPTER R11—SHEAR AND TORSION

R11.1—Shear strength

This chapter includes shear and torsion provisions for both nonprestressed and prestressed concrete members. The shear-friction concept (11.6) is particularly applicable to design of reinforcement details in precast structures. Special provisions are included for deep flexural members (11.7), brackets and corbels (11.8), and shear walls (11.9). Shear provisions for slabs and footings are given in 11.11.

The shear strength is based on an average shear stress on the full effective cross section $b_w d$. 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 Joint ACI-ASCE Committee 426 (1973), MacGregor and Hanson (1969), and Joint ACI-ASCE Committee 326 (1962).

Appendix B allows the use of strut-and-tie models in the shear design of disturbed regions. The traditional shear design procedures, which ignore D-regions, are acceptable in shear spans that include B-regions.

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 Joint ACI-ASCE Committee (1973) as well as in Barney et al. (1977) and Schlaich et al. (1987).

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 (Joint Committee 1940).

R11.1.2 Because of a lack of test data and practical experience with concretes having compressive strengths greater than 10,000 psi, ACI 318-89 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 this 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 Mphonde and Frantz (1984), Elzanaty et al. (1986), Roller and Russell (1990), and Johnson and Ramirez (1989), an increase in the minimum amount of transverse reinforcement is required for high-

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having minimum web reinforcement in accordance with 11.4.6.3, 11.4.6.4, or 11.5.5.2.

11.1.3 Computation of maximum 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

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 V_u computed at a distance d.

COMMENTARY

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 ACI 318-89 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).

R11.1.3.1 The closest inclined crack to the support of the beam in Fig R11.1.3.1(a) will extend upward 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. Two things are 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. R11.1.3.1(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 should 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.

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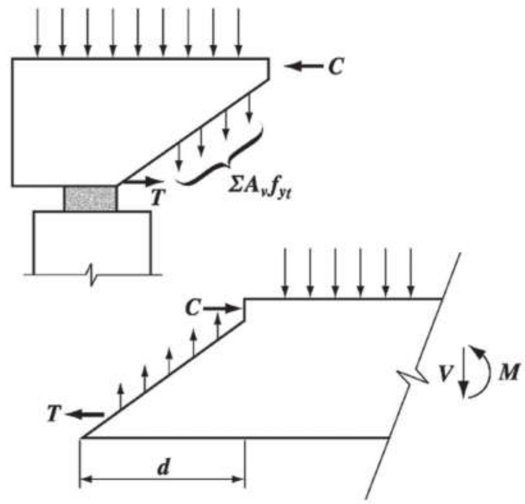

Fig. R11.1.3.1(a)—Free body diagrams of the end of a beam

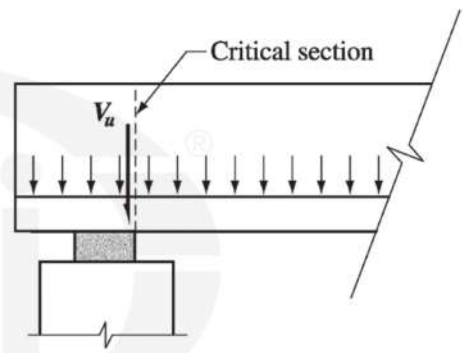

Fig. R11.1.3.1(b)—Location of critical section for shear in a member loaded near bottom.

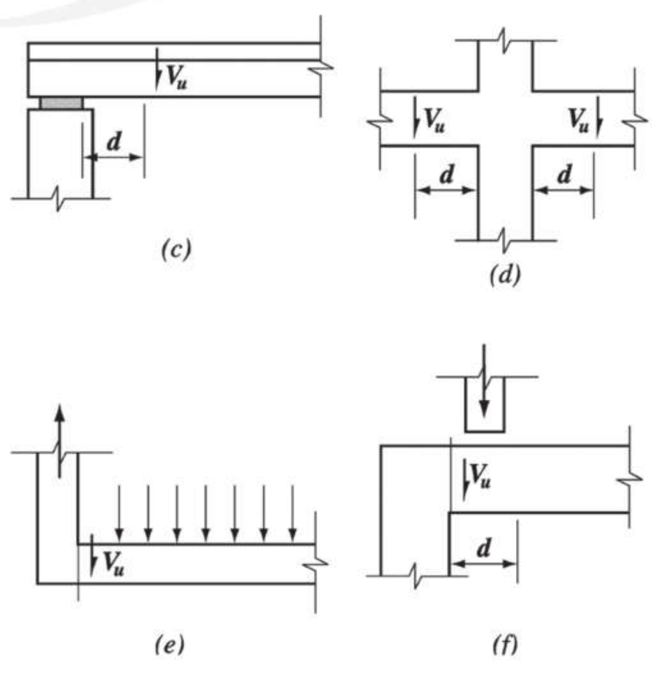

Fig. R11.1.3.1(c), (d), (e), (f)—Typical support conditions for locating factored shear force $\mathbf{V_u}$.

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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 applied below the potential inclined crack.

(3) Members loaded such that the shear at sections between the support and a distance $\boldsymbol{d}$ from the support differs radically from the shear at distance $\boldsymbol{d}$. 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 $\boldsymbol{h}/\mathbf{2}$ from face of support shall be permitted to be designed for $\boldsymbol{V_u}$ computed at a distance $\boldsymbol{h}/\mathbf{2}$.

R11.1.3.2 Because $\boldsymbol{d}$ frequently varies in prestressed members, the location of the critical section has arbitrarily been taken as $\boldsymbol{h}/\mathbf{2}$ from the face of the support.

11.1.4 For deep beams, brackets and corbels, walls, and slabs and footings, the special provisions of 11.7 through 11.11 shall apply.

11.2—Shear strength provided by concrete for nonprestressed members

11.2.1 $\boldsymbol{V_c}$ shall be computed by provisions of 11.2.1.1 through 11.2.1.3, unless a more detailed calculation is made in accordance with 11.2.2. Throughout this chapter, except in 11.6, $\boldsymbol{\lambda}$ shall be as defined in 8.6.1

R11.2—Shear strength provided by concrete for nonprestressed members

11.2.1.1 For members subject to shear and flexure only

$$V_c = 2\lambda\sqrt{f_c'}b_w d \qquad (11\text{-}3)$$

R11.2.1.1 Refer to R11.2.2.1.

11.2.1.2 For members subject to axial compression

$$V_c = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f_c'}b_w d \qquad (11\text{-}4)$$

Quantity $\boldsymbol{N_u}/\boldsymbol{A_g}$ shall be expressed in psi.

R11.2.1.2 and R11.2.1.3 Refer to R11.2.2.2.

11.2.1.3 For members subject to significant axial tension, $\boldsymbol{V_c}$ shall be taken as zero unless a more detailed analysis is made using 11.2.2.3.

11.2.2 $\boldsymbol{V_c}$ shall be permitted to be computed by the more detailed calculation of 11.2.2.1 through 11.2.2.3.

11.2.2.1 For members subject to shear and flexure only

$$V_c = \left(1.9\lambda\sqrt{f_c'}+2500\rho_w\frac{V_u d}{M_u}\right)b_w d \qquad (11\text{-}5)$$

R11.2.2.1 Equation (11-5) is the basic expression for shear strength of members without shear reinforcement (Joint ACI-ASCE Committee 326). The three variables in Eq. (11-5)—$\lambda\sqrt{f_c'}$ (as a measure of concrete tensile strength), $\boldsymbol{\rho_w}$, and

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but not greater than $3.5\lambda\sqrt{f_c'}\,b_wd$. When computing V_c by Eq. (11-5), V_ud/M_u shall not be taken greater than 1.0 where M_u occurs simultaneously with V_u at section considered.

11.2.2.2 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_ud/M_u not then limited to 1.0, where

$$M_m = M_u - N_u\frac{(4h-d)}{8} \quad (11\text{-}6)$$

However, V_c shall not be taken greater than

$$V_c = 3.5\lambda\sqrt{f_c'}b_wd\sqrt{1+\frac{N_u}{500A_g}} \quad (11\text{-}7)$$

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).

11.2.2.3 For members subject to significant axial tension

$$V_c = 2\left(1+\frac{N_u}{500A_g}\right)\lambda\sqrt{f_c'}b_wd \quad (11\text{-}8)$$

but not less than zero, where N_u is negative for tension. N_u/A_g shall be expressed in psi.

COMMENTARY

V_ud/M_u—are known to affect shear strength, although some research data (Join ACI-ASCE Committee 426 1973; MacGregor and Hanson 1969) indicate that Eq. (11-5) overestimates the influence of f_c' and underestimates the influence of ρ_w and V_ud/M_u. Further information (Kani 1966) has indicated that shear strength decreases as the overall depth of the member increases.

The minimum value of M_u equal to V_ud 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 $2\sqrt{f_c'}\,b_wd$ as permitted in 11.2.1.1.

R11.2.2.2 Equations (11-6) and (11-7) for members subject to axial compression in addition to shear and flexure are derived in the Joint ACI-ASCE Committee 326 (1962) report. 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, Eq. (11-7) or Eq. (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. R11.2.2.2. The background for these equations is discussed and comparisons are made with test data in MacGregor and Hanson (1969).

Because of the complexity of Eq. (11-5) and (11-6), an alternative design provision, Eq. (11-4), is permitted.

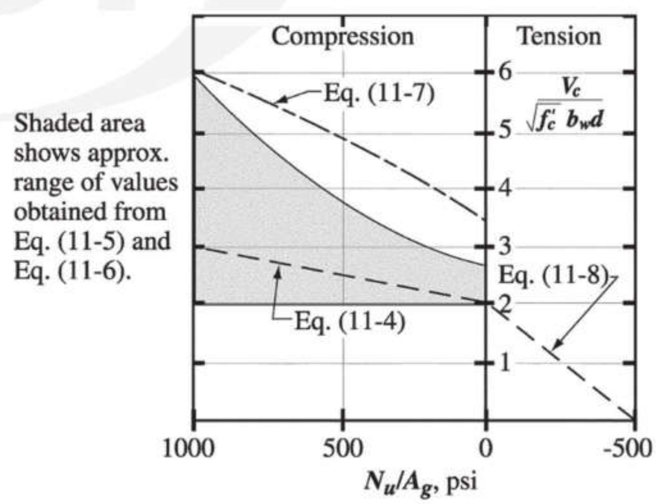

Fig. R11.2.2.2—Comparison of shear strength equations for members subject to axial load.

R11.2.2.3 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 judgment is required 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 adequate expansion joints and minimum reinforcement. It may be desirable to

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design shear reinforcement to carry total shear if there is uncertainty about the magnitude of axial tension.

11.2.3 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 d as 0.8 times the diameter of the concrete section.

R11.2.3 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 (Joint ACI-ASCE Committee 426 1973; Faradji and Diaz de Cossio 1965; Khalifa and Collins 1981).

11.3—Shear strength provided by concrete for prestressed members

11.3.1 For the provisions of 11.3, d shall be taken as the distance from extreme compression fiber to centroid of prestressed and nonprestressed longitudinal tension reinforcement, if any, but need not be taken less than $0.80h$.

11.3.2 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 11.3.3

$$V_c = \left(0.6\lambda\sqrt{f_c'} + 700\frac{V_u d_p}{M_u}\right)b_w d \qquad (11\text{-}9)$$

but V_c need not be taken less than $2\lambda\sqrt{f_c'}\,b_w d$. V_c shall not be taken greater than $5\lambda\sqrt{f_c'}\,b_w d$ or the value given in 11.3.4 or 11.3.5. $V_u d_p/M_u$ shall not be taken greater than 1.0, where M_u occurs simultaneously with V_u at the section considered.

R11.3—Shear strength provided by concrete for prestressed members

R11.3.2 Equation (11-9) offers a simple means of computing V_c for prestressed concrete beams (MacGregor and Hanson 1969). It may be applied to beams having prestressed reinforcement only, or to members reinforced with a combination of prestressed reinforcement and nonprestressed deformed bars. Eq. (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, $V_u d_p/M_u$ can be expressed as

$$\frac{V_u d_p}{M_u} = \frac{d_p(\ell - 2x)}{x(\ell - x)}$$

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 11.3.2 varies as shown in Fig. R11.3.2. Design aids based on this equation are given in PCI (2017).

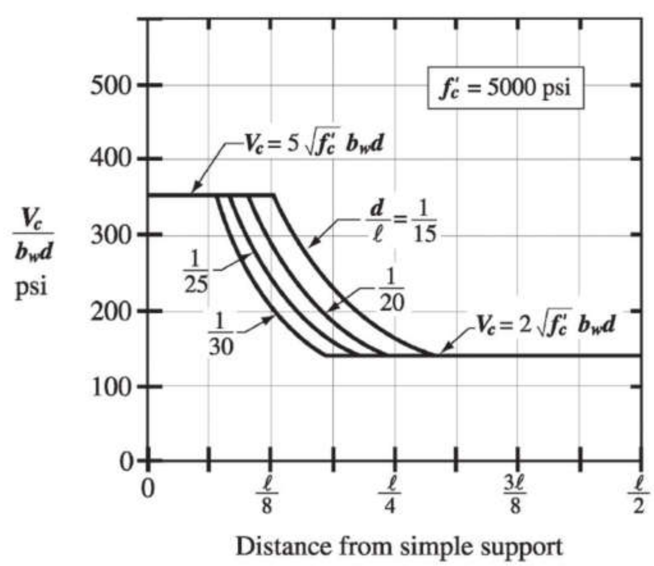

Fig. R11.3.2—Application of Eq. (11-9) to uniformly loaded prestressed members.

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11.3.3 V_c shall be permitted to be computed in accordance with 11.3.3.1 and 11.3.3.2, where V_c shall be the lesser of V_{ci} and V_{cw}.

COMMENTARY

R11.3.3 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. R11.3.3.

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.

Equations (11-10) and (11-12) may be used to determine the shear forces causing flexure-shear and web-shear cracking, respectively. The nominal 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 ACI Committee 318 (1965).

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

$$V = \frac{V_i M_{cre}}{M_{max}}$$

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, and live load. In computing M_{cre} 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).

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

$$V_{ci} = 0.6\lambda\sqrt{f_c'}b_w d + \frac{V_u M_{ct}}{M_u}$$

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11.3.3.1 V_{ci} shall be computed by

$$V_{ci} = 0.6\lambda\sqrt{f_c'}b_w d_p + V_d + \frac{V_i M_{cre}}{M_{max}} \quad (11\text{-}10)$$

where d_p need not be taken less than **0.80*h*** and

$$M_{cre} = (I/y_t)(6\lambda\sqrt{f_c'} + f_{pe} - f_d) \quad (11\text{-}11)$$

and values of M_{max} and V_i shall be computed from the load combination causing maximum factored moment to occur at the section. V_{ci} need not be taken less than $1.7\lambda\sqrt{f_c'}b_w d$.

11.3.3.2 V_{cw} shall be computed by

$$V_{cw} = (3.5\lambda\sqrt{f_c'} + 0.3f_{pc})b_w d_p + V_p \quad (11\text{-}12)$$

where d_p need not be taken less than **0.80*h*.**

Alternatively, V_{cw} shall be computed as the shear force corresponding to dead load plus live load that results in a principal tensile stress of $4\lambda\sqrt{f_c'}$ at the centroidal axis of

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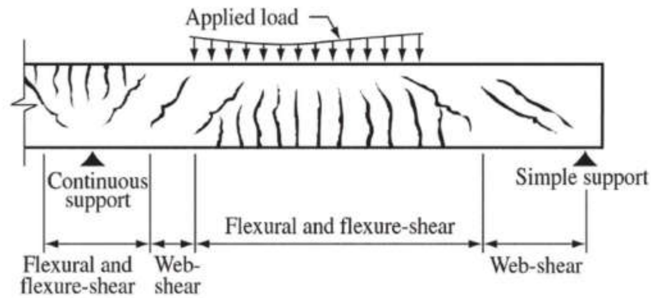

Fig. R11.3.3—Types of cracking in concrete beams.

where

$$M_{ct} = (I/y_t)(6\lambda\sqrt{f_c'} + f_{pe})$$

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_{cre} in 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. The cracking moment M_{ct} reflects the total stress change from effective prestress to a tension of $6\lambda\sqrt{f_c'}$, 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 $4\lambda\sqrt{f_c'}$ at the centroidal axis of the cross section. V_p is calculated from the effective prestress force without load factors.

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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.

11.3.4 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.

11.3.5 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 11.3.2 or 11.3.3. 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.

11.4—Shear strength provided by shear reinforcement

11.4.1 *Types of shear reinforcement*

11.4.1.1 Shear reinforcement consisting of the following shall be permitted:

(a) Stirrups perpendicular to axis of member

(b) Welded wire reinforcement with wires located perpendicular to axis of member

(c) Spirals, circular ties, or hoops

11.4.1.2 For nonprestressed members, shear reinforcement shall be permitted to also consist of:

(a) Stirrups making an angle of 45 degrees or more with longitudinal tension reinforcement

(b) Longitudinal reinforcement with bent portion making an angle of 30 degrees or more with the longitudinal tension reinforcement

(c) Combinations of stirrups and bent longitudinal reinforcement

11.4.2 The values of f_y and f_{yt} used in design of shear reinforcement shall not exceed 60,000 psi, except the value shall not exceed 80,000 psi for welded deformed wire reinforcement.

COMMENTARY

R11.3.4 and R11.3.5 The effect of the reduced prestress near the ends of pretensioned beams on the shear strength should be taken into account. Section 11.3.4 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 11.3.5 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.

R11.4—Shear strength provided by shear reinforcement

R11.4.2 Limiting the values of f_y and f_{yt} used in the design of shear reinforcement to 60,000 psi provides a control on diagonal crack width. In ACI 318-95, the limitation of 60,000 psi for shear reinforcement was raised to 80,000 psi for welded deformed wire reinforcement. Research (Guimares et al. 1992; Griezic et al 1994; Furlong 1991) has indicated that the performance of higher-strength steels as shear reinforcement has been satisfactory. In particular,

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full-scale beam tests described in Griezic et al. (1994) indicated that the widths of inclined shear cracks at service load levels were less for beams reinforced with smaller-diameter deformed welded wire reinforcement cages designed on the basis of a yield strength of 75,000 psi than beams reinforced with deformed Grade 60 stirrups.

11.4.3 Where the provisions of 11.4 are applied to prestressed members, d shall be taken as the distance from extreme compression fiber to centroid of the prestressed and nonprestressed longitudinal tension reinforcement, if any, but need not be taken less than $0.80h$.

R11.4.3 Although the value of d may vary along the span of a prestressed beam, studies (MacGregor and Hanson 1969) have shown that, for prestressed concrete members, d need not be taken less than $0.80h$. The beams considered had some straight tendons or reinforcing bars at the bottom of the section and had stirrups that enclosed the steel.

11.4.4 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fiber and shall be developed at both ends according to 12.8.13.

R11.4.4 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 12.8.13.

11.4.5 *Spacing limits for shear reinforcement*

11.4.5.1 Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/2$ in nonprestressed members and $0.75h$ in prestressed members.

11.4.5.2 Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45-degree 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.

11.4.5.3 Where V_s exceeds $4\sqrt{f_c'}\,b_w d$, maximum spacings given in 11.4.5.1 and 11.4.5.2 shall be reduced by one-half.

11.4.5.4 Spacing of shear reinforcement shall not exceed 12 in.

R11.4.5.4 A maximum spacing of 12 in. for reinforcement is required for crack control.

11.4.6 *Minimum shear reinforcement*

R11.4.6 *Minimum shear reinforcement*

11.4.6.1 A minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where V_u exceeds $0.5\phi V_c$, except in members satisfying one or more of (a) through (e):

(a) Footings and solid slabs

R11.4.6.1 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 V_u is greater than $0.5\phi V_c$. Solid 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. However, research (Angelakos et al 2001; Lubell et al. 2004; Brown et al. 2006) has shown that deep, lightly reinforced one-way slabs and beams, particularly if constructed with high-strength concrete or concrete having a small coarse aggre-

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gate size, may fail at shear loads less than V_c calculated from Eq. (11-3), particularly if subjected to concentrated loads. Because of this, the exclusion for certain beam types in 11.4.6.1(e) is restricted to cases in which h does not exceed 24 in. For beams where f_c' is greater than 7000 psi, consideration should be given to providing minimum shear reinforcement when h is greater than 18 in. and V_u is greater than $0.5\phi V_c$.

Even when V_u is less than $0.5\phi V_c$, 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 11.4.5 and 11.4.6 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.

(b) Hollow-core units with total untopped depth not greater than 12.5 in. and hollow-core units where V_u is not greater than $0.5\phi V_{cw}$

R11.4.6.1(b) Test results of hollow-core units (Becker and Buettner 1985; Anderson 1978) with h values of 12.5 in. and less have shown shear strengths greater than those calculated by Eq. (11-12) and (11-10). Test results of precast prestressed concrete hollow-core units (Hawkins and Ghosh 2006) with greater depths have shown that web-shear strengths in end regions can be less than strengths computed by Eq. (11-12). By contrast, flexure-shear strengths in those tests equaled or exceeded strengths computed by Eq. (11-10).

(c) Concrete joist construction defined by 8.13

(d) Beams with h not greater than 10 in.

(e) Beams integral with slabs with h not greater than 24 in., and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web

(f) Beams constructed of steel fiber-reinforced, normal-weight concrete with f_c' not exceeding 6000 psi, h not greater than 24 in., and V_u not greater than $2\phi\sqrt{f_c'}\,b_w d$.

R11.4.6.1(f) This exception is intended to provide a design alternative to the use of shear reinforcement, as defined in 11.4.1.1, for members with longitudinal flexural reinforcement in which V_u does not exceed $\phi 2\sqrt{f_c'}\,b_w d$. Fiber-rein-

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forced concrete beams with hooked or crimped steel fibers in dosages as required by 5.6.6.2 have been shown, through laboratory tests, to exhibit shear strengths larger than $3.5\sqrt{f_c'}b_wd$ (Hawkins and Ghosh 2006). There are no data for the use of steel fibers as shear reinforcement in concrete members exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources. Where steel fibers are used as shear reinforcement in corrosive environments, corrosion protection should be considered.

11.4.6.2 Minimum shear reinforcement requirements of 11.4.6.1 shall be permitted to be waived if shown by test that required M_n and V_n 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.

R11.4.6.2 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 (ϕ = **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 a reduced member strength, a satisfactory margin of safety will be retained.

11.4.6.3 Where shear reinforcement is required by 11.4.6.1 or for strength and where 11.5.1 allows torsion to be neglected, $A_{v,min}$ for prestressed (except as provided in 11.4.6.4) and nonprestressed members shall be computed by

$$A_{v,min} = 0.75\sqrt{f_c'}\frac{b_w s}{f_{yt}} \tag{11-13}$$

but shall not be less than $(50b_ws)/f_{yt}$.

R11.4.6.3 Previous versions of the Code have required a minimum area of transverse reinforcement that is independent of concrete strength. Tests (Roller and Russell 1990) 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 (11-13) provides for a gradual increase in the minimum area of transverse reinforcement, while maintaining the previous minimum value.

11.4.6.4 For prestressed members with an effective prestress force not less than 40 percent of the tensile strength of the flexural reinforcement, $A_{v,min}$ shall not be less than the smaller value from Eq. (11-13) and (11-14).

$$A_{v,min} = \frac{A_{ps}f_{pu}s}{80f_{yt}d}\sqrt{\frac{d}{b_w}} \tag{11-14}$$

R11.4.6.4 Tests (Oleson et al. 1967) 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 11.4.6.4. This equation is discussed in Olesen et al. (1967).

11.4.7 *Design of shear reinforcement*

11.4.7.1 Where V_u exceeds ϕV_c, shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where V_s shall be computed in accordance with 11.4.7.2 through 11.4.7.9.

11.4.7.2 Where shear reinforcement perpendicular to axis of member is used

$$V_s = \frac{A_v f_{yt} d}{s} \tag{11-15}$$

where A_v is the area of shear reinforcement within spacing s.

R11.4.7 *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. However, considerable research on both nonprestressed and prestressed members 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 degrees.

Equations (11-15), (11-16), and (11-17) are presented in terms of nominal shear strength provided by the shear reinforcement V_s. 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

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11.4.7.3 Where circular ties, hoops, or spirals are used as shear reinforcement, V_s shall be computed using Eq. (11-15) where d is defined in 11.2.3 for circular members. A_v shall be taken as two times the area of the bar in a circular tie, hoop, or spiral at a spacing s; s is measured in a direction parallel to longitudinal reinforcement; and f_{yt} is the specified yield strength of circular tie, hoop, or spiral reinforcement.

11.4.7.4 Where inclined stirrups are used as shear reinforcement

$$V_s = \frac{A_v f_{yt}(\sin\alpha + \cos\alpha)d}{s} \quad (11\text{-}16)$$

where α is angle between inclined stirrups and longitudinal axis of the member, and s is measured in direction parallel to longitudinal reinforcement.

11.4.7.5 Where shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support

$$V_s = A_v f_y \sin\alpha \quad (11\text{-}17)$$

but not greater than $3\sqrt{f_c'}\,b_w d$ where α is angle between bent-up reinforcement and longitudinal axis of the member.

11.4.7.6 Where shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, V_s shall be computed by Eq. (11-16).

11.4.7.7 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

11.4.7.8 Where more than one type of shear reinforcement is used to reinforce the same portion of a member, V_s shall be computed as the sum of the values computed for the various types of shear reinforcement.

11.4.7.9 V_s shall not be taken greater than $8\sqrt{f_c'}\,b_w d$.

COMMENTARY

$$\frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_{yt} d}$$

Research (Anderson and Ramirez 1989; Leonhardt and Walther 1964) 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.

R11.4.7.3 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 11.2.3 (Faradji and Diaz de Cossio 1965; Khalifa and Collins 1981).

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11.5—Design for torsion

Design for torsion shall be in accordance with 11.5.1 through 11.5.6, or 11.5.7.

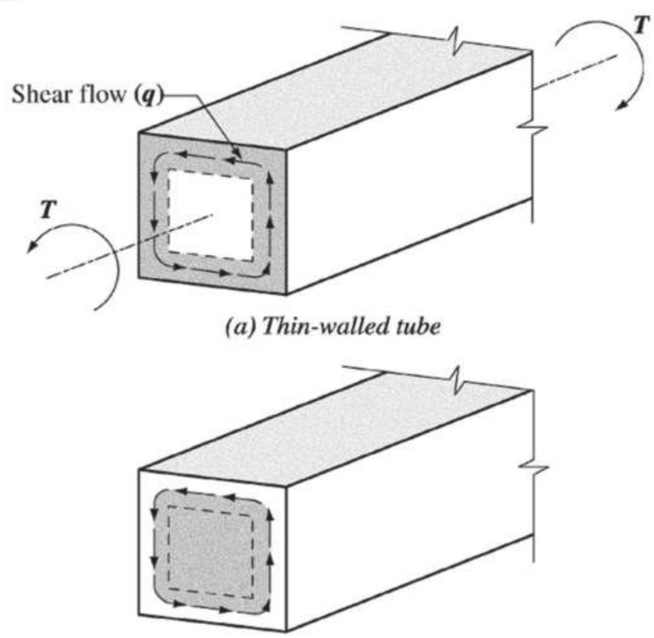
COMMENTARY

R11.5—Design for torsion

The design for torsion in 11.5.1 through 11.5.6 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 Fig. R11.5(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 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 acts as shown in Fig. R11.5(a) and 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. R11.5(b)), and t is the thickness of the wall at the point where τ is being computed. The shear flow follows the midthickness of the walls of the tube and A_o is the area enclosed by the path of the shear flow. For a hollow member with continuous walls, A_o includes the area of the hole.

In ACI 318-95, the elliptical interaction between the nominal shear strength provided by the concrete, V_c, and the nominal torsion strength provided by the concrete has been eliminated. V_c remains constant at the value it has when



(b) Area enclosed by shear flow path

Fig. R11.5—(a) Thin-walled tube; and (b) area enclosed by shear flow path.

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11.5.1 *Threshold torsion*

It shall be permitted to neglect torsion effects if the factored torsional moment $\boldsymbol{T_u}$ is less than:

(a) for nonprestressed members

$$\phi\lambda\sqrt{f_c'}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)$$

(b) for prestressed members

$$\phi\lambda\sqrt{f_c'}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)\sqrt{1+\frac{f_{pc}}{4\lambda\sqrt{f_c'}}}$$

(c) for nonprestressed members subjected to an axial tensile or compressive force

$$\phi\lambda\sqrt{f_c'}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)\sqrt{1+\frac{N_u}{4A_g\lambda\sqrt{f_c'}}}$$

For members cast monolithically with a slab, the overhanging flange width used in computing $\boldsymbol{A_{cp}}$ and $\boldsymbol{p_{cp}}$ shall conform to 14.2.4. For a hollow section, $\boldsymbol{A_g}$ shall be used in place of $\boldsymbol{A_{cp}}$ in 11.5.1, and the outer boundaries of the section shall conform to 14.2.4.

11.5.1.1 For isolated members with flanges and for members cast monolithically with a slab, the overhanging flange width used to compute $\boldsymbol{A_{cp}}$ and $\boldsymbol{p_{cp}}$ shall conform to 14.2.4, except that the overhanging flanges shall be neglected in cases where the parameter $\boldsymbol{A^2{}_{cp}/p_{cp}}$ calculated for a beam with flanges is less than that computed for the same beam ignoring the flanges.

COMMENTARY

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 MacGregor and Ghoneim (1995) and Hsu (1997).

R11.5.1 *Threshold torsion*

Torques that do not exceed approximately one-quarter of the cracking torque $\boldsymbol{T_{cr}}$ 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 $\boldsymbol{T_{cr}}$ is derived by replacing the actual section with an equivalent thin-walled tube with a wall thickness $\boldsymbol{t}$ prior to cracking of $\boldsymbol{0.75A_{cp}/p_{cp}}$ and an area enclosed by the wall centerline $\boldsymbol{A_o}$ equal to $\boldsymbol{2A_{cp}/3}$. Cracking is assumed to occur when the principal tensile stress reaches $\boldsymbol{4\lambda\sqrt{f_c'}}$. In a nonprestressed beam loaded with torsion alone, the principal tensile stress is equal to the torsional shear stress, $\boldsymbol{\tau = T/(2A_o t)}$. Thus, cracking occurs when $\boldsymbol{\tau}$ reaches $\boldsymbol{4\lambda\sqrt{f_c'}}$, giving the cracking torque $\boldsymbol{T_{cr}}$ as

$$T_{cr} = 4\lambda\sqrt{f_c'}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

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 $\boldsymbol{0.25T_{cr}}$, used in 11.5.1, corresponds to a reduction of 3 percent in the inclined cracking shear. This reduction in the inclined cracking shear was considered negligible. The stress at cracking $\boldsymbol{4\lambda\sqrt{f_c'}}$ 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 $\boldsymbol{4\lambda\sqrt{f_c'}}$ is $\boldsymbol{\sqrt{1+f_{pc}/(4\lambda\sqrt{f_c'})}}$ times the corresponding torque in a nonprestressed beam. A similar modification is made in part (c) of 11.5.1 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 $\boldsymbol{A_g/A_{cp}}$ greater than or equal to 0.95, can be ignored when computing the threshold torque in 11.5.1. 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 $\boldsymbol{0.25T_{cr}}$ would cause a reduction in the inclined cracking shear of approximately 25 percent. This reduction was judged to be excessive.

In ACI 350-06, two changes were made to modify 11.5.1 to apply to hollow sections. First, the minimum torque limits from ACI 350-01 were multiplied by $\boldsymbol{(A_g/A_{cp})}$ because tests of

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solid and hollow beams (Hsu 1968) 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 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.

11.5.2 *Calculation of factored torsional moment*

11.5.2.1 If the factored torsional moment T_u in a member is required to maintain equilibrium and exceeds the minimum value given in 11.5.1, the member shall be designed to carry T_u in accordance with 11.5.3 through 11.5.6.

11.5.2.2 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 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 11.5.2.4

$$\phi 4\lambda\sqrt{f_c'}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)$$

(b) For prestressed members, at the sections described in 11.5.2.5

$$\phi 4\lambda\sqrt{f_c'}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)\sqrt{1+\frac{f_{pc}}{4\lambda\sqrt{f_c'}}}$$

(c) For nonprestressed members subjected to an axial tensile or compressive force

$$\phi 4\lambda\sqrt{f_c'}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)\sqrt{1+\frac{N_u}{4A_g\lambda\sqrt{f_c'}}}$$

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 11.5.2.2.

R11.5.2 *Calculation of factored torsional moment*

R11.5.2.1 and R11.5.2.2 In designing for torsion in reinforced concrete structures, two conditions may be identified (Collins and Lampert 1973; Hsu and Burton 1974):

(a) The torsional moment cannot be reduced by redistribution of internal forces (11.5.2.1). 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. R11.5.2.1, torsion reinforcement designed according to 11.5.3 through 11.5.6 is to be provided to resist the total design torsional moments.

(b) The torsional moment can be reduced by redistribution of internal forces after cracking (11.5.2.2) 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. R11.5.2.2, the torsional stiffness before cracking corresponds to that of

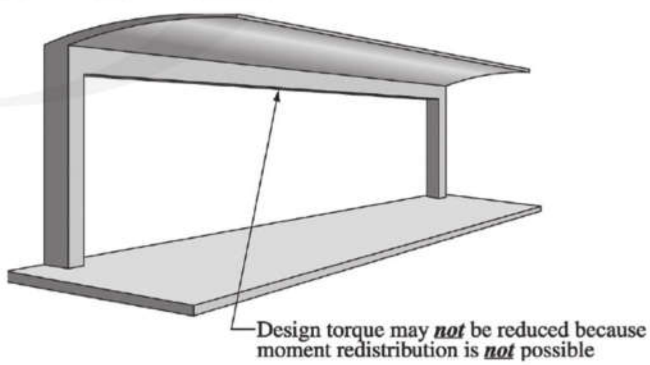

Fig. R11.5.2.1—Design torque may not be reduced (11.5.2.1).

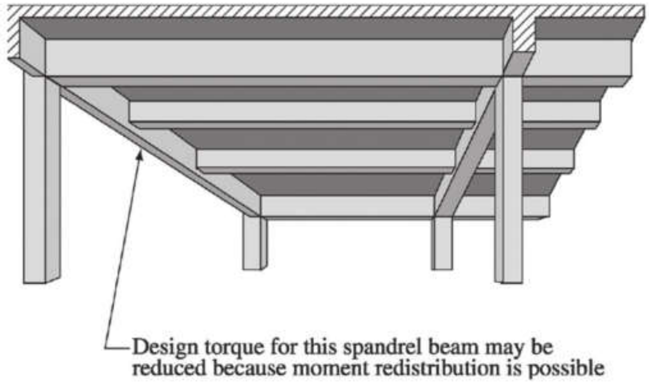

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Fig. R11.5.2.2—Design torque may be reduced (11.5.2.2).

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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 (Collins and Lampert 1973; Hsu and Burton 1974). The cracking torque under combined shear, flexure, and torsion corresponds to a principle tensile stress somewhat less than the $4\lambda\sqrt{f_c'}$ quoted in R11.5.1.

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 11.5.1, is not applied here. Thus, the torque after redistribution is larger and hence more conservative.

Section 11.5.2.2 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 11.5.1 and the values given in this section, torsion reinforcement should be designed to resist the computed torsional moments.

11.5.2.3 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.

11.5.2.4 In nonprestressed members, sections located less than a distance d from the face of a support shall be designed for not less than T_u computed at a distance d. If a concentrated torsional moment occurs within this distance, the critical section for design shall be at the face of the support.

R11.5.2.4 and R11.5.2.5 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.

11.5.2.5 In prestressed members, sections located less than a distance $h/2$ from the face of a support shall be designed for not less than T_u computed at a distance $h/2$. If a concentrated torsional moment occurs within this distance, the critical section for design shall be at the face of the support.

11.5.3 *Torsional moment strength*

R11.5.3 *Torsional moment strength*

11.5.3.1 The cross-sectional dimensions shall be such that:

(a) For solid sections

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2+\left(\frac{T_u p_h}{1.7A_{oh}^2}\right)^2} \le \phi\left(\frac{V_c}{b_w d}+8\sqrt{f_c'}\right) \quad (11\text{-}18)$$

(b) For hollow sections

$$\left(\frac{V_u}{b_w d}\right)+\left(\frac{T_u p_h}{1.7A_{oh}^2}\right) \le \phi\left(\frac{V_c}{b_w d}+8\sqrt{f_c'}\right) \quad (11\text{-}19)$$

R11.5.3.1 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 11.4.7.9 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

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For prestressed members, d shall be determined in accordance with 11.4.3.

11.5.3.2 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.

11.5.3.3 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.

11.5.3.4 The values of f_y and f_{yt} used for design of torsional reinforcement shall not exceed 60,000 psi.

11.5.3.5 Where T_u exceeds the threshold torsion, design of the cross section shall be based on

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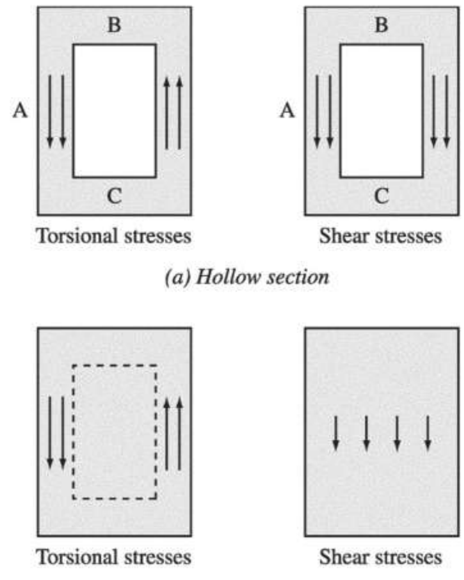

Fig. R11.5.3.1—Addition of torsional and shear stresses.

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. R11.5.3.1(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. R11.5.3.1(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.

R11.5.3.2 Generally, the maximum will be on the wall where the torsional and shearing stresses are additive (Point A in Fig. R11.5.3.1(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. R11.5.3.1(a). At these points, the stresses due to the shear force are usually negligible.

R11.5.3.4 Limiting the values of f_y and f_{yt} used in the design of torsion reinforcement to 60,000 psi provides a control on diagonal crack width.

R11.5.3.5 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

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$$\phi T_n \geq T_u \qquad (11\text{-}20)$$

11.5.3.6 T_n shall be computed by

$$T_n = \frac{2A_o A_t f_{yt}}{s}\cot\theta \qquad (11\text{-}21)$$

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 degrees nor larger than 60 degrees. It shall be permitted to take θ equal to:

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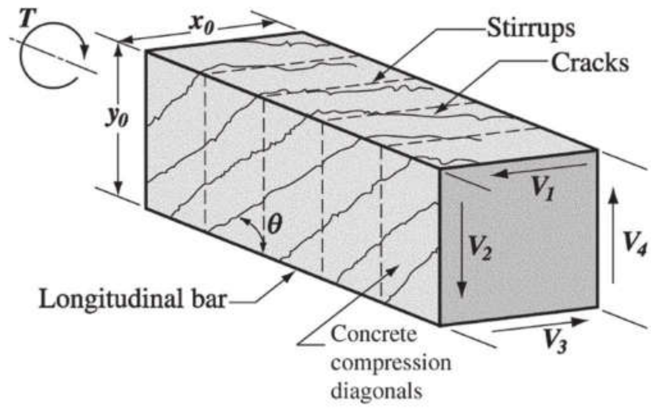

Fig. R11.5.3.6a—Space truss analogy.

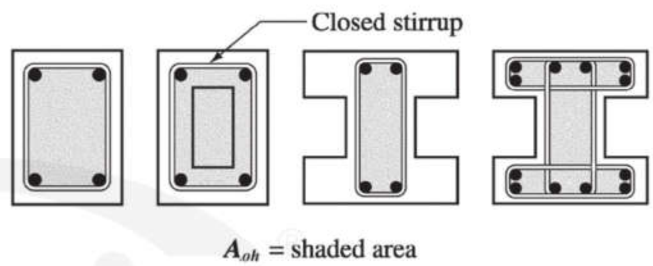

Fig. R11.5.3.6b—Definition of $\mathbf{A_{oh}}$.

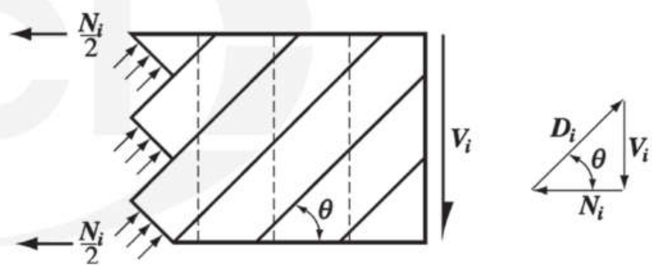

Fig. R11.5.3.7—Resolution of shear force $\mathbf{V_i}$ *into diagonal compression force* $\mathbf{D_i}$ *and axial tension force* N_i *in one wall of the tube.*

by stirrups and longitudinal steel with $T_c = 0$. At the same time, the nominal shear strength provided by concrete V_c is assumed to be unchanged by the presence of torsion. For beams with V_u greater than approximately $0.8\phi V_c$, the resulting amount of combined shear and torsional reinforcement is essentially the same as required by the ACI 318-89. For smaller values of V_u, more shear and torsion reinforcement will be required.

R11.5.3.6 Equation (11-21) is based on the space truss analogy shown in Fig. R11.5.3.6a 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 gross 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

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(a) 45 degrees for nonprestressed members or members with less prestress than in (b), or

(b) 37.5 degrees for prestressed members with an effective prestress force not less than 40 percent of the tensile strength of the longitudinal reinforcement

11.5.3.7 The additional area of longitudinal reinforcement to resist torsion, A_ℓ, shall not be less than

$$A_\ell = \frac{A_t}{s} p_h \left(\frac{f_{yt}}{f_y} \right) \cot^2 \theta \qquad (11\text{-}22)$$

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 11.5.5.2 or 11.5.5.3; f_{yt} refers to closed transverse torsional reinforcement, and f_y refers to longitudinal torsional reinforcement.

11.5.3.8 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.

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of the outermost closed transverse torsional reinforcement. The area A_{oh} is shown in Fig. R11.5.3.6b 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. R11.5.3.6b. The expression for A_o given by Hsu (1990) may be used if greater accuracy is desired.

The shear flow q in the walls of the tube, discussed in R11.5, 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. R11.5.3.6a.

The angle θ can be obtained by analysis (Hsu 1990) or may be taken to be equal to the values given in 11.5.3.6(a) or (b). The same value of θ should be used in both Eq. (11-21) and (11-22). As θ gets smaller, the number of stirrups required by Eq. (11-21) decreases. At the same time the amount of longitudinal steel required by Eq. (11-22) increases.

R11.5.3.7 Fig. R11.5.3.6a 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_i 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.

Fig. R11.5.3.7 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 strength $A_\ell f_y$ should be provided to resist the sum of the N_i forces, $\sum N_i$, acting in all 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 number 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, 11.5.3.7 states that the A_t/s used in calculating A_ℓ at any given section should be taken as the A_t/s calculated at that section using Eq. (11-21).

R11.5.3.8 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:

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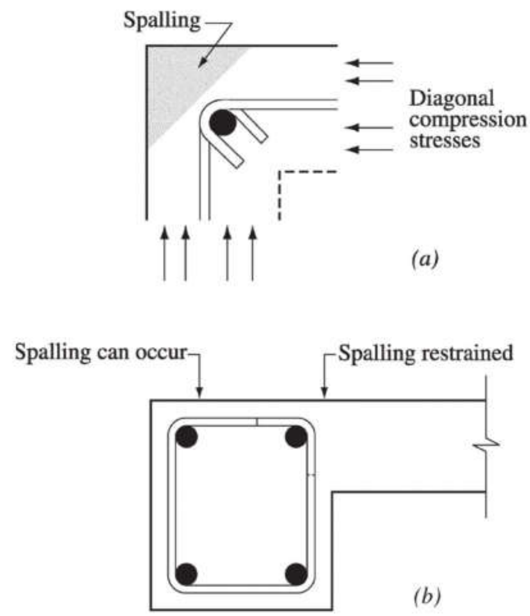

Fig. R11.5.4.2—Spalling of corners of beams loaded in torsion.

$$\text{Total}\left(\frac{A_{v+t}}{s}\right) = \frac{A_v}{s} + 2\frac{A_t}{s}$$

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, such as the 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 should be satisfied. The flexural steel should be extended a distance $\boldsymbol{d}$, but not less than $\mathbf{12}\boldsymbol{d_b}$, past where it is no longer needed for flexure as required in 12.8.10.3.

11.5.3.9 It shall be permitted to reduce the area of longitudinal torsion reinforcement in the flexural compression zone by an amount equal to $\boldsymbol{M_u/(0.9df_{yl})}$, where $\boldsymbol{M_u}$ occurs at the section simultaneously with $\boldsymbol{T_u}$, except that the reinforcement provided shall not be less than that required by 11.5.5.3 or 11.5.6.2.

R11.5.3.9 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.

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11.5.3.10 In prestressed beams:

(a) the total longitudinal reinforcement including prestressing steel at each section shall resist M_u at that section plus an additional concentric longitudinal tensile force equal to $A_\ell f_y$, based on T_u at that section, and

(b) the spacing of the longitudinal reinforcement including tendons shall satisfy the requirements in 11.5.6.2

11.5.3.11 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 11.5.3.10 in accordance with 11.5.3.9.

11.5.4 *Details of torsional reinforcement*

11.5.4.1 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

(b) A closed cage of welded-wire reinforcement with transverse wires perpendicular to the axis of the member

(c) In nonprestressed beams, spiral reinforcement.

11.5.4.2 Transverse torsional reinforcement shall be anchored by one of the following:

(a) A 135-degree standard hook or seismic hook conforming to 2.2 or seismic hook conforming to 2.2 around a longitudinal bar

(b) According to 12.8.13.2.1, 12.8.13.2.2, or 12.8.13.2.3 in regions where the concrete surrounding the anchorage is restrained against spalling by a flange or slab or similar member

COMMENTARY

R11.5.3.10 As explained in R11.5.3.7, torsion causes an axial tension force. In a nonprestressed beam, this force is resisted by longitudinal reinforcement having an axial tensile strength of $A_\ell f_y$. This steel is in addition to the flexural reinforcement and is distributed uniformly around the sides of the perimeter so that the resultant of $A_\ell f_y$ acts along the axis of the member.

In a prestressed beam, the same technique (providing additional reinforcing bars with capacity $A_\ell f_y$) can be followed, or overstrength of the prestressing steel can be used to resist some of the axial force $A_\ell f_y$ as outlined in the next paragraph.

In a prestressed beam, the stress in the prestressing steel at nominal strength will be between f_{se} and f_{ps}. A portion of the $A_\ell f_y$ force can be resisted by a force of $A_{ps}\Delta f_{pt}$ in the prestressing steel, where Δf_{pt} is the difference between the stress that can be developed in the strand at the section under consideration and the stress required to resist the bending moment at this section, M_u. The stress required to resist the bending moment can be calculated as $M_u/(\phi 0.9 d_p A_{ps})$. For pretensioned strands, the stress which can be developed near the free end of the strand can be calculated using the procedure illustrated in Fig. R12.8.9. Note that near the ends of a pretensioned member the available stress in the prestressing steel will need to be reduced to account for lack of full development and should be determined in conjunction with 9.3.2.7.

R11.5.4 *Details of torsional reinforcement*

R11.5.4.1 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 (Mitchell and Collins 1976). This renders lapped-spliced stirrups ineffective, leading to a premature torsional failure (Behera 1969). In such cases, closed stirrups should not be made up of pairs of U-stirrups lapping one another.

R11.5.4.2 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. 11.5.4.2(a). In tests (Mitchell and Collins 1976), closed stirrups anchored by 90-degree hooks failed when this occurred. For this reason, 135-degree hooks or seismic hooks are preferable for torsional stirrups in all cases. In regions where this spalling is prevented by an adjacent slab or flange, 11.5.4.2(b) relaxes this and allows 90-degree hooks (refer to Fig. R11.5.4.2(b)).

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11.5.4.3 Longitudinal torsion reinforcement shall be developed at both ends.

11.5.4.4 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 $0.5A_{oh}/p_h$.

11.5.5 *Minimum torsion reinforcement*

11.5.5.1 A minimum area of torsional reinforcement shall be provided in all regions where T_u exceeds the threshold torsion given in 11.5.1.

11.5.5.2 Where torsional reinforcement is required by 11.5.5.1, the minimum area of transverse closed stirrups shall be computed by

$$(A_v + 2A_t) = 0.75\sqrt{f_c'}\frac{b_w s}{f_{yt}} \qquad (11\text{-}23)$$

but shall not be less than $(50b_w s)/f_{yt}$.

11.5.5.3 Where torsional reinforcement is required by 11.5.5.1, the minimum total area of longitudinal torsional reinforcement, $A_{\ell,min}$, shall be computed by

$$A_{\ell,min} = \frac{5\sqrt{f_c'}A_{cp}}{f_y} - \left(\frac{A_t}{s}\right)p_h\frac{f_{yt}}{f_y} \qquad (11\text{-}24)$$

where A_t/s shall not be taken less than $25b_w/f_{yt}$; f_{yt} refers to closed transverse torsional reinforcement; and f_y refers to longitudinal reinforcement.

11.5.6 *Spacing of torsion reinforcement*

11.5.6.1 The spacing of transverse torsion reinforcement shall not exceed the smaller of $p_h/8$ or 12 in.

11.5.6.2 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

COMMENTARY

R.11.5.4.3 If high torsion acts near the end of a beam, the longitudinal torsion reinforcement should be adequately anchored. Sufficient development length should 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.

R11.5.4.4 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.

R11.5.5 *Minimum torsion reinforcement*

R11.5.5.1 and R11.5.5.2 If a member is subject to a factored torsional moment T_u greater than the values specified in 11.5.1, the minimum amount of transverse web reinforcement for combined shear and torsion is $50b_w s/f_{yt}$. 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 whereas A_t is the area of only one leg of a closed stirrup.

Tests (Roller and Russell 1990) 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.

R11.5.5.3 Reinforced concrete beam specimens with less than 1 percent torsional reinforcement by volume have failed in pure torsion at torsional cracking (MacGregor and Ghoneim 1995). In ACI 318-89 and prior editions, 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 approximately 0.5 percent.

R11.5.6 *Spacing of torsion reinforcement*

R11.5.6.1 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 11.4.5.1.

R11.5.6.2 In R11.5.3.7, 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

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least one longitudinal bar or tendon in each corner of the stirrups. Longitudinal bars shall have a diameter at least 0.042 times the stirrup spacing, but not less than 3/8 in.

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.

11.5.6.3 Torsional reinforcement shall be provided for a distance of at least $(b_t + d)$ beyond the point required by analysis.

R11.5.6.3 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.

11.5.7 *Alternative design for torsion*

For torsion design of solid sections within the scope of this Code with an aspect ratio h/b_t of 3 or greater, it shall be permitted to use another procedure, the adequacy of which has been shown by analysis and substantial agreement with results of comprehensive tests. Sections 11.5.4 and 11.5.6 shall apply.

R11.5.7 *Alternative design for torsion*

Examples of such procedures are to be found in Zia and McGee (1974), Zia and Hsu (2004), and Collins and Mitchell (1980), which have been extensively and successfully used for design of precast, prestressed concrete beams with ledges. The procedure described in Zia and McGee (1974) and Zia and Hsu (2004) is an extension to prestressed concrete sections of the torsion procedures of pre-1995 editions of ACI 318. The seventh edition of the *PCI Design Handbook* (PCI 2014) describes the procedure of Zia and Hsu (2004) and Collins and Mitchell (1980). This procedure was experimentally verified by the tests described in Klein (1986).

11.6—Shear-friction

11.6.1 Provisions of 11.6 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.

R11.6—Shear-friction

R11.6.1 With the exception of 11.6, virtually all provisions regarding shear are intended to prevent diagonal tension failures rather than direct shear transfer failures. The purpose of 11.6 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 (refer to Birkeland and Birkeland [1966] and Mattock and Hawkins [1972]).

11.6.2 Design of cross sections subject to shear transfer as described in 11.6.1 shall be based on Eq. (11-1), where V_n is calculated in accordance with provisions of 11.6.3 or 11.6.4.

11.6.3 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 11.6.4 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests.

11.6.3.1 Provisions of 11.6.5 through 11.6.10 shall apply for all calculations of shear transfer strength.

R11.6.3 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 is to 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.

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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 11.6 depends on proper selection of the location of an assumed crack (PCI 2014; Birkeland and Birkeland 1966).

The relationship between shear-transfer strength and the reinforcement crossing the shear plane can be expressed in various ways. Equations (11-25) and (11-26) of 11.6.4 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 (PCI 2014; Mattock et al. 1976; Mattock 1974) can be used under the provisions of 11.6.3. For example, when the shear-friction reinforcement is perpendicular to the shear plane, the shear strength V_n is given by (Mattock et al. 1976; Mattock 1974)

$$V_n = 0.8\,A_{vf}f_y + A_cK_1$$

where A_c is the area of concrete section resisting shear transfer (in.^{2}), 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 11.6.9.

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 the resistance to shearing of protrusions on the crack faces, and 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

$$V_n = A_{vf}f_y(0.8\sin\alpha + \cos\alpha) + A_cK_1\sin^2\alpha$$

where α is the angle between the shear-friction reinforcement and the shear plane, (that is $0 < \alpha < 90$ degrees).

When using the modified shear-friction method, the terms ($A_{vf}f_y/A_c$) or ($A_{vf}f_y\sin\alpha/A_c$) should not be less than 200 psi for the design equations to be valid.

11.6.4 *Shear-friction design method*

11.6.4.1 When shear-friction reinforcement is perpendicular to shear plane, V_n shall be computed by

$$V_n = A_{vf}f_y\mu \tag{11-25}$$

where μ is coefficient of friction in accordance with 11.6.4.3.

11.6.4.2 When shear-friction reinforcement is inclined to the shear plane such that the shear force produces tension in shear-friction reinforcement, V_n shall be computed by

R11.6.4 *Shear-friction design method*

R11.6.4.1 The required area of shear-friction 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.

R11.6.4.2 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

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$$V_n = A_{vf}f_y(\mu\sin\alpha + \cos\alpha) \quad (11\text{-}26)$$

where **α** is angle between shear-friction reinforcement and shear plane.

11.6.4.3 The coefficient of friction **μ** in Eq. (11-25) and Eq. (11-26) shall be taken as:

Concrete placed monolithically: **1.4λ**.

Concrete placed against hardened concrete with surface intentionally roughened as specified in 11.6.9: **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 (refer to 11.6.10): **0.7λ**.

where **λ = 1.0** for normalweight concrete and 0.75 for all lightweight concrete. Otherwise, **λ** shall be determined based on volumetric proportions of lightweight and normalweight aggregates as specified in 8.6.1 but shall not exceed 0.85.

11.6.5 For normalweight concrete either placed monolithically or placed against hardened concrete with surface intentionally roughened as specified in 11.6.9, V_n shall not exceed the smallest of $\mathbf{0.2f_c'A_c}$, $\mathbf{(480 + 0.08f_c')A_c}$, and $\mathbf{1600A_c}$, where A_c is area of concrete section resisting shear

COMMENTARY

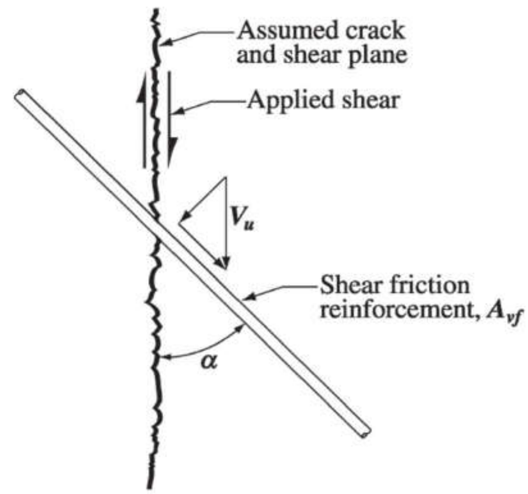

Fig. R11.6.4.2—Shear-friction reinforcement at an angle to assumed crack.

tension in the reinforcement, as shown in Fig. R11.6.4.2, part of the shear is resisted by the component parallel to the shear plane of the tension force in the reinforcement (Mattock 1974). Equation (11-26) should be used only when the shear force component parallel to the reinforcement produces tension in the reinforcement, as shown in Fig. R11.6.4.2. When **α** is greater than 90 degrees, the relative movement of the surfaces tends to compress the bar and Eq. (11-26) is not valid.

R11.6.4.3 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 concrete cast against hardened concrete not roughened in accordance with 11.6.9, shear resistance is primarily due to dowel action of the reinforcement and tests (Mattock 1977) indicate that reduced value of **μ = 0.6λ** specified for this case is appropriate.

The value of **μ** 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 AISC (1986).

R11.6.5 These upper limits on shear friction strength are necessary, as Eq. (11-25) and (11-26) may become unconservative for some cases. Test data (Kahn and Mitchell 2002; Mattock 2001) on normalweight concrete either placed monolithically or placed against hardened concrete

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transfer. For all other cases, V_n shall not exceed the smaller of $0.2f_c'A_c$ or $800A_c$. Where concretes of different strengths are cast against each other, the value of f_c' used to evaluate V_n shall be that of the lower-strength concrete.

11.6.6 The value of f_y used for design of shear-friction reinforcement shall not exceed 60,000 psi.

11.6.7 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 $A_{vf}f_y$, the force in the shear-friction reinforcement, when calculating required A_{vf}.

11.6.8 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop f_y on both sides by embedment, hooks, or welding to special devices.

COMMENTARY

with surface intentionally roughened as specified in 11.6.9 show that a higher upper limit can be used on shear friction strength for concrete with f_c' greater than 4000 psi than was allowed before the revisions in ACI 318-08. In higher-strength concretes, additional effort may be required to achieve the roughness specified in 11.6.9.

R11.6.7 If a resultant tensile force acts across a shear plane, reinforcement to carry that tension should 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 (Mattock et al. 1975).

It has also been demonstrated experimentally (Mattock and Hawkins 1972) 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 certain that the compressive force is permanent.

R11.6.8 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 should have full tensile anchorage on both sides of the shear plane. Further, the shear-friction reinforcement anchorage should 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, refer to PCI (2014).

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11.6.9 For the purpose of 11.6, 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.

11.6.10 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.

11.7—Deep beams

11.7.1 The provisions of 11.7 shall apply to members with ℓ_n not exceeding $4h$, the overall member depth or regions of beams with concentrated loads within a distance $2h$ from the support that are loaded on one face and supported on the opposite face so that compression struts can develop between the loads and the supports. Refer also to 12.8.10.6.

11.7.2 Deep beams shall be designed either by taking into account nonlinear distribution of strain or by Appendix B. In all cases, minimum distribution reinforcement shall be provided in accordance with 11.7.4

11.7.3 Deep beams shall be proportioned such that V_u is less than or equal to $\phi 10\sqrt{f_c'}\,b_w d$.

11.7.4 Total distributed reinforcement along the two side faces of deep beams shall not be less than that required in 11.7.4.1 and 11.7.4.2.

11.7.4.1 The area of shear reinforcement perpendicular to the longitudinal axis of the beam, A_v, shall not be less than $0.0025 b_w s$, and s shall not exceed the smaller of $d/5$, and 12 in.

11.7.4.2 The area of shear reinforcement parallel to the longitudinal axis of the beam, A_{vh}, shall not be less than $0.0025 b_w s_2$, and s_2 shall not exceed the smaller of $d/5$, and 12 in.

11.8—Provisions for brackets and corbels

COMMENTARY

R11.7—Deep beams

R11.7.1 The behavior of a deep beam is discussed in Schlaich et al. (1987), Rogowksy and MacGregor (1986), Marti (1985), and Crist (1966). For a deep beam supporting gravity loads, 11.7.1 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, strut-and-tie models as defined in Appendix B should be used to design reinforcement to suspend the loads within the beam and transfer them to adjacent supports.

The longitudinal reinforcement in deep beams should be extended to the supports and adequately anchored by embedment, hooks, headed deformed bars, or welding to special devices. Bent-up bars are not recommended.

R11.7.2 Deep beams can be designed using strut-and-tie models, regardless of how they are loaded and supported. Section 10.7.1 allows the use of analyses that take into account nonlinear strain and stress distributions when proportioning deep beams. Such analyses, including nonlinear finite element analyses, should consider the effects of cracking on the stress distribution.

R11.7.3 This limit is imposed to control cracking under service loads and guard against diagonal compression failures in deep beams.

R11.7.4 The amount of shear reinforcement required for strength should be proportioned to be consistent with the analysis method used. The minimum reinforcement requirements in 11.7.4.1 and 11.7.4.2 are to be used irrespective of the analysis method and are intended to control the width and propagation of inclined cracks. Tests (Rogowsky and MacGregor 1986; Marti 1985; Crist 1966) have shown that vertical shear reinforcement (perpendicular to the longitudinal axis of the member) is more effective for member strength than horizontal shear reinforcement (parallel to the longitudinal axis of the member) in a deep beam, but the specified minimum reinforcement in both directions is required to control the growth and width of diagonal cracks.

R11.8—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 11.2.

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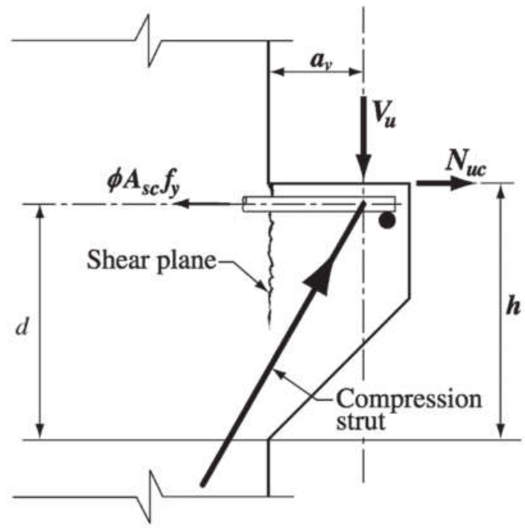

Fig. R11.8a—Structural action of a corbel.

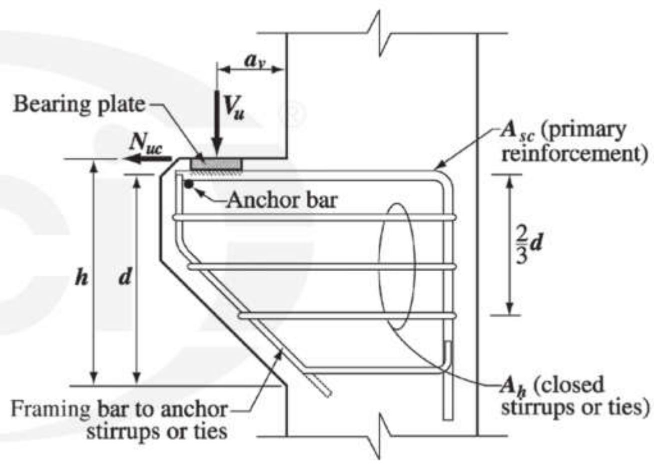

Fig. R11.8b—Notation used in Section 11.8.

The corbel shown in Fig. R11.8a 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 Joint ACI-ASCE Committee 426 (1974). The notation used in 11.8 is illustrated in Fig. R11.8b.

11.8.1 Brackets and corbels with a shear span-to-depth ratio a_v/d less than 2 shall be permitted to be designed using Appendix B. Design shall be permitted using 11.8.3 and 11.8.4 for brackets and corbels with:

(a) a_v/d not greater than 1, and

(b) subject to factored horizontal tensile force N_{uc} not larger than V_u

The requirements of 11.8.2, 11.8.3.2.1, 11.8.3.2.2, 11.8.5, 11.8.6, and 11.8.7 shall apply to design of brackets and corbels. Effective depth d shall be determined at the face of the support.

R11.8.1 An upper limit of 1.0 for a_v/d is imposed for design by 11.8.3 and 11.8.4 for two reasons. First, for a_v/d, 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 11.8.4 is not appropriate. Second, this method of design has only been validated experimentally for a_v/d of unity or less. An upper limit is provided 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.

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11.8.2 Depth at outside edge of bearing area shall not be less than $\mathbf{0.5}\boldsymbol{d}$.

11.8.3 Section at face of support shall be designed to resist simultaneously V_u, a factored moment $[V_u a_v + N_{uc}(h - d)]$, and a factored horizontal tensile force N_{uc}.

11.8.3.1 In all design calculations in accordance with 11.8, ϕ shall be taken equal to 0.75.

11.8.3.2 Design of shear-friction reinforcement A_{vf} to resist V_u shall be in accordance with 11.6.

11.8.3.2.1 For normalweight concrete, V_n shall not exceed the smallest of $0.2f_c'b_wd$, $(480 + 0.08f_c')b_wd$, and $1600b_wd$.

11.8.3.2.2 For all-lightweight or sand-lightweight concrete, V_n shall not be taken greater than the smaller of $(0.2 - 0.07a_v/d)f_c'b_wd$ and $(800 - 280a_v/d)b_wd$.

11.8.3.3 Reinforcement A_f to resist factored moment $[V_ua_v + N_{uc}(h - d)]$ shall be computed in accordance with 10.2 and 10.3.

11.8.3.4 Reinforcement A_n to resist factored tensile force N_{uc} shall be determined from $\phi A_nf_y \geq N_{uc}$. Factored tensile force N_{uc} shall not be taken less than $0.2V_u$ unless provisions are made to avoid tensile forces. N_{uc} shall be regarded as a live load even if tension results from restraint of creep, shrinkage, or temperature change.

11.8.3.5 Area of primary tension reinforcement A_{sc} shall not be less than the greater of $(A_f + A_n)$ and $(2A_{vf}/3 + A_n)$.

COMMENTARY

R11.8.2 A minimum depth is required 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 (Kriz and Raths 1965) in corbels having depths at the outside edge of the bearing area less than required in this section of the Code.

R11.8.3.1 Corbel and bracket behavior is predominantly controlled by shear; therefore, a single value of $\phi = 0.75$ is required for all design conditions.

R11.8.3.2.2 Tests (Mattock et al. 1976) have shown that the maximum shear strength of lightweight concrete corbels or brackets is a function of both f_c' and a_v/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.

R11.8.3.3 Reinforcement required to resist moment can be calculated using flexural theory. The factored moment is calculated by summing moments about the flexural reinforcement at the face of support.

R11.8.3.4 Because the magnitude of horizontal forces acting on corbels or brackets cannot usually be determined with great accuracy, it is required that N_{uc} be regarded as a live load.

R11.8.3.5 Tests (Mattock et al. 1976) suggest that the total amount of reinforcement $(A_{sc} + A_h)$ required to cross the face of support should be the greater of:

(a) The sum of A_{vf} calculated according to 11.8.3.2 and A_n calculated according to 11.8.3.4

(b) The sum of **1.5** times A_f calculated according to 11.8.3.3 and A_n calculated according to 11.8.3.4

If (a) controls, $A_{sc} = (2A_{vf}/3 + A_n)$ is required as primary tensile reinforcement, and the remaining $A_{vf}/3$ should be provided as closed stirrups parallel to A_{sc} and distributed within $2d/3$, adjacent to A_{sc}. Section 11.8.4 satisfies this by requiring $A_h = 0.5(2A_{vf}/3)$.

If (b) controls, $A_{sc} = (A_f + A_n)$ is required as primary tension reinforcement, and the remaining $A_f/2$ should be provided as

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closed stirrups parallel to A_{sc} and distributed within $2d/3$, adjacent to A_{sc}. Again, 11.8.4 satisfies this requirement.

11.8.4 Total area A_h of closed stirrups or ties parallel to primary tension reinforcement shall not be less than $0.5(A_{sc} - A_n)$. Distribute A_h uniformly within $(2/3)d$ adjacent to primary tension reinforcement.

R11.8.4 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 $A_h = 0.5(A_{sc} - A_n)$ automatically yields the appropriate amounts, as discussed in R11.8.3.5.

11.8.5 A_{sc}/bd shall not be less than $0.04(f_c'/f_y)$.

R11.8.5 A minimum amount of reinforcement is required 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}.

11.8.6 At front face of bracket or corbel, primary tension reinforcement 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 f_y of primary tension reinforcement

(b) By bending primary tension reinforcement back to form a horizontal loop, or

(c) By some other means of positive anchorage

R11.8.6 Because the horizontal component of the inclined concrete compression strut (refer to Fig. R11.8a) is transferred to the primary tension reinforcement at the location of the vertical load, the primary tension reinforcement 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 specified yield strength from the face of support to the vertical load. Satisfactory anchorage at the outer end can be obtained by bending the primary tension reinforcement 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 primary tension reinforcement bars. The welds should be designed to develop the yield strength of the primary tension reinforcement. The weld detail used

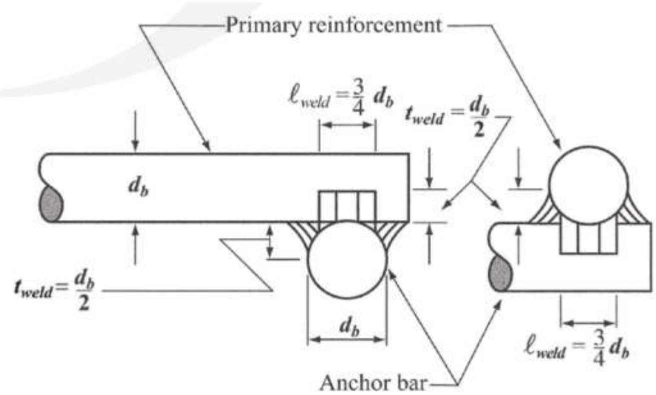

Fig. R11.8.6—Weld details used in tests of Mattock et al. (1976).

successfully in the corbel tests reported in Mattock et al. (1976) is shown in Fig. R11.8.6. The primary tension reinforcement should be anchored within the supporting column in accordance with the requirements of Chapter 12. Refer to additional discussion on end anchorage in R12.8.10.6.

11.8.7 Bearing area on bracket or corbel shall not project beyond straight portion of primary tension reinforcement, nor project beyond interior face of transverse anchor bar (if one is provided).

R11.8.7 The restriction on the location of the bearing area is necessary to ensure development of the specified yield strength of the primary tension reinforcement near the load. When corbels are designed to resist horizontal forces,

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the bearing plate should be welded to the primary tension reinforcement.

11.9—Provisions for walls

11.9.1 Design for shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in 11.11. Design for horizontal in-plane shear forces in a wall shall be in accordance with 11.9.2 through 11.9.9. Alternatively, it shall be permitted to design walls with a height not exceeding two times the length of the wall for horizontal shear forces in accordance with Appendix B and 11.9.9.2 through 11.9.9.5.

R11.9—Provisions for walls

R11.9.1 Shear in the plane of the wall is primarily of importance for shear walls 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.

11.9.2 Design of horizontal section for shear in plane of wall shall be based on Eq. (11-1) and (11-2), where V_c shall be in accordance with 11.9.5 or 11.9.6 and V_s shall be in accordance with 11.9.9.

11.9.3 V_n at any horizontal section for shear in plane of wall shall not be taken greater than $10\sqrt{f_c'}hd$, where h is thickness of wall, and d is defined in 11.9.4.

R11.9.3 Although the width-to-depth ratio of shear walls is less than that for ordinary beams, tests (Cardenas et al. 1973) on shear walls with a thickness equal to $\ell_w/25$ have indicated that ultimate shear stresses in excess of $10\sqrt{f_c'}$ can be obtained.

11.9.4 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.

11.9.5 Unless a more detailed calculation is made in accordance with 11.9.6, V_c shall not be taken greater than $2\lambda\sqrt{f_c'}hd$ for walls subject to axial compression, or V_c shall not be taken greater than the value given in 11.2.2.3 for walls subject to axial tension.

11.9.6 V_c shall be permitted to be the lessor of the values computed from Eq. (11-27) and (11-28)

$$V_c = 3.3\lambda\sqrt{f_c'}hd + \frac{N_u d}{4\ell_w} \tag{11-27}$$

or

$$V_c = \left[0.6\lambda\sqrt{f_c'} + \frac{\ell_w\left(1.25\lambda\sqrt{f_c'} + 0.2\dfrac{N_u}{\ell_w h}\right)}{\dfrac{M_u}{V_u} - \dfrac{\ell_w}{2}}\right]hd \tag{11-28}$$

where ℓ_w is the overall length of the wall, and N_u is positive for compression and negative for tension. If $(M_u/V_u - \ell_w/2)$ is negative, Eq. (11-28) shall not apply.

R11.9.5 and R11.9.6 Equations (11-27) and (11-28) may be used to determine the inclined cracking strength at any section through a shear wall. Equation (11-27) corresponds to the occurrence of a principal tensile stress of approximately $4\lambda\sqrt{f_c'}$ at the centroid of the shear wall cross section. Equation (11-28) corresponds approximately to the occurrence of a flexural tensile stress of $6\lambda\sqrt{f_c'}$ 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. (11-27) will control before this term becomes negative. When this term becomes negative, Eq. (11-27) should be used.

11.9.7 Sections located closer to wall base than a distance $\ell_w/2$ or one-half the wall height, whichever is less, shall be

R11.9.7 The values of V_c computed from Eq. (11-27) and (11-28) at a section located the lesser distance of $\ell_w/2$ and $h_w/2$ above the base apply to that and all sections between

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permitted to be designed for the same V_c as that computed at a distance $\ell_w/2$ or one-half the height.

11.9.8 Where V_u is less than $0.5\phi V_c$, reinforcement shall be provided in accordance with 11.9.9 or in accordance with Chapter 14. Where V_u exceeds $0.5\phi V_c$, wall reinforcement for resisting shear shall be provided in accordance with 11.9.9.

11.9.9 *Design of shear reinforcement for walls*

11.9.9.1 Where V_u exceeds ϕV_c, horizontal shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where V_s shall be computed by

$$V_s = \frac{A_v f_y d}{s} \quad (11\text{-}29)$$

where A_v is area of horizontal shear reinforcement within spacing s and d is determined in accordance with 11.9.4. Vertical shear reinforcement shall be provided in accordance with 11.9.9.4.

11.9.9.2 Ratio of horizontal shear reinforcement area to gross concrete area of vertical section, ρ_t, shall not be less than 0.0025.

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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 11.9.3.

R11.9.9 *Design of shear reinforcement for walls*

Both horizontal and vertical shear reinforcement are required for all walls. The notation used to identify the direction of the distributed shear reinforcement in walls was updated in ACI 318-05 to eliminate conflicts between the notation used for ordinary structural walls in Chapters 11 and 14 and the notation used for special structural walls in Chapter 21. The distributed reinforcement is now identified as being oriented parallel to either the longitudinal or transverse axis of the wall. Therefore, for vertical wall segments, the notation used to describe the horizontal distributed reinforcement ratio is ρ_t, and the notation used to describe the vertical distributed reinforcement ratio is ρ_ℓ.

For low walls, test data (Barda et al. 1977) 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. (11-30); if h_w/ℓ_w is less than 0.5, the amount of vertical reinforcement is equal to the amount of horizontal reinforcement. If h_w/ℓ_w is greater than 2.5, only a minimum amount of vertical reinforcement is required **(0.0025*sh*)**.

Equation (11-29) 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 should be provided in accordance with 11.9.9.4 within the spacing limitation of 11.9.9.5.

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11.9.9.3 Spacing of horizontal shear reinforcement shall not exceed the smallest of $\ell_w/5$, $3h$, and 12 in., where ℓ_w is the overall length of the wall.

11.9.9.4 Ratio of vertical shear reinforcement area to gross concrete area of horizontal section, ρ_ℓ, shall not be less than the larger of

$$\rho_\ell = 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_t - 0.0025) \tag{11-30}$$

and 0.0025. The value of ρ_ℓ calculated by Eq. (11-30) need not be greater than ρ_τ required by 11.9.9.1. In Eq. (11-30), ℓ_w is the overall length of the wall, and h_w is the overall height of the wall.

11.9.9.5 Spacing of vertical shear reinforcement shall not exceed the least of $\ell_w/3$, $3h$, and 12 in. where ℓ_w is the overall length of the wall.

11.10—Transfer of moments to columns

11.10.1 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 transverse reinforcement in the columns.

11.10.2 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 transverse 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. Refer also to 12.12.

11.11—Provisions for slabs and footings

11.11.1 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:

11.11.1.1 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 11.4.

11.11.1.2 For two-way action, 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

COMMENTARY

R11.9.9.3 Maximum spacing of 12 in. for reinforcement is required for crack control.

R11.9.9.5 Maximum spacing of 12 in. for reinforcement is required for crack control.

R11.10—Transfer of moments to columns

R11.10.1 Tests (Hanson and Conner 1967) 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. However, joints without lateral confinement, such as at the exterior of a building, need shear reinforcement to prevent deterioration due to shear cracking (ACI 352R).

For regions where strong earthquakes may occur, joints may be required to withstand several reversals of loading that develop the flexural strength of the adjoining beams. Refer to Chapter 13 for provisions for seismic design.

R11.11—Provisions for slabs and footings

R11.11.1 Differentiation should 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.

R11.11.1.2 The critical section for shear in slabs subjected to bending in two directions follows the perimeter at the edge of the loaded area (Joint ACI-ASCE Committee 326 1962). The shear stress acting on this section at factored loads is a

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(a) Edges or corners of columns, concentrated loads, or reaction areas, and

(b) Changes in slab thickness such as edges of capitals, drop panels, or shear caps

For two-way action, the slab or footing shall be designed in accordance with 11.11.2 through 11.11.6.

11.11.1.3 For square or rectangular columns, concentrated loads, or reaction areas, the critical sections with four straight sides shall be permitted.

11.11.2 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 11.11.2.1, 11.11.2.2, or 11.11.3.1. V_s shall be computed in accordance with 11.11.3. For slabs with shearheads, V_n shall be in accordance with 11.11.4. When moment is transferred between a slab and a column, 11.11.7 shall apply.

11.11.2.1 For nonprestressed slabs and footings, V_c shall be the smallest of (a), (b), and (c)

(a) $$V_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f_c'}b_o d \qquad (11\text{-}31)$$

where β is the ratio of long side to short side of the column, concentrated load or reaction area;

(b) $$V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f_c'}b_o d \qquad (11\text{-}32)$$

where α_s is 40 for interior columns, 30 for edge columns, 20 for corner columns, and

(c) $$V_c = 4\lambda\sqrt{f_c'}b_o d \qquad (11\text{-}33)$$

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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 11.11.1.3 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 such as the edge of drop panels or shear caps, 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.

R11.11.2.1 For square columns, the shear stress due to ultimate loads in slabs subjected to bending in two directions is limited to $4\lambda\sqrt{f_c'}$. However, tests (Joint ACI-ASCE Committee 426 1974) have indicated that the value of $4\lambda\sqrt{f_c'}$ is unconservative when the ratio β 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 approximately $4\lambda\sqrt{f_c'}$ around the corners of the column or loaded area, down to $2\lambda\sqrt{f_c'}$ or less along the long sides between the two end sections. Other tests (Vanderbilt 1972) indicate that v_c decreases as the ratio b_o/d increases. Equations (11-31) and (11-32) were developed to account for these two effects. The words "interior", "edge", and "corner columns" in 11.11.2.1(b) refer to critical sections with four, three, or two sides, respectively.

For shapes other than rectangular, β is taken to be the ratio of the longest overall dimension of the effective loaded area to the largest overall perpendicular dimension of the effective loaded area, as illustrated for an L-shaped reaction area in Fig. R11.11.2.1. The effective loaded area is that area totally enclosing the actual loaded area, for which the perimeter is a minimum.

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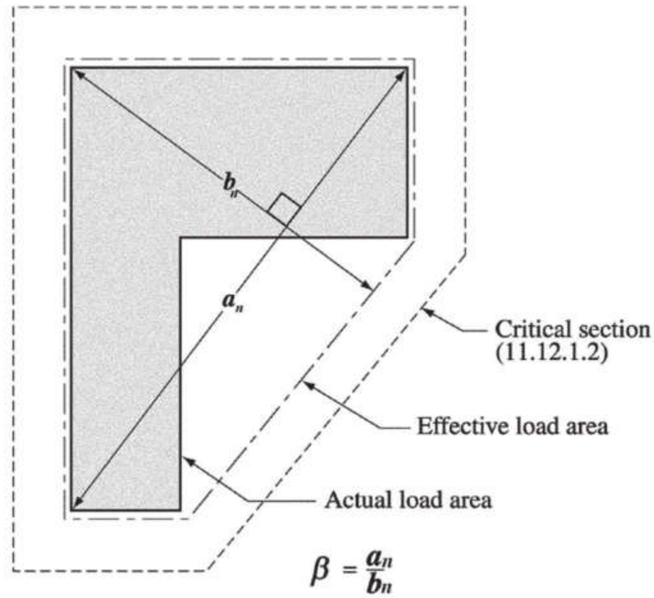

Fig. R11.11.2.1—Value of β *for a nonrectangular loaded area.*

11.11.2.2 At columns of two-way prestressed slabs and footings that meet the requirements of 19.9.3

$$V_c = (\beta_p \lambda \sqrt{f_c'} + 0.3 f_{pc}) b_o d + V_p \quad (11\text{-}34)$$

where β_p is the smaller of 3.5 and $(\alpha_s d/b_o + 1.5)$; 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 11.11.1.2; f_{pc} is taken as 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. (11-34) if the following are satisfied; otherwise, 11.11.2.1 shall apply:

(a) No portion of the column cross section shall be closer to a discontinuous edge than four times the slab thickness

(b) The value of $\sqrt{f_c'}$ used in Eq. (11-34) shall not be taken greater than 70 psi, and

(c) In each direction, f_{pc} shall not be less than 125 psi, nor be taken greater than 500 psi

R11.11.2.2 For prestressed slabs and footings, a modified form of Code Eq. (11-31) and (11-32) is specified for two-way action shear strength. Research (ACI 423.3R; Burns and Hemakom 1977) indicates that the shear strength of two-way prestressed slabs around interior columns is conservatively predicted by Eq. (11-34). V_c from Eq. (11-32) corresponds to a diagonal tension failure of the concrete initiating at the critical section defined in 11.11.1.2. 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. (11-31). Consequently, the term β does not enter into Eq. (11-34). Values for $\sqrt{f_c'}$ 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 shear walls and other structural elements should be taken into account.

In a prestressed slab with distributed tendons, the V_p term in Eq. (11-34) 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 should be noted.

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 b_o, the total perimeter of the critical section. Shear strength in this case is therefore conservatively taken the same as for a nonprestressed slab.

11.11.3 Shear reinforcement consisting of bars or wires and single- or multiple-leg stirrups shall be permitted in slabs and footings with d greater than or equal to 6 in., but not less than 16 times the shear reinforcement bar diameter. Shear reinforcement shall be in accordance with 11.11.3.1 through 11.11.3.4.

R11.11.3 Research (Hawkins 1974; Broms 1990; Yamada et al. 1991; Hawkins et al. 1975; ACI 421.1R) 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 limits given in 11.11.3.3 correspond to slab shear

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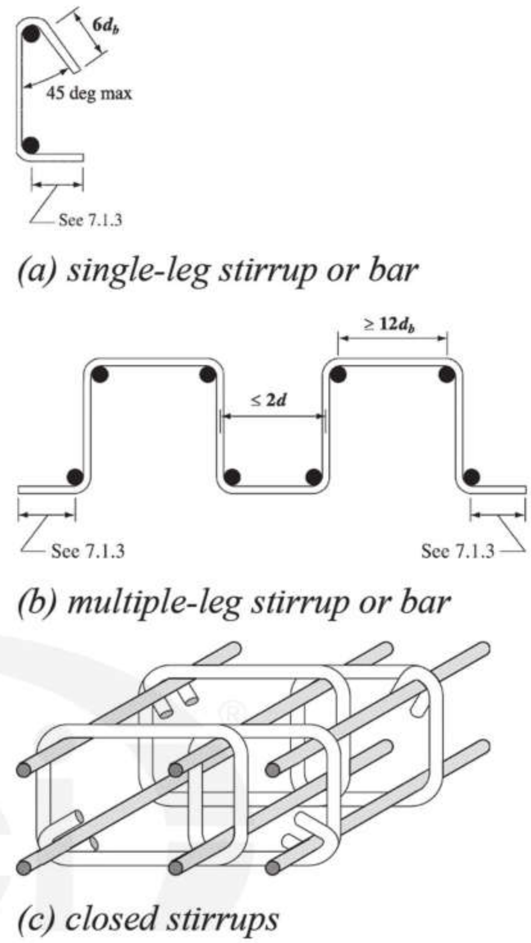

Fig. R11.11.3(a)-(c)—Single- or multiple-leg stirrup-type slab shear reinforcement.

11.11.3.1 V_n shall be computed by Eq. (11-2), where V_c shall not be taken greater than $2\lambda\sqrt{f_c'}\,b_o d$, and V_s shall be calculated in accordance with 11.4. In Eq. (11-15), A_v shall be taken as the cross-sectional area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the column section.

11.11.3.2 V_n shall not be taken greater than $6\sqrt{f_c'}\,b_o d$.

11.11.3.3 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.

11.11.3.4 Slab shear reinforcement shall satisfy the anchorage requirements of 12.8.13 and shall engage the longitudinal flexural reinforcement in the direction being considered.

reinforcement details that have been shown to be effective. Sections 12.8.13.2 and 12.8.13.3 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 typical details in Fig. R11.11.3(a) to (c). Anchorage of shear reinforcement according to the requirements of 12.8.13 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 (ACI 421.1R).

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. R11.11.3(d)). Spacing limits defined in 11.11.3.3 are also shown in Fig. R11.11.3(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. R11.11.3(e) are lower than on face AB, the closed stirrups extending from faces

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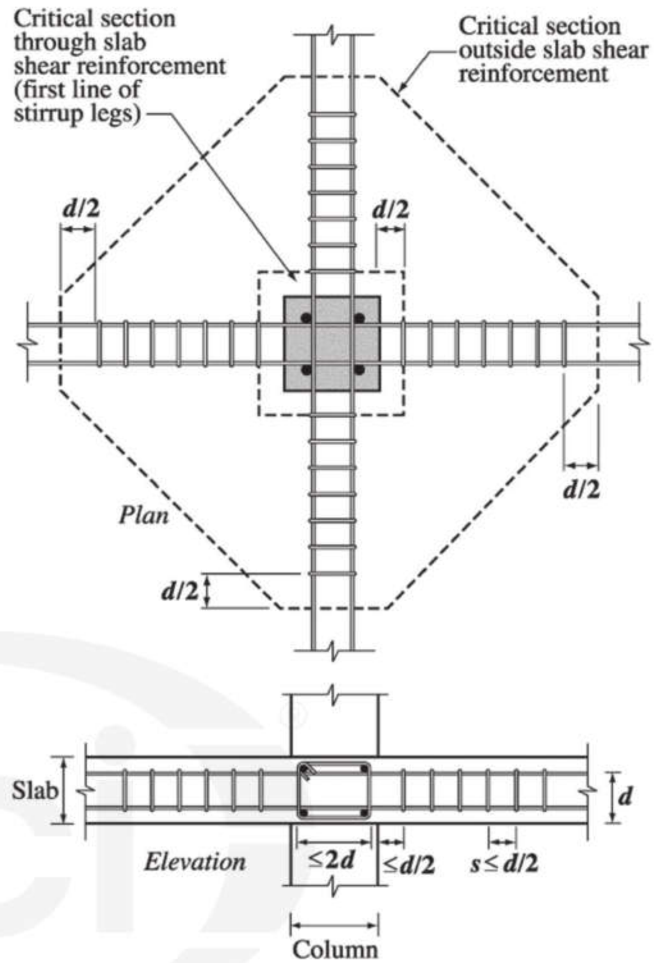

Fig. R11.11.3(d)—Arrangement of stirrup shear reinforcement, interior column.

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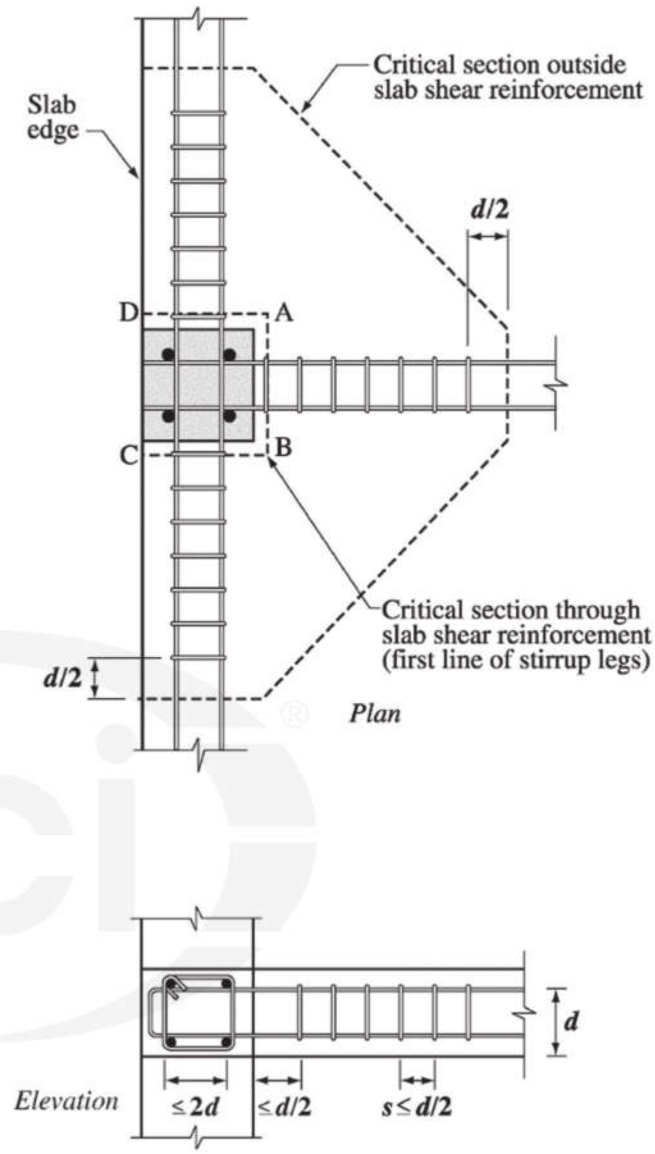

Fig. R11.11.3(e)—Arrangement of stirrup shear reinforcement, edge column.

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11.11.4 Shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) shall be permitted in slabs. The provisions of 11.11.4.1 through 11.11.4.9 shall apply where shear due to gravity load is transferred at interior column supports. Where moment is transferred to columns, 11.11.7.3 shall apply.

11.11.4.1 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.

11.11.4.2 A shearhead shall not be deeper than 70 times the web thickness of the steel shape.

11.11.4.3 The ends of each shearhead arm shall be permitted to be cut at angles not less than 30 degrees 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.

11.11.4.4 All compression flanges of steel shapes shall be located within $\mathbf{0.3}\boldsymbol{d}$ of compression surface of slab.

11.11.4.5 The ratio $\boldsymbol{\alpha_v}$ between the flexural stiffness of each shearhead arm and that of the surrounding composite cracked slab section of width $(\boldsymbol{c_2} + \boldsymbol{d})$ shall not be less than 0.15.

11.11.4.6 Plastic moment strength $\boldsymbol{M_p}$ required for each arm of the shearhead shall be computed by

$$M_p = \frac{V_u}{2\phi\eta}\left[h_v + \alpha_v\left(\ell_v - \frac{c_1}{2}\right)\right] \tag{11-35}$$

where $\boldsymbol{\phi}$ is for tension-controlled members; $\boldsymbol{\eta}$ is the number of shearhead arms; and $\boldsymbol{\ell_v}$ is the minimum length of each shearhead arm required to comply with requirements of 11.11.4.7 and 11.11.4.

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AD and *BC* provide some torsional strength along the edge of the slab.

R11.11.4 Based on reported test data (Corley and Hawkins 1968), 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 11.11.7.3.

Three basic criteria should be considered in the design of shearhead reinforcement for connections transferring shear due to gravity load. First, a minimum flexural strength should be provided to ensure 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 should be limited. Third, after these two requirements are satisfied, the negative moment slab reinforcement can be reduced in proportion to the moment contribution of the shearhead at the design section.

R11.11.4.5 and R11.11.4.6 The assumed idealized shear distribution along an arm of a shearhead at an interior column is shown in Fig. R11.11.4.5. The shear along each of the arms is taken as $\boldsymbol{\alpha_v\phi V_c/n}$, where $\boldsymbol{V_c}$ is definded in 11.11.2.1(c). However, the peak shear at the face of the column is taken as the total shear considered per arm $\boldsymbol{V_u/\eta}$ minus the shear considered carried to the column by the concrete compression zone of the slab. The latter term is expressed as $\boldsymbol{\phi(V_c/n)}$ $\boldsymbol{(1 - \alpha_v)}$, so that it approaches zero for a heavy shearhead and approaches $\boldsymbol{V_u/\eta}$ when a light shearhead is used. Equation (11-35) then follows from the assumption that $\boldsymbol{\phi V_c}$ is about

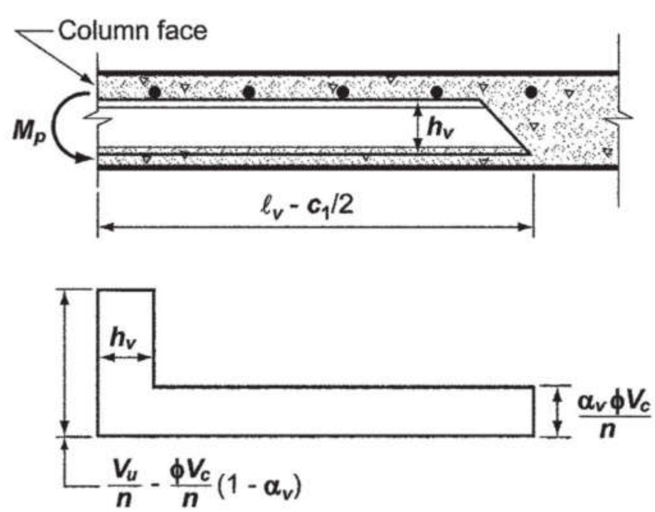

Fig. R11.11.4.5—Idealized shear acting on shearhead.

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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 ensure that 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.

11.11.4.7 The critical slab section for shear shall be perpendicular to the plane of the slab and shall cross each shearhead arm at three-fourths 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 11.11.1.2(a).

R11.11.4.7 The test results (Corley and Hawkins 1968) indicated that slabs containing under-reinforcing 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 over-reinforcing 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 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. Refer to Fig. R11.11.4.7.

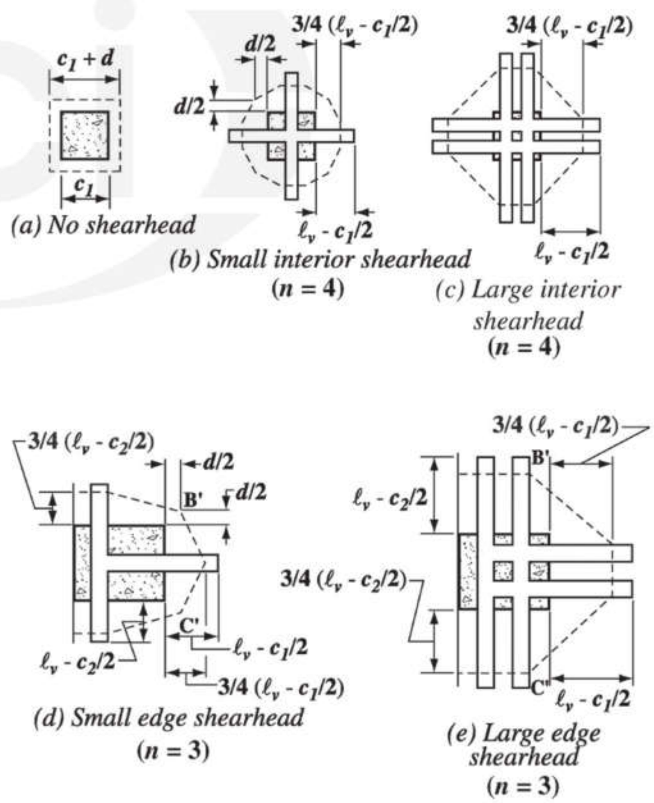

Fig. R11.11.4.7—Location of critical section defined in 11.11.4.7.

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COMMENTARY

11.11.4.8 V_n shall not be taken greater than $4\sqrt{f_c'}\,b_o d$ on the critical section defined in 11.11.4.7. When shearhead reinforcement is provided, V_n shall not be taken greater than $7\sqrt{f_c'}\,b_o d$ on the critical section defined in 11.11.1.2(a).

11.11.4.9 Moment resistance M_v contributed to each slab column strip by a shearhead shall not be taken greater than

$$M_v = \frac{\phi\alpha_v V_u}{2\eta}\left(\ell_v - \frac{c_1}{2}\right) \quad (11\text{-}36)$$

where ϕ is for tension-controlled members; η is the number of arms; and ℓ_v is the length of each shearhead arm provided. However, M_v shall not be taken larger than the smallest 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) M_p computed by Eq. (11-37)

R11.11.4.9 If the peak shear at the face of the column is neglected, and ϕV_c is again assumed to be about one-half of V_u, the moment resistance contribution of the shearhead M_v can be conservatively computed from Eq. (11-36), in which ϕ is the factor for flexure.

11.11.4.10 When unbalanced moments are considered, the shearhead shall have adequate anchorage to transmit M_p to column.

R11.11.4.10 Refer to R11.11.7.3.

11.11.5 Headed shear stud reinforcement, placed perpendicular to the plane of a slab or footing, shall be permitted

R11.11.5 Headed shear stud reinforcement was introduced in the ACI 318-08. Using headed stud assemblies, as shear

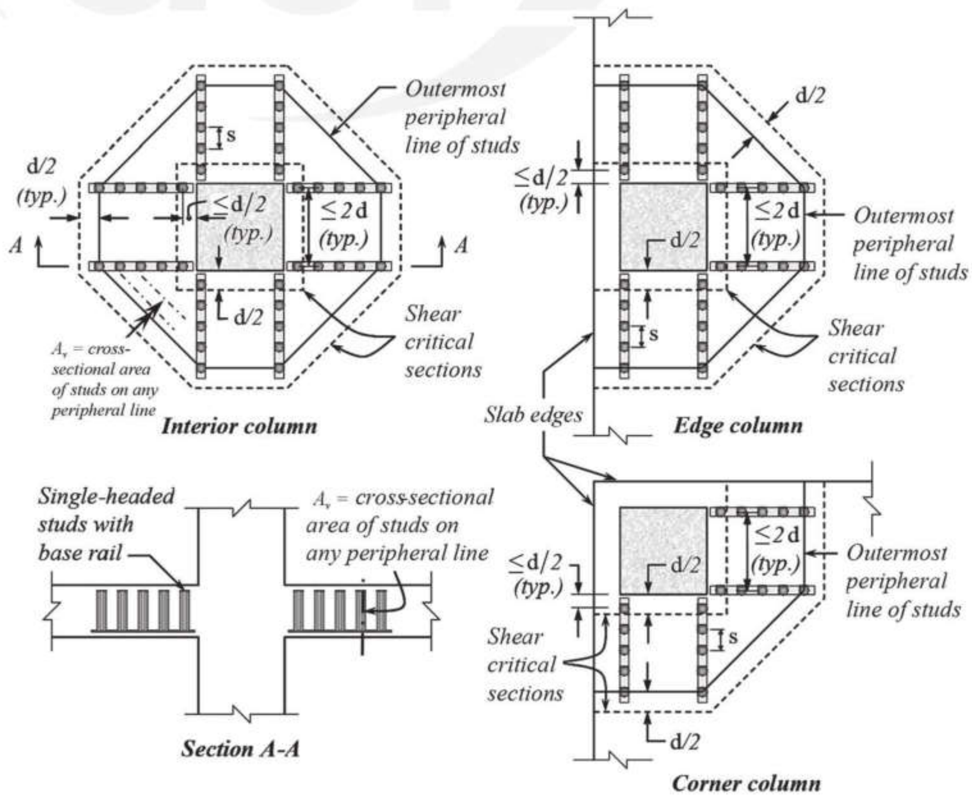

Fig. R11.11.5—Typical arrangements of headed shear stud reinforcement and critical sections outside the shear-reinforced zone.

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in slabs and footings in accordance with 11.11.5.1 through 11.11.5.4. The overall height of the shear stud assembly shall not be less than the thickness of the member less the sum of: 1) the concrete cover on the top flexural reinforcement; 2) the concrete cover on the base rail; and 3) one-half the bar diameter of the tension flexural reinforcement. Where flexural tension reinforcement is at the bottom of the section, as in a footing, the overall height of the shear stud assembly shall not be less than the thickness of the member less the sum of: 1) the concrete cover on the bottom flexural reinforcement; 2) the concrete cover on the head of the stud; and 3) one-half the bar diameter of the bottom flexural reinforcement.

11.11.5.1 For the critical section defined in 11.11.1.2, V_n shall be computed using Eq. (11-2), with V_c and V_n not exceeding $3\sqrt{f_c'}\,b_o d$ and $8\sqrt{f_c'}\,b_o d$, respectively. V_s shall be calculated using Eq. (11-15) with A_v equal to the cross-sectional area of all the shear reinforcement on one peripheral line that is approximately parallel to the perimeter of the column section, where s is the spacing of the peripheral lines of headed shear stud reinforcement. $A_v f_y/(b_o s)$ shall not be less than $2\sqrt{f_c'}$.

11.11.5.2 The spacing between the column face and the first peripheral line of shear reinforcement shall not exceed $d/2$. The spacing between peripheral lines of shear reinforcement, measured in a direction perpendicular to any face of the column, shall be constant. For prestressed slabs or footings satisfying 11.11.2.2, this spacing shall not exceed $0.75d$; for all other slabs and footings, the spacing shall be based on the value of the shear stress due to factored shear force and unbalanced moment at the critical section defined in 11.11.1.2, and shall not exceed:

(a) $0.75d$ where maximum shear stresses due to factored loads are less than or equal to $6\phi\sqrt{f_c'}$, and

(b) $0.5d$ where maximum shear stresses due to factored loads are greater than $6\phi\sqrt{f_c'}$

11.11.5.3 The spacing between adjacent shear reinforcement elements, measured on the perimeter of the first peripheral line of shear reinforcement, shall not exceed $2d$.

11.11.5.4 Shear stress due to factored shear force and moment shall not exceed $2\phi\sqrt{f_c'}$ at the critical section located $d/2$ outside the outermost peripheral line of shear reinforcement.

11.11.6 *Openings in slabs*

COMMENTARY

reinforcement in slabs and footings, requires specifying the stud shank diameter, the spacing of the studs, and the height of the assemblies for the particular applications.

Tests (Hawkins et al. 1975) show that vertical studs mechanically anchored as close as possible to the top and bottom of slabs are effective in resisting punching shear. The bounds of the overall specified height achieve this objective while providing a reasonable tolerance in specifying that height as shown in Fig. R12.7.5.

Compared with a leg of a stirrup having bends at the ends, a stud head exhibits smaller slip and, thus, results in smaller shear crack widths. The improved performance results in larger limits for shear strength and spacing between peripheral lines of headed shear stud reinforcement. Typical arrangements of headed shear stud reinforcement are shown in Fig. R11.11.5. The critical section beyond the shear reinforcement generally has a polygonal shape. Equations for calculating shear stresses on such sections are given in Hawkins et al. (1975).

R11.11.5.1 When there is unbalanced moment transfer, the design will be based on stresses. The maximum shear stress due to a combination of V_u and the fraction of unbalanced moment $\gamma_v M_u$ should not exceed ϕv_n, where v_n is taken as the sum of $3\lambda\sqrt{f_c'}$ and $A_v f_{yt}/(b_o s)$.

R11.11.5.2 The specified spacings between peripheral lines of shear reinforcement are justified by experiments (Hawkins et al. 1975). The clear spacing between the heads of the studs should be adequate to permit placing of the flexural reinforcement.

R11.11.6 *Openings in slabs*

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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 14, the critical slab sections for shear defined in 11.11.1.2 and 11.11.4.7 shall be modified as follows:

11.11.6.1 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.

11.11.6.2 For slabs with shearheads, the ineffective portion of the perimeter shall be one-half of that defined in 11.11.6.1.

COMMENTARY

Provisions for design of openings in slabs (and footings) were developed in Joint ACI-ASCE Committee 326 (1962). The locations of the effective portions of the critical section near typical openings and free edges are shown by the dashed lines in Fig. R11.11.6. Additional research (Joint ACI-ASCE Committee 426 1974) has confirmed that these provisions are conservative.

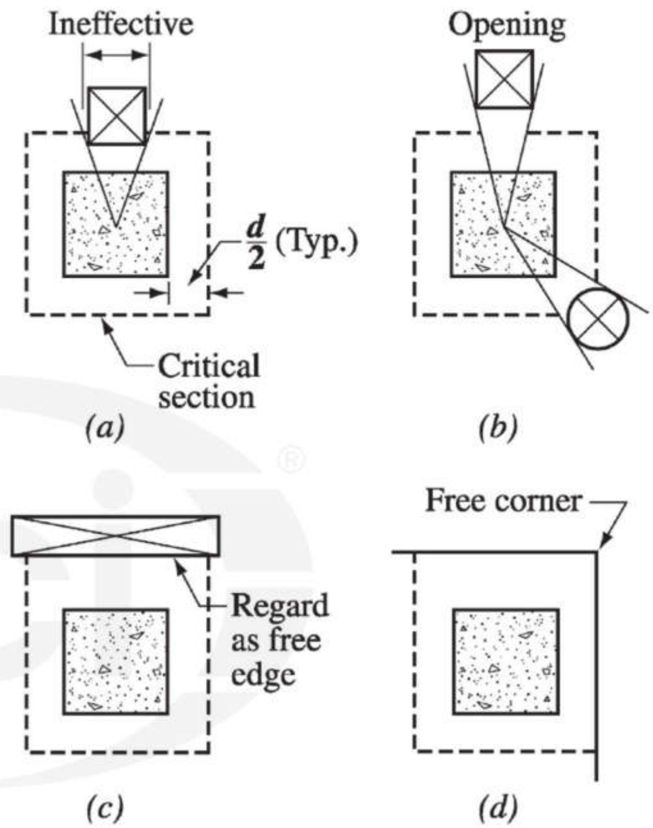

Fig. R11.11.6—Effect of openings and free edges (effective perimeter shown with dashed lines).

11.11.7 *Transfer of moment in slab-column connections*

11.11.7.1 Where gravity load, wind, earthquake, or other lateral forces cause transfer of unbalanced moment M_u between a slab and a column, $\gamma_f M_u$ shall be transferred by flexure in accordance with 14.5.3. The remainder of the unbalanced moment given by $\gamma_v M_u$ shall be considered transferred by eccentricity of shear about the centroid of the critical section defined in 11.11.1.2, where

$$\gamma_v = (1 - \gamma_f) \qquad (11\text{-}37)$$

R11.11.7.1 In Hanson and Hanson (1968), 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 11.11.1.2, and 40 percent by eccentricity of the shear about the centroid of the critical section. For rectangular columns, 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. (14-1).

Most of the data in Hanson and Hanson (1968) were obtained from tests of square columns, and little information is available for round columns. These can be approximated as square columns. Figure R14.6.2.5 shows square supports having the same area as some nonrectangular members.

11.11.7.2 The shear stress resulting from moment transfer by eccentricity of shear shall be assumed to vary linearly

R11.11.7.2 The stress distribution is assumed as illustrated in Fig. R11.11.7.2 for an interior or exterior column.

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about the centroid of the critical sections defined in 11.11.1.2. The maximum shear stress due to $\boldsymbol{V_u}$ and $\boldsymbol{M_u}$ shall not exceed $\boldsymbol{\phi v_n}$:

(a) For members without shear reinforcement

$$\phi v_n = \phi V_c/(b_o d) \qquad (11\text{-}38)$$

where $\boldsymbol{V_c}$ is as defined in 11.11.2.1 or 11.11.2.2.

(b) For members with shear reinforcement other than shearheads

$$\phi v_n = \phi(V_c + V_s)/(b_o d) \qquad (11\text{-}39)$$

where $\boldsymbol{V_c}$ and $\boldsymbol{V_s}$ are defined in 11.11.3.1. 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 $\boldsymbol{\phi(2\lambda\sqrt{f_c'})}$ at the critical section located $\boldsymbol{d/2}$ outside the outermost line of stirrup legs that surround the column.

COMMENTARY

The perimeter of the critical section, *ABCD*, is determined in accordance with 11.11.1.2. The factored shear force $\boldsymbol{V_u}$ and unbalanced factored moment $\boldsymbol{M_u}$ 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

$$v_{u(CD)} = \frac{V_u}{A_c} - \frac{\gamma_v M_u c_{CD}}{J_c}$$

where γ_v is given by Eq. (11-37). For an interior column, $\boldsymbol{A_c}$ and $\boldsymbol{J_c}$ may be calculated by: $\boldsymbol{A_c}$ is area of concrete of assumed critical section; $= \boldsymbol{2d(c_1 + c_2 + 2d)}$; $\boldsymbol{J_c}$ is 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 $\boldsymbol{A_c}$ and $\boldsymbol{J_c}$ 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 should be transferred by flexure in accordance with 14.5.3. A conservative method assigns the fraction transferred by flexure over an effective slab width defined in 14.5.3.2. Often, column strip reinforcement is concentrated near the column to accommodate this unbalanced moment. Available test data (Hanson and Hanson 1968) seem to indicate that this prac-

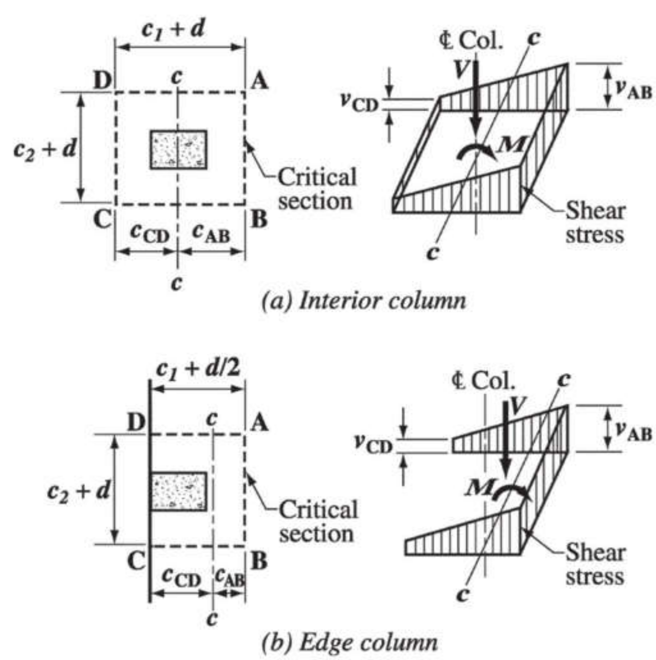

Fig. R11.11.7.2—Assumed distribution of shear stress.

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tice does not increase shear strength but may be desirable to increase the stiffness of the slab-column junction.

Test data (Hawkins 1981) indicate that the moment transfer strength of a prestressed slab to column connection can be calculated using the procedures of 11.11.7 and 14.5.3.

Where shear reinforcement has been used, the critical section beyond the shear reinforcement generally has a polygonal shape (Fig. R11.11.3(d) and (e)). Equations for calculating shear stresses on such sections are given in ACI 421.1R.

11.11.7.3 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 11.11.4.7 and the shear stresses resulting from moment transferred by eccentricity of shear about the centroid of the critical section defined in 11.11.1.2(a) and 11.11.1.3 shall not exceed $\phi 4\lambda \sqrt{f_c'}$.

R11.11.7.3 Tests (Hawkins and Corley 1974) indicate that the critical section defined in 11.11.1.2(a) and 11.11.1.3 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 11.11.4.10 requires the moment M_p to be transferred to the column in shearhead connections transferring unbalanced moments. This may be done by bearing within the column or by mechanical anchorage.

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CHAPTER 12—REINFORCEMENT—DETAILS, DEVELOPMENT, AND SPLICES

12.1—Standard hooks

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

12.1.1 180-degree bend plus $4d_b$ extension, but not less than 2-1/2 in. at free end of bar.

12.1.2 90-degree bend plus $12d_b$ extension at free end of bar.

12.1.3 For stirrup and tie hooks*
(a) No. 5 bar and smaller, 90-degree bend plus $6d_b$ extension at free end of bar; or
(b) No. 6, No. 7, and No. 8 bar, 90-degree bend plus $12d_b$ extension at free end of bar; or
(c) No. 8 bar and smaller, 135-degree bend plus $6d_b$ extension at free end of bar.

12.1.4 Seismic hooks as defined in 2.2.

12.2—Minimum bend diameters

12.2.1 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 12.2.

12.2.2 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 12.2.

Table 12.2—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	$10d_b$

*For closed ties and continuously wound ties defined as hoops in Chapter 13, a 135-degree bend plus an extension of at least 6db, but not less than 3 in. (Refer to definition of "hoop" in 13.1.)

COMMENTARY

CHAPTER R12—REINFORCEMENT—DETAILS, DEVELOPMENT, AND SPLICES

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* (ACI SP-66(04)), reported by ACI Committee 315.

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.

R12.1—Standard hooks

R12.1.3 Standard stirrup and tie hooks are limited to No. 8 bars and smaller, and the 90-degree 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-degree hooks and $6d_b$ extensions tend to "pop out" under high load.

R12.2—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.

R12.2.2 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- or 135-degree standard stirrup hook will permit multiple bending on standard stirrup bending equipment.

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12.2.3 Inside diameter of bend in welded wire reinforcement 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.

12.3—Bending

12.3.1 All reinforcement shall be bent cold, unless otherwise permitted by the licensed design professional.

12.3.2 Reinforcement partially embedded in concrete shall not be field bent, except as shown on the contract documents or permitted by the licensed design professional.

12.4—Surface conditions of reinforcement

12.4.1 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 3.5.3.8 and 3.5.3.9 shall be permitted.

12.4.2 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.

12.4.3 Prestressing steel shall be clean and free of oil, dirt, scale, pitting, and excessive rust. A light coating of rust shall be permitted.

COMMENTARY

R12.2.3 Welded wire reinforcement 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 A1064).

R12.3—Bending

R12.3.1 For unusual bends with inside diameters less than ASTM bend test requirements, special fabrication may be required.

R12.3.2 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 licensed design professional. Contract documents should specify whether the bars will be permitted to be bent cold or if heating should be used. Bends should be gradual and should be straightened as required.

Tests (Black 1973; Stecich et al. 1984) have shown that A615 Grade 40 and Grade 60 reinforcing bars can be cold bent and straightened up to 90 degrees at or near the minimum diameter specified in 12.2. 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.

R12.4—Surface conditions of reinforcement

Specific limits on rust are based on tests (Kemp et al. 1968) plus a review of earlier tests and recommendations. Kemp et al. (1968) 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.

R12.4.3 Guidance for evaluating the degree of rusting on strand is given in Sason (1992).

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12.5—Placing reinforcement

12.5.1 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 12.5.2.

12.5.2 Unless otherwise specified by the licensed design professional, reinforcement, including tendons, and post-tensioning ducts shall be placed within the tolerances in 12.5.2.1, 12.5.2.2, and 12.5.2.3.

12.5.2.1 Tolerances for $\boldsymbol{d}$ and for concrete cover in flexural members, walls, and compression members shall be as follows:

	Tolerance on d	Tolerance on specified 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. In addition, tolerance for cover shall also not exceed minus 1/3 the concrete cover specified in the contract documents.

12.5.2.2 Tolerance for depth $\boldsymbol{d}$, and minimum concrete cover for domes and membrane slabs that meet the requirements of Chapter 19 and Chapter 22, respectively:

	Tolerance on d	Tolerance on minimum concrete cover
$d \leq 6$ in.	±1/4 in.	–1/4 in.

COMMENTARY

R12.5—Placing reinforcement

R12.5.1 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.

R12.5.2 Generally accepted practice, as reflected in ACI 117, has established tolerances on total depth (formwork or finish) and fabrication of truss bent reinforcing bars and closed ties, stirrups, and spirals. The licensed design professional 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 licensed design professional or owner. In such cases, the licensed design professional should specify the necessary tolerances. Recommendations are given in PCI (2004).

R12.5.2.1 The Code permits a reinforcement placement tolerance on effective depth $\boldsymbol{d}$, which is directly related to the flexural and shear strength of the member. Because reinforcing steel is placed with respect to edges of members and formwork surfaces, $\boldsymbol{d}$ is not always conveniently measured in the field. Cover tolerances are also provided. For guidance on including field tolerances in project specifications, refer to ACI 117.

R12.5.2.2 The tolerances provided herein have been used for membrane slabs and spherical domes with prestressed dome rings in circular prestressed storage tanks. Measures are taken to achieve tighter tolerances in these thin elements. For thin shell domes, maximum spacings of reinforcement supports of 36 in. for slab bolsters and 24 in. for individual chairs are typical. For membrane slabs, the maximum spacing of reinforcement supports is typically 36 in. These maximum spacings of reinforcement supports for thin shell

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COMMENTARY

domes and membrane slabs have been effective in achieving the tolerances listed in 12.5.2.2.

12.5.2.3 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 12.5.2.1 shall also apply at discontinuous ends of members.

12.5.3 Welded wire reinforcement (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.

12.5.4 Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the licensed design professional.

R12.5.4 "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 reinforcement.

12.6—Spacing limits for reinforcement

12.6.1 The minimum clear spacing between parallel bars in a layer shall be $\boldsymbol{d_b}$, but not less than 1 in. Refer also to 3.3.2.

12.6.2 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.

12.6.3 In spirally reinforced or tied reinforced compression members, clear distance between longitudinal bars shall be not less than $\boldsymbol{1.5d_b}$ nor less than 1-1/2 in. Refer also to 3.3.2.

12.6.4 Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.

12.6.5 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.

12.6.6 *Bundled bars*

12.6.6.1 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four in any one bundle.

12.6.6.2 Bundled bars shall be enclosed within stirrups or ties.

R12.6—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.

R12.6.5 A limitation has been placed on the maximum spacing of reinforcement for crack control in liquid-retaining structures.

R12.6.6 *Bundled bars*

Bond research (ACI Committee 408 1966) 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 building size members. (AASHTO's "Standard Specifica-

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12.6.6.3 Bars larger than No. 11 shall not be bundled in beams.

12.6.6.4 Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least $40d_b$ stagger.

12.6.6.5 Where spacing limitations or concrete cover requirements 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.

12.6.7 *Tendons and ducts*

12.6.7.1 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 specified compressive strength of concrete at time of initial 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. Refer also to 3.3.2. Closer vertical spacing and bundling of tendons shall be permitted in the middle portion of a span.

12.6.7.2 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.

12.7—Concrete protection for reinforcement

12.7.1 *Cast-in-place concrete (nonprestressed)*

Unless a greater concrete cover is required by 12.7.7, specified cover for reinforcement shall not be less than the following:

Concrete cover

(a) Concrete cast against and permanently exposed to earth: 3 in.

COMMENTARY

tions for Highway Bridges" 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.

R12.6.7 *Tendons and ducts*

R12.6.7.1 The allowed decreased spacing in this section for transfer strengths of 4000 psi or greater is based on Deatherage et al. (1994) and Russell and Burns (1996).

R12.6.7.2 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.

R12.7—Concrete protection for reinforcement

The concrete cover requirements listed for protection of reinforcement against weather and other effects are based on the nominal bar diameter, not considering the effects of deformations. Where concrete cover is prescribed for a class of structural members, 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; or to the outermost part of the head on headed bars.

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 liquid contents in closed structures (openings in slab less than 25 percent

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COMMENTARY

of total area) are considered 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.

(b) Concrete exposed to earth, liquid, weather, or bearing on work mat or slabs supporting earth cover:

Slabs and joists: 2 in.

Beams and columns

Stirrups, spirals, and ties: 2 in.

Primary reinforcement: 2-1/2 in.

Walls: 2 in.

Footings and base slabs

Formed surfaces: 2 in.

Top of footings and base slabs: 2 in.

Shells, folded plate members: 1-1/2 in.

Controlled slabs meeting the requirements of Chapter 22

Bottom cover where cast against and permanently exposed to subgrade or cast against plastic sheeting: 2 in.

Top cover: 2 in.

Membrane slabs meeting the requirements of Chapter 22

Bottom cover where cast against and permanently exposed to plastic sheeting: 1-1/2 in.

Top cover: 1-1/4 in.

(c) Conditions not covered in 12.7.1(a) and (b):

Slabs and joists

No. 11 bars and smaller: 3/4 in.

No. 14 and No. 18 bars: 1-1/2 in.

Beams and columns

Stirrups, spirals, and ties: 1-1/2 in.

Primary reinforcement: 2 in.

Walls

No. 11 bars and smaller: 3/4 in.

No. 14 and No. 18 bars: 1-1/2 in.

Shells, folded plate members

No. 5 bars, W31 or D31 wire and smaller: 1/2 in.

No. 6 bars and larger: 3/4 in.

R12.7.1(b) A work mat prevents intermixing of the concrete with subgrade materials and provides a stable base for placement of reinforcement supports. A work mat may consist of a lean concrete slab or a base material meeting the requirements of Chapter 22.

Membrane slabs are thin, flexible concrete elements used for liquid containment on storage tanks. The lower required cover concrete for these types of slabs needs an increased level of quality.

12.7.2 *Precast concrete (manufactured under plant control conditions)*

Unless a greater concrete cover is required by 12.7.7, specified cover for prestressed and nonprestressed reinforcement, ducts, and end fittings shall not be less than the following:

R12.7.2 *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 specifi-

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Concrete cover
(a) Concrete exposed to earth, liquid, weather, or bearing on work mat, or slabs supporting earth cover:
Slabs and joists: 1-1/2 in.
Beams and columns
Stirrups, spirals, and ties: 1-1/2 in.
Primary reinforcement: 2 in.
Walls: 1-1/2 in.
Shells, folded plate members: 1 in.
(b) Conditions not covered in 12.7.2(a):
Slabs and joists
No. 11 bars, prestressing tendons 1-1/2 in. diameter and smaller: 3/4 in.
No. 14 and No. 18 bars, prestressing tendons larger than 1-1/2 in. diameter: 1-1/2 in.
Beams and columns
Stirrups, spirals, and ties: 1 in.
Primary reinforcement: 1-1/2 in.
Walls
No. 11 bars, prestressing tendons 1-1/2 in. diameter and smaller: 3/4 in.
No. 14 and No. 18 bars, prestressing tendons larger than 1-1/2 in. diameter: 1-1/2 in.
Shells, folded plate members
No. 5 bars, W31 or D31 wire and smaller: 3/4 in.
No. 6 bars and larger: 1 in.

12.7.3 *Cast-in-place concrete (prestressed)*

12.7.3.1 Unless a greater concrete cover is required by 12.7.7, specified cover for prestressed and nonprestressed reinforcement, ducts, and end fittings shall not be less than the following:

Concrete cover
(a) Concrete cast against and permanently exposed to earth: 3 in.
(b) Concrete exposed to earth, liquid, weather, or bearing on work mat, or slabs supporting earth cover:
Slabs and joists: 1-1/2 in.
Beams and columns
Stirrups, spirals, and ties: 1-1/2 in.
Primary reinforcement: 2 in.
Walls: 1-1/2 in.
Shells, folded plate members: 1 in.
Controlled slabs meeting the requirements of Chapter 22
Bottom cover where cast against and permanently exposed to subgrade: 2 in.
Bottom cover where cast against and permanently exposed to plastic sheeting: 1-3/4 in.
Top cover: 1-1/2 in.
Membrane slabs meeting the requirements of Chapter 22
Bottom cover where cast against and permanently exposed to plastic sheeting: 1-1/2 in.
Top cover: 1-1/4 in.

COMMENTARY

cally 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.

R12.7.3 *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. Spherical thin shell domes with integral prestressed ring beams meeting the requirements of Chapter 19, including the serviceability requirements of 19.4, are considered prestressed elements.

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(c) Conditions not covered in 12.7.3.1(a) and (b):

Slabs and joists
No. 11 bars and smaller: 3/4 in.
No. 14 and No. 18 bars: 1-1/2 in.

Beams and columns
Stirrups, spirals, and ties: 1 in.
Primary reinforcement: 1-1/2 in.

Walls
No. 11 bars and smaller: 3/4 in.
No. 14 and No. 18 bars: 1-1/2 in.

Shells, folded plate members
No. 5 bars, W31 and D31 wire and smaller: 3/4 in.
No. 6 bars and larger: 1 in.

12.7.3.2 For prestressed concrete members exposed to earth, weather, or corrosive environments, and in which permissible tensile stress of 19.4.2(c) is exceeded, minimum cover shall be increased 50 percent.

12.7.3.3 For prestressed concrete members manufactured under plant control conditions, minimum concrete cover for nonprestressed reinforcement shall be as required in 12.7.2.

12.7.4 *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.

12.7.5 *Headed shear stud reinforcement*

For headed shear stud reinforcement, specified concrete cover for the heads or base rails shall not be less than that required for the reinforcement in the type of member in which the headed shear stud reinforcement is placed.

COMMENTARY

R12.7.5 *Headed shear stud reinforcement*

The shanks, the heads, and the base rails need to be protected by the specified concrete cover. For efficiency in controlling inclined shear cracks, the overall height of the headed stud assembly should be as great as permissible (R11.11.5). The maximum overall height of the headed stud assembly is equal to the thickness of the member less the sum of the specified concrete covers required for the heads and base rails as shown in Fig. R12.7.5.

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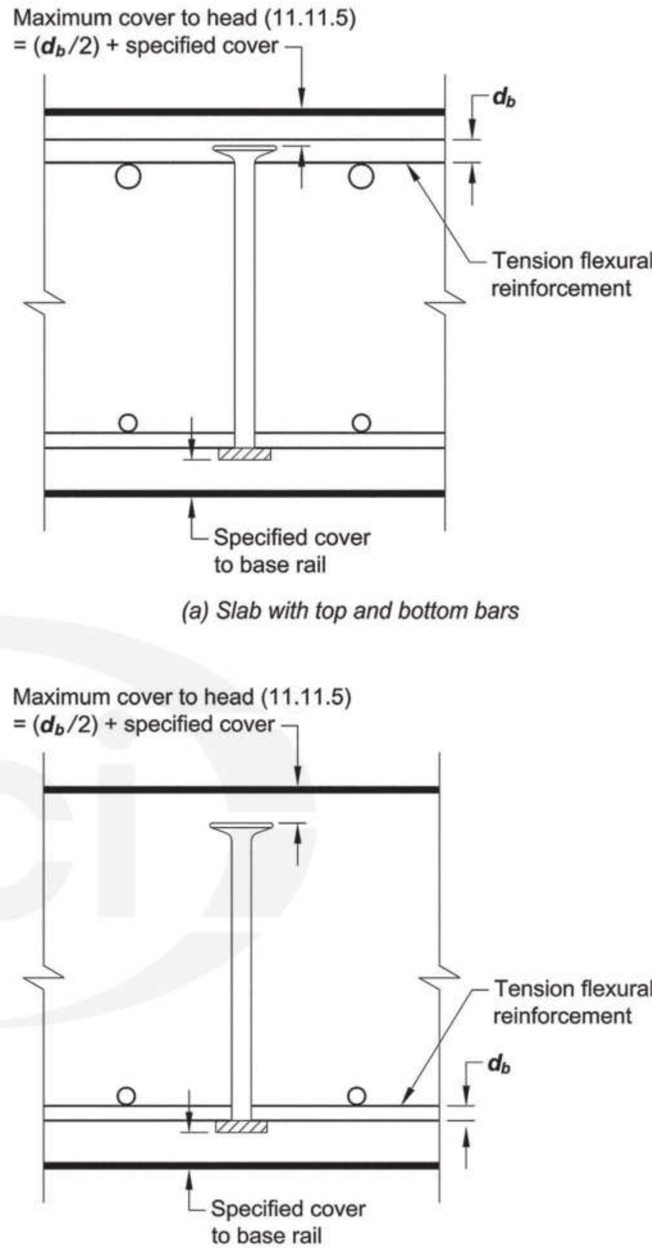
COMMENTARY



(b) Footing with only bottom bars

Fig. R12.7.5—Concrete cover requirements for headed shear stud reinforcement.

12.7.6 *Future extensions*

Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

12.7.7 *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 concrete cover specified in 12.7, such greater thicknesses shall be used.

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12.8—Development

12.8.1 *Development of reinforcement—General*

12.8.1.1 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, headed deformed bar, mechanical device, or a combination thereof. Hooks and headed deformed bars shall not be used to develop bars in compression.

12.8.1.2 The values of $\sqrt{f_c'}$ used in this chapter shall not exceed 100 psi.

12.8.1.3 In addition to requirements in this chapter that affect detailing of reinforcement, structural integrity requirements of 12.14 shall be satisfied.

12.8.2 *Development of deformed bars and deformed wire in tension*

12.8.2.1 Development length for deformed bars and deformed wire in tension, ℓ_d, shall be determined from either 12.8.2.2 or 12.8.2.3, but shall not be less than 12 in.

12.8.2.2 For deformed bars or deformed wire, ℓ_d shall be as follows:

COMMENTARY

R12.8—Development

R12.8.1 *Development of reinforcement—General*

The development length concept for anchorage of reinforcement was first introduced in ACI 318-71, 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 (ACI Committee 408 1966).

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 12.8.10.2.

Structural integrity requirements of 12.14 may control detailing of reinforcement at splices and terminations.

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.

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.

R12.8.2 *Development of deformed bars and deformed wire in tension*

The general development length equation (Eq. (12-1)) is given in 12.8.2.3. The equation is based on the expression for development length previously endorsed by ACI Committee 408 (ACI 408.1R; Jirsa et al. 1979). In Eq. (12-1), c_b is a factor that represents the 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

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COMMENTARY

Spacing and cover	No. 6 and smaller bars and deformed wires	No. 7 and larger bars
Clear spacing of bars or wires 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 or wires being developed or spliced not less than $2d_b$ and clear cover not less than d_b	$\left(\frac{f_y\psi_t\psi_e}{25\lambda\sqrt{f_c'}}\right)d_b$	$\left(\frac{f_y\psi_t\psi_e}{20\lambda\sqrt{f_c'}}\right)d_b$
Other cases	$\left(\frac{3f_y\psi_t\psi_e}{50\lambda\sqrt{f_c'}}\right)d_b$	$\left(\frac{3f_y\psi_t\psi_e}{40\lambda\sqrt{f_c'}}\right)d_b$

the bars or wires. K_{tr} is a factor that represents the contribution of confining reinforcement across potential splitting planes. ψ_t is the traditional reinforcement location factor to reflect the adverse effects of the top reinforcement casting position. ψ_e is a coating factor reflecting the effects of epoxy coating. There is a limit on the product $\psi_t\psi_e$. The reinforcement size factor ψ_s reflects the more favorable performance of smaller diameter reinforcement. A limit of 2.5 is placed on the term $(c_b + K_{tr})/d_b$. When $(c_b + K_{tr})/d_b$ 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) includes the effects of all variables controlling the development length. Terms in Eq. (12-1) may be disregarded when such omission results in longer and, hence, more conservative, development lengths.

The provisions of 12.8.2.2 and 12.8.2.3 give a two-tier approach. The user can either calculate ℓ_d based on the actual $(c_b + K_{tr})/d_b$ (Section 12.8.2.3) or calculate ℓ_d using 12.8.2.2, which is based on two preselected values of $(c_b + K_{tr})/d_b$.

Section 12.8.2.2 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 $(c_b + K_{tr})/d_b$ 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 $\ell_d = [f_y\psi_t\psi_e/(20\lambda\sqrt{f_c'})]d_b$. 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 (ACI 408.1R) indicated that for No. 6 deformed bars and smaller, as well as for deformed wire, the development lengths could be reduced 20 percent using $\psi_s = 0.80$. This is the basis for the middle column of the table in 12.8.2.2. With less cover and in the absence of minimum ties or stirrups, the minimum clear spacing limits of 12.6.1 and the minimum concrete cover requirements of 12.7 result in minimum values of c_b of equal to d_b. Thus, for "other cases," the values are based on using $(c_b + K_{tr})/d_b = 1.0$ in Eq. (12-1).

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COMMENTARY

The user may easily construct simple, useful expressions. For example, in all structures with normalweight concrete (**λ = 1.0**), uncoated reinforcement (**ψ_e = 1.0**), No. 7 or larger bottom bars (**ψ_t = 1.0**) with **f_c' = 4000 psi** and Grade 60 reinforcement, the equations reduce to

$$\ell_d = \frac{(60{,}000)(1.0)(1.0)}{20(1.0)\sqrt{4000}}d_b = 47d_b$$

or

$$\ell_d = \frac{3(60{,}000)(1.0)(1.0)}{40(1.0)\sqrt{4000}}d_b = 71d_b$$

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, then $\ell_d = 47d_b$. 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 12.8.2.3 to produce significantly shorter development lengths than allowed by 12.8.2.2. 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 $(c_b + K_{tr})/d_b$ value of 2.5, and would require a development length of only $28d_b$ for the example above.

Before ACI 318-08, Eq. (12-2) for K_{tr} included the yield strength of the transverse reinforcement. The current expression includes only the area and spacing of the transverse reinforcement and the number of wires or bars being developed or lap spliced because tests demonstrate that transverse reinforcement rarely yields during a bond failure (Azizinamini et al. 1995).

CODE

12.8.2.3 For deformed bars or deformed wire, ℓ_d shall be

$$\ell_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f_c'}}\frac{\psi_t\psi_e\psi_s}{\left(\dfrac{c_b + K_{tr}}{d_b}\right)}\right)d_b \tag{12-1}$$

in which the confinement term $(c_b + K_{tr})/d_b$ shall not be taken greater than 2.5, and

$$K_{tr} = \frac{40A_{tr}}{sn} \tag{12-2}$$

where n is the number of bars or wires being spliced or developed along the plane of splitting. It shall be permitted to use $K_{tr} = 0$ as a design simplification even if transverse reinforcement is present.

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12.8.2.4 The factors used in the expressions for development of deformed bars and deformed wires in tension in 12.8.2 are as follows:

(a) Where horizontal reinforcement is placed such that more than 12 in. of fresh concrete is cast below the development length or splice, ψ_t = **1.3**. For other situations, ψ_t = **1.0**.

(b) For epoxy-coated bars or wires with cover less than $3d_b$, or clear spacing less than $6d_b$, ψ_e = **1.5**. For all other epoxy-coated bars or wires, ψ_e = **1.2**. For uncoated and zinc-coated (galvanized) reinforcement, ψ_e = **1.0**. However, the product of $\psi_t\psi_e$ need not be taken greater than 1.7.

(c) For No. 6 and smaller bars and deformed wires, ψ_s = **0.8**. For No. 7 and larger bars, ψ_s = **1.0**.

(d) Where lightweight concrete is used, λ shall not exceed **0.75** unless f_{ct} is specified (refer to 8.6.1). Where normalweight concrete is used, λ = **1.0.**

12.8.2.5 *Excess reinforcement*

Reduction in ℓ_d shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for f_y is specifically required or the reinforcement is designed under provisions of 13.1.1.6: (A_s required)/(A_s provided)

COMMENTARY

R12.8.2.4 The reinforcement location factor ψ_t accounts for position of the reinforcement in freshly placed concrete. The factor was reduced to 1.3 in ACI 318-89 to reflect research (Jirsa and Breen 1981; Jeanty et al. 1988).

The factor λ for lightweight concrete was made the same for all types of lightweight aggregates in ACI 318-89. 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 (used at the time as a multiplier in the numerator of development length equations), was selected. A unified definition of λ was adopted in ACI 318-08. Because a single definition of λ is now used in the Code, the term λ has been moved from the numerator to the denominator in the development length equations (1.0/0.75 = 1.33). Section 12.8.2.4 allows a higher factor to be used when the splitting tensile strength of the lightweight concrete is specified. Refer to ACI 318-08 Section 5.1.4.

Studies (Treece and Jirsa 1989; Johnston and Zia 1982; Mathey and Clifton 1976) 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 (Orangun et al. 1977) 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 or zinc and epoxy-dual 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, research (Azizinamini et al. 1999a,b) 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.

R12.8.2.5 *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 for development of positive moment reinforcement at supports according to 12.8.11.2, for development of shrinkage and temperature reinforcement according to 12.13.2.9, or for development of reinforcement provided according to 12.14 and 14.3.8.5.

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12.8.3 *Development of deformed bars and deformed wire in compression*

12.8.3.1 Development length for deformed bars and deformed wire in compression, ℓ_{dc}, shall be determined from 12.8.3.2 and applicable modification factors of 12.8.3.3, but ℓ_{dc} shall not be less than 8 in.

12.8.3.2 For deformed bars and deformed wire, ℓ_{dc} shall be taken as the larger of $(0.02f_y/\lambda\sqrt{f_c'})d_b$ and $(0.0003f_y)d_b$, where the constant 0.0003 carries the unit of in.^{2}/lb.

12.8.3.3 The length ℓ_{dc} in 12.8.2.2 shall be permitted to be multiplied by the applicable factors for:

(a) Reinforcement in excess of that required by analysis: (A_s required)/(A_s 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 12.10.1.5 and spaced at not more than 4 in. on center 0.75

12.8.4 *Development of bundled bars*

12.8.4.1 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.

12.8.4.2 For determining the appropriate spacing and cover values in 12.8.2, the confinement term in 12.8.2.3, and the ψ_e factor in 12.8.2.4(b), a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area and having a centroid that coincides with that of the bundled bars.

12.8.5 *Development of standard hooks in tension*

12.8.5.1 Development length, in inches, for deformed bars in tension terminating in a standard hook (refer to 12.1), ℓ_{dh}, shall be determined from 12.8.5.2 and the applicable modification factors of 12.8.5.3, but ℓ_{dh} shall not be less than $8d_b$ nor less than 6 in.

12.8.5.2 For deformed bars, ℓ_{dh} shall be $(0.02\psi_e f_y/\lambda\sqrt{f_c'})d_b$, with ψ_e taken as 1.2 for epoxy-coated reinforcement and

COMMENTARY

R12.8.3 *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.

R12.8.4 *Development of bundled bars*

R12.8.4.1 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.

It is important to also note 12.6.6.4 relating to the cutoff points of individual bars within a bundle and 12.9.1.2.2 relating to splices of bundled bars. The increases in development length of 12.8.4 do apply when computing splice lengths of bundled bars in accordance with 12.9.1.2.2. The development of bundled bars by a standard hook of the bundle is not covered by the provisions of 12.8.5.

R12.8.4.2 Although splice and development lengths of bundled bars are a multiplier of the diameter of the individual bars being spliced 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 the spacing and cover values in 12.8.2.2, the confinement term $[c_b + K_{tr})/d_b]$ in 12.8.2.3, and the ψ_e factor in 12.8.2.4(b). For bundled bars, bar diameter d_b outside the brackets in the expressions of 12.8.2.2 and of Eq. (12-1) is that of a single bar.

R12.8.5 *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 stan-

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λ taken as 0.75 for lightweight-aggregate concrete. For other cases, ψ_e and λ shall be taken as 1.0.

12.8.5.3 Length ℓ_{dh} in 12.8.5.2 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-degree hook with cover on bar extension beyond hook not less than 2 in.: 0.7

(b) For 90-degree 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 ℓ_{dh}; 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-degree 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 ℓ_{dh}: 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 12.8.3.3(b) and 12.8.5.3(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.

COMMENTARY

dard hooks (refer to 12.1) are considered and the influence of larger radius of bend cannot be evaluated by 12.8.5.

The hooked bar anchorage provisions give the total hooked bar embedment length as shown in Fig. R12.8.5. 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_{dh}, of 12.8.5.2 can be reduced by all applicable modification factors of 12.8.5.3. As an example, if the conditions of both 12.8.5.3(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 ACI 408.1R and Jirsa et al. (1979).

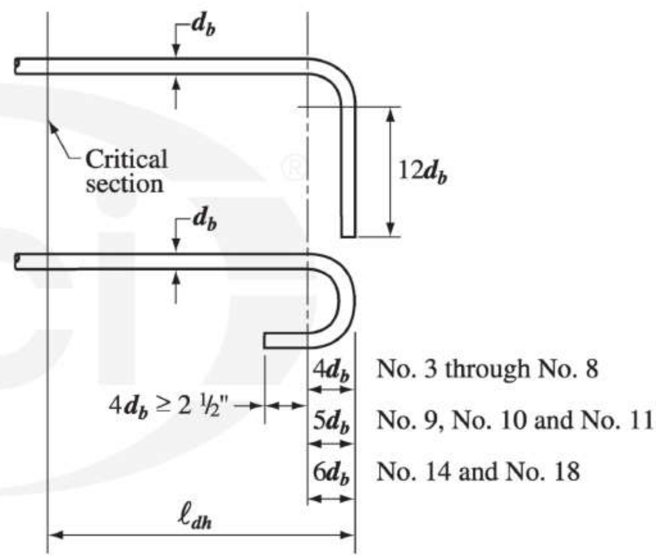

Fig. R12.8.5—Hooked bar details for development of standard hooks.

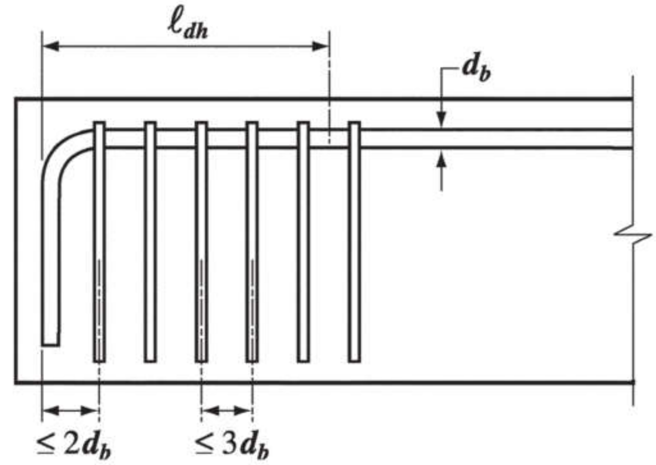

Fig. R12.8.5.3a—Ties or stirrups placed perpendicular to the bar being developed, spaced along the development length ℓ_{dh}.

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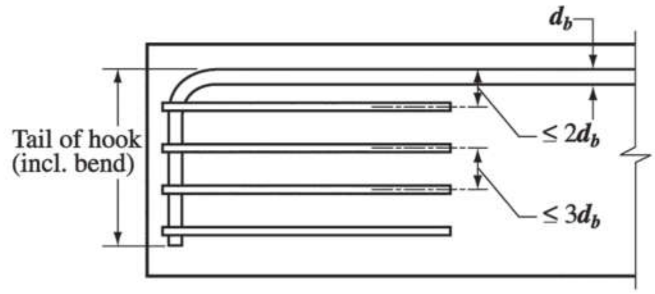

Fig. R12.8.5.3b—Ties or stirrups placed parallel to the bar being developed, spaced along the length of the tail extension of the hook plus bend.

Tests (Jirsa and Marques 1975) 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 12.8.5.3(b) may be used are illustrated in Fig. R12.8.5.3a and R12.8.5.3b. Figure R12.8.5.3a shows placement of ties or stirrups perpendicular to the bar being developed, spaced along the development length ℓ_{dh} of the hook. Figure R12.8.5.3b shows placement of ties or stirrups parallel to the bar being 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 12.8.5.3(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 (Hamad et al. 1993) indicate that the development length for hooked bars should be increased by 20 percent to account for reduced bond when reinforcement is epoxy coated.

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12.8.5.4 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 ℓ_{dh}. The first tie or stirrup shall enclose the bent portion of the hook, within $2d_b$ of the outside of the bend, where d_b is the diameter of the hooked bar. For this case, the factors of 12.8.5.3(b) and (c) shall not apply.

COMMENTARY

R12.8.5.4 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. Refer to Fig. R12.8.5.4. 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 confinement 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.

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12.8.5.5 Hooks shall not be considered effective in developing bars in compression.

12.8.6 *Development of headed and mechanically anchored deformed bars in tension*

12.8.6.1 Development length for headed deformed bars in tension, ℓ_{dt}, shall be determined from 12.8.6.2. Use of heads to develop deformed bars in tension shall be limited to conditions satisfying (a) through (f):

(a) bar f_y shall not exceed 60,000 psi
(b) bar size shall not exceed No. 11
(c) concrete shall be normalweight
(d) net bearing area of head A_{brg} shall not be less than four times the area of the bar A_b
(e) clear cover for bar shall not be less than $2d_b$
(f) clear spacing between bars shall not be less than $4d_b$

12.8.6.2 For headed deformed bars satisfying 3.5.9, development length in tension, ℓ_{dt}, shall be $(0.016\psi_e f_y/\sqrt{f_c'})d_b$, where the value of f_c' used to calculate ℓ_{dt} shall not exceed 6000 psi, and factor ψ_e shall be taken as 1.2 for epoxy-coated reinforcement and 1.0 for other cases. ℓ_{dt} shall not be less than the larger of $8d_b$ and 6 in.

12.8.6.2.1 Where reinforcement is in excess of that required by analysis, except where development of f_y is specifically required, ℓ_{dt} shall be permitted to be multiplied by (A_s required)/(A_s provided).

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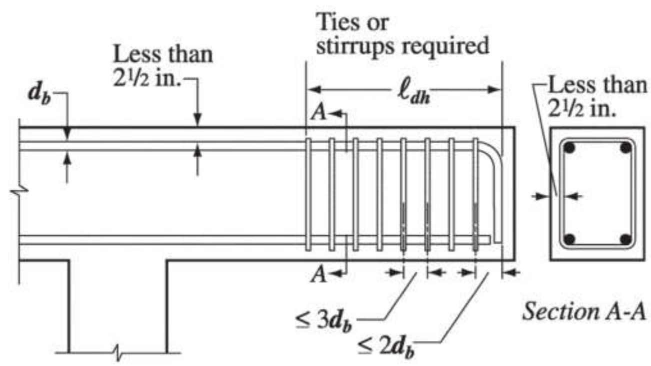

Fig. R12.8.5.4—Concrete cover per 12.8.5.4.

Also, provisions of 12.8.5.4 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.

R12.8.5.5 In compression, hooks are ineffective and may not be used as anchorage.

R12.8.6 *Development of headed and mechanically anchored deformed bars in tension*

The development of headed deformed bars and the development and anchorage of deformed bars through the use of mechanical devices within concrete are addressed in 12.8.6. As used in 12.8.6, "development" describes cases in which the force in the bar is transferred to the concrete through a combination of a bearing force at the head and bond forces along the bar. Such cases are covered in 12.8.6.1 and 12.8.6.2. In contrast, "anchorage" describes cases in which the force in the bar is transferred through bearing to the concrete at the head alone. Design requirements for anchors are given in Appendix E. Headed bars are limited to those types that meet the requirements of HA heads in ASTM A970/A970M because a wide variety of methods are used to attach heads to bars, some of which involve significant obstructions or interruptions of the bar deformations. Headed bars with significant obstructions or interruptions of the bar deformations were not evaluated in the tests used to formulate the provisions in 12.8.6.2. The headed bars evaluated in the tests were limited to those types that meet the criteria in 3.5.9 for HA heads.

The provisions for headed deformed bars were written with due consideration of the provisions for anchorage in Appendix E and the bearing strength provisions of 10.14 (Thompson et al. 2005, 2006a,b). Appendix E contains provisions for headed anchors related to the individual failure modes of concrete breakout, side-face blowout, and pullout, all of which were considered in the formulation of 12.8.6.2. The restrictions on normalweight concrete, maximum bar size of No. 11, and upper limit of 60,000 psi for f_y are based on the available data from tests (Thompson et al. 2005, 2006b).

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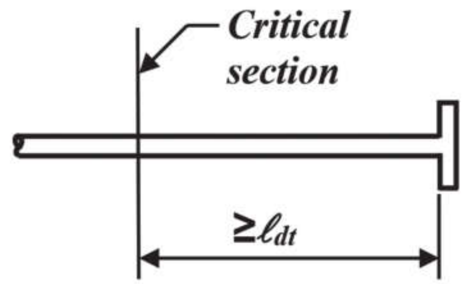

Fig. R12.8.6a—Development of headed deformed bars.

The provisions for developing headed deformed bars give the length of bar, ℓ_{dt}, measured from the critical section to the bearing face of the head, as shown in Fig. R12.8.6a.

For bars in tension, heads allow the bars to be developed in a shorter length than required for standard hooks (Thompson et al. 2005, 2006a,b). The minimum limits on clear cover, clear spacing, and head size are based on the lower limits of these parameters used in the tests to establish the expression for ℓ_{dt} in 12.8.6.2. The clear cover and clear spacing requirements in 12.8.6.1 are based on dimensions measured to the bar, not to the head. The head is considered part of the bar for the purposes of satisfying the specified cover requirements in 12.7, and aggregate size requirements of 3.3.2(c). To avoid congestion, it may be desirable to stagger the heads. Headed bars with $A_{brg} < 4A_b$ have been used in practice, but their performance is not accurately represented by the provisions in 12.8.6.2, and they should be used only with designs that are supported by test results under 12.8.6.4. These provisions do not address the design of studs or headed stud assemblies used for shear reinforcement.

A 1.2 factor is conservatively used for epoxy-coated headed deformed reinforcing bars, the same value used for epoxy-coated standard hooks. The upper limit on the value of f_c' in 12.8.6.1 for use in calculating ℓ_{dt} is based on the concrete strengths used in the tests (Thompson et al. 2005, 2006a,b). Because transverse reinforcement has been shown to be largely ineffective in improving the anchorage of headed deformed bars (Thompson et al. 2005, 2006a,b), additional reductions in development length, such as those allowed for standard hooks with additional confinement provided by transverse reinforcement in 12.8.5.3, are not used for headed deformed reinforcing bars. Transverse reinforcement, however, helps limit splitting cracks in the vicinity of the head and for that reason is recommended.

In 2011, the excess reinforcement factor for headed bars was removed from the ACI 318 code. The excess reinforcement factor (A_s required/A_s provided), applicable to deformed bars without heads, is not applicable for headed bars where force is transferred through a combination of bearing at the head and bond along the bar. Concrete breakout due to bearing at the head was considered in developing the provisions of 12.6. Because the concrete breakout capacity of a headed bar is a function of the embedment depth to the 1.5 power (refer to Appendix E Eq. (E-6)), a reduction in development length with the application of

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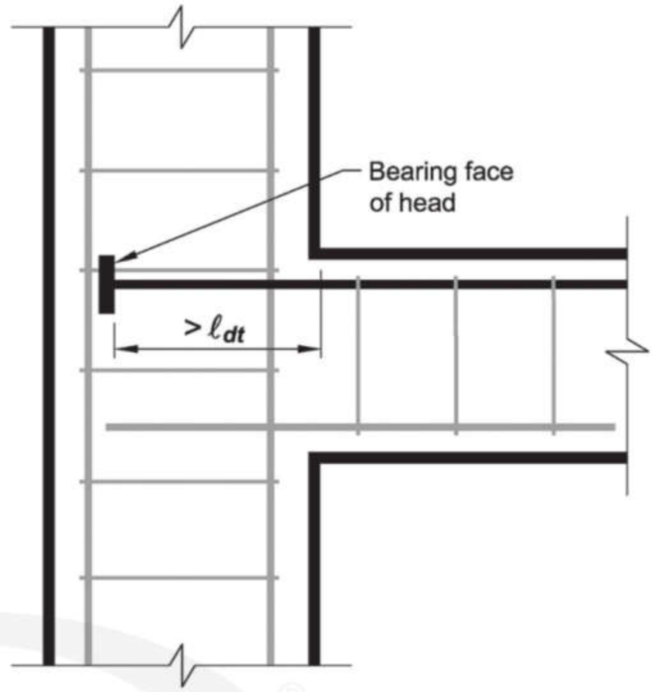

Fig. R12.8.6b—Headed deformed bar extended to far side of column with anchorage length that exceeds ℓ_{dt}.

the excess reinforcement factor could result in a potential concrete breakout failure.

Where longitudinal headed bars from a beam or a slab terminate at a supporting member, such as the column shown in Fig. R12.8.6b, the bars should extend through the joint to the far face of the supporting member, allowing for cover and avoidance of interference with column reinforcement, even though the resulting anchorage length exceeds ℓ_{dt}. Extending the bar to the far side of the column helps to anchor compressive forces (as identified in a strut-and-tie model) that are likely to form in such a connection and improves the performance of the joint.

12.8.6.3 Heads shall not be considered effective in developing bars in compression.

R12.8.6.3 No data are available that demonstrate that the use of heads adds significantly to anchorage capacity in compression.

12.8.6.4 Any mechanical attachment or device capable of developing f_y of reinforcement is allowed, provided that test results showing the adequacy of such attachment or device are approved by the building official. Development of reinforcement shall be permitted to consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between the critical section and the mechanical attachment or device.

R12.8.6.4 Headed deformed bars that do not meet the requirements in 3.5.9, including the limitation on obstructions and interruptions of the deformations, or is not anchored in accordance with 12.8.6.2 may be used if tests demonstrate the ability of the head and bar system to develop or anchor the desired force in the bar, as described in 12.8.6.4.

12.8.7 *Development of welded deformed wire reinforcement in tension*

R12.8.7 *Development of welded deformed wire reinforcement in tension*

Figure R12.8.7 shows the development requirements for welded deformed wire reinforcement with one cross wire within the development length. ASTM A1064 for welded

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12.8.7.1 Development length for welded deformed wire reinforcement in tension, ℓ_d, measured from the point of critical section to the end of wire shall be computed as the product ℓ_d from 12.8.2.2 or 12.8.2.3 times welded deformed wire reinforcement factor ψ_w from 12.8.7.2 or 12.8.7.3. It shall be permitted to reduce ℓ_d in accordance with 12.8.2.5 when applicable, but ℓ_d shall not be less than 8 in. except in computation of lap splices by 12.9.5. When using the ψ_w factor from 12.8.7.2, it shall be permitted to use an epoxy-coating factor ψ_e of 1.0 for epoxy-coated welded deformed wire reinforcement in 12.8.2.2 and 12.8.2.3.

12.8.7.2 For welded deformed wire reinforcement with at least one cross wire within ℓ_d and not less than 2 in. from the point of the critical section, ψ_w, the wire reinforcement factor, shall be the greater of

$$\left(\frac{f_y - 35{,}000}{f_y}\right)$$

or

$$\left(\frac{5d_b}{s}\right)$$

but not greater than 1.0, where s is the spacing between the wires to be developed.

12.8.7.3 For welded deformed wire reinforcement with no cross wires within ℓ_d or with a single cross wire less than 2 in. from the point of the critical section, ψ_w shall be taken as 1.0, and ℓ_d shall be determined as for deformed wire.

12.8.7.4 When any plain wires are present in the welded deformed wire reinforcement in the direction of the development length, the reinforcement shall be developed in accordance with 12.8.8.

12.8.8 *Development of welded plain wire reinforcement in tension*

Yield strength of welded plain wire reinforcement 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, ℓ_d shall not be less than

$$\ell_d = 0.27\frac{A_b}{s}\frac{f_y}{\lambda\sqrt{f_c'}} \tag{12-3}$$

where ℓ_d is measured from the point of the critical section to the outermost crosswire, s is the spacing between the wires to be developed, and λ as given in 12.8.2.4(d). Where reinforcement provided is in excess of that required, ℓ_d may be reduced in accordance with 12.8.2.5. ℓ_d shall not be less than 6 in. except in computation of lap splices by 12.9.6.

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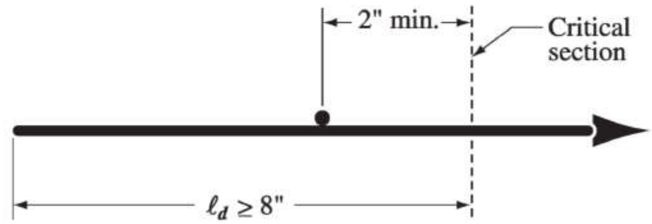

Fig. R12.8.7—Development of welded deformed wire reinforcement.

deformed wire reinforcement requires the same strength of the weld as required for welded plain wire reinforcement. 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 ACI 318 code provisions for wire development by assuming that only one crosswire is contained in the development length. The welded deformed wire reinforcement factor ψ_w of 12.8.7.2 is applied to the deformed wire development length computed from 12.8.2. The factor ψ_w was derived using the general relationships between welded wire reinforcement and deformed wires in the ℓ_{db} values of ACI 318-83.

Tests (Bartoletti and Jirsa 1995) have indicated that epoxy-coated welded wire reinforcement has essentially the same development and splice strengths as uncoated welded wire reinforcement 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 reinforcement with cross wires within the splice or development length.

Deformed wire larger than D-31 is treated as plain wire because tests show that D-45 wire will achieve only approximately 60 percent of the bond strength in tension given by Eq. (12-1) (Rutledge and DeVries 2002).

R12.8.8 *Development of welded plain wire reinforcement in tension*

Figure R12.8.8 shows the development requirements for welded plain wire reinforcement with development primarily dependent on the location of cross wires. For welded plain wire reinforcement made with the smaller wires, an embedment of at least two cross wires 2 in. or more beyond the

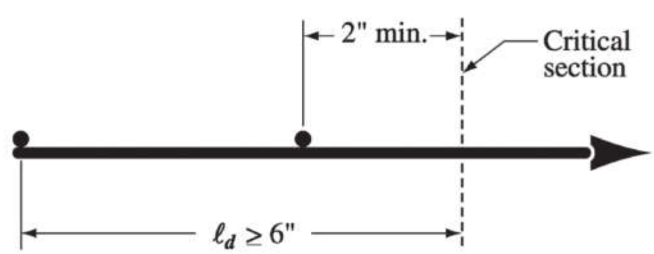

Fig. R12.8.8—Development of welded plain wire reinforcement.

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point of critical section is adequate to develop the full yield strength of the anchored wires. However, for welded plain wire reinforcement made with larger closely spaced wires a longer embedment is required, and a minimum development length is provided for this reinforcement.

12.8.9 *Development of prestressing strand*

12.8.9.1 Except as provided in 12.8.9.1.1, seven-wire strand shall be bonded beyond the critical section a distance not less than

$$\ell_d = \left(\frac{f_{se}}{3000}\right)d_b + \left(\frac{f_{ps} - f_{se}}{1000}\right)d_b \qquad (12\text{-}4)$$

The expressions in parentheses are used as a constant without units.

R12.8.9 *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 (***w*/*cm***), 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-***w*/*cm***, no-slump concrete is used.

The first term in Eq. (12-4) 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 in the prestressing steel at nominal strength of the member, f_{ps}, may develop.

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 (Rose and Russell 1997; Logan 1997). 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 12.8.9 do not apply to plain wires or to end anchored tendons. The length for plain 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.

12.8.9.1.1 Embedment less than ℓ_d 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. (12-4).

R12.8.9.1.1 Figure R12.8.9.1.1 shows the relationship between steel stress and the distance over which the strand is bonded to the concrete represented by Eq. (12-4). This idealized variation of strand stress may be used for analyzing sections within the development region (Martin and Korkosz 1995; PCI 2004). 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 kip/in.^{2}; refer

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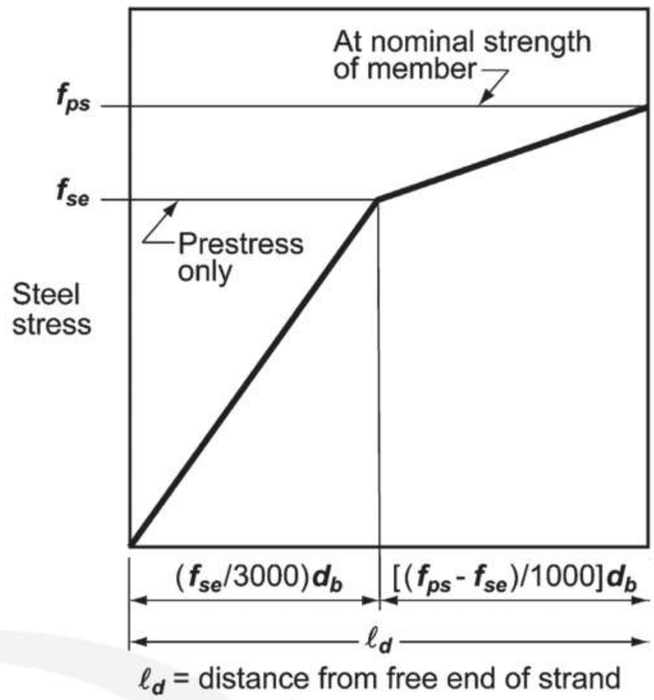

Fig. R12.8.9.1.1—Idealized bilinear relationship between steel stress and distance from the free end of the strand.

to Kaar and Magura (1965), Hanson and Kaar (1959), and Kaar et al. (1963).

12.8.9.2 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.

R12.8.9.2 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. Martin and Korkosz (1995) and PCI (2004) 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.

12.8.9.3 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 19.4.2, ℓ_d specified in 12.8.9.1 shall be doubled.

R12.8.9.3 Exploratory tests (Kaar and Magura 1965) 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 12.8.9.1 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 (Rabbat et al. 1979) indicated that in pretensioned members designed for zero tension in the concrete under service load conditions (refer to 19.4.2), the development length for debonded strands need not be doubled. For

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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.

12.8.10 *Development of flexural reinforcement—General*

R12.8.10 *Development of flexural reinforcement—General*

12.8.10.1 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.

12.8.10.2 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 12.8.11.3 must be satisfied.

R12.8.10.2 Critical sections for a typical continuous beam are indicated with a "c" or an "x" in Fig. R12.8.10.2. For uniform loading, the positive reinforcement extending into the support is more apt to be governed by the requirements of 12.8.11.3 rather than by development length measured from a point of maximum moment or bar cutoff.

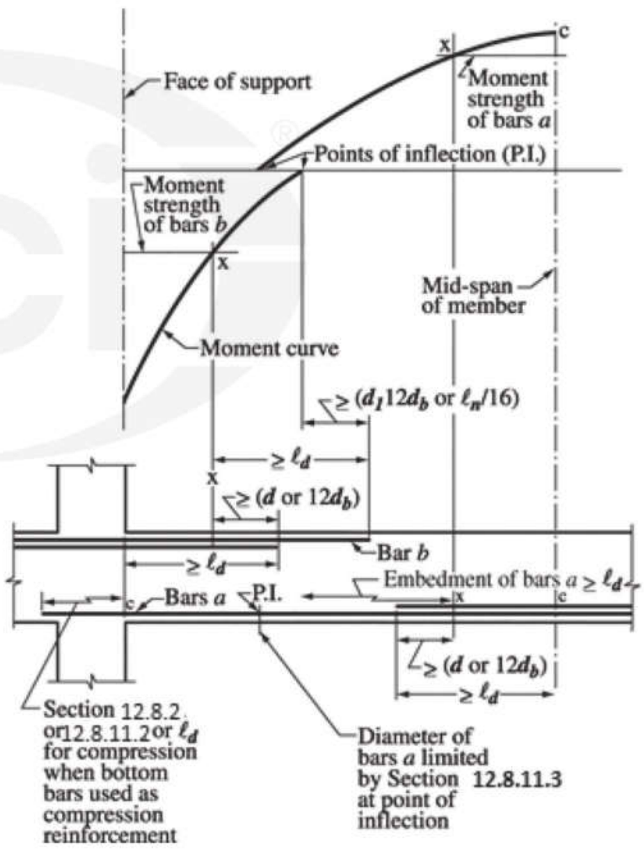

Fig. R12.8.10.2—Development of flexural reinforcement in a typical continuous beam.

12.8.10.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to d or $12d_b$, whichever is greater, except at supports of simple spans and at free end of cantilevers.

R12.8.10.3 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 toward a point of zero moment. When

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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 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. R12.8.10.2.

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 middepth of the member.

12.8.10.4 Continuing reinforcement shall have an embedment length not less than ℓ_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

R12.8.10.4 Peak stresses exist in the remaining bars wherever adjacent bars are cut off, or bent, in tension regions. In Fig. R12.8.10.2, 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.

12.8.10.5 Flexural reinforcement shall not be terminated in a tension zone unless 12.8.10.5.1, 12.8.10.5.2, or 12.8.10.5.3 is satisfied:

12.8.10.5.1 V_u at the cutoff point does not exceed $(2/3)\phi V_n$.

12.8.10.5.2 Stirrup area in excess of that required for shear and torsion is provided along each terminated bar or wire over a distance $(3/4)d$ from the termination point. Excess stirrup area A_v shall be not less than $60b_w s/f_y$. Spacing s shall not exceed $d/(8\beta_b)$.

12.8.10.5.3 For No. 11 bars and smaller, continuing reinforcement provides twice the area required for flexure at the cutoff point and V_u does not exceed $(3/4)\phi V_n$.

R12.8.10.5 Reduced shear strength and loss of ductility when bars are cut off in a tension zone, as in Fig. R12.8.10.2, 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 (refer to 12.8.10.5.1). Diagonal cracks can be restrained by closely spaced stirrups (refer to 12.8.10.5.2). A lower steel stress reduces the probability of such diagonal cracking (refer to 12.8.10.5.3). These requirements are not intended to apply to tension splices that are covered by 12.8.2, 12.8.13.5, and the related 12.9.2.

12.8.10.6 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. Refer to 12.8.11.4 and 12.8.12.4 for deep flexural members.

R12.8.10.6 Brackets, members of variable depth, and other members where f_s, calculated stress in reinforcement at service loads, 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. R12.8.10.6, 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. ACI Committee 408 (1966) suggests a welded crossbar 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

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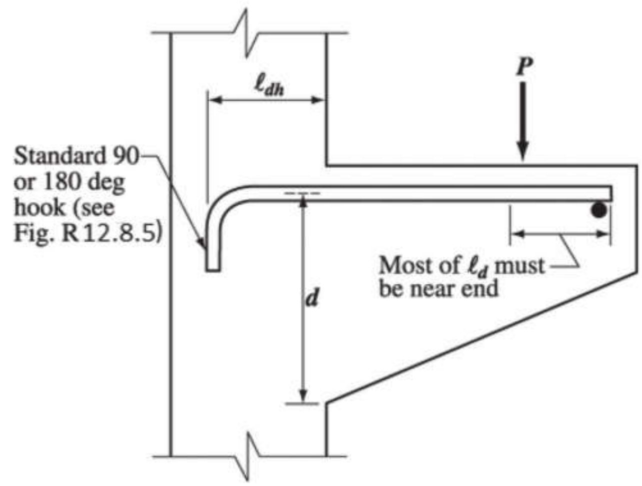

Fig. R12.8.10.6—Special member largely dependent on end anchorage.

not applied close to the corners, U-shaped bars in a horizontal plane provide effective end hooks.

12.8.11 *Development of positive moment reinforcement*

12.8.11.1 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.

12.8.11.2 When a flexural member is part of a primary lateral-load-resisting system, positive moment reinforcement required to be extended into the support by 12.8.11.1 shall be anchored to develop the specified yield strength f_y in tension at the face of support.

12.8.11.3 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 12.8.2 satisfies Eq. (12-5); except Eq. (12-5) 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

$$\ell_d \leq \frac{M_n}{V_u} + \ell_a \tag{12-5}$$

where M_n is calculated assuming all reinforcement at the section to be stressed to f_y; V_u is calculated 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 d or $12d_b$, whichever is greater.

R12.8.11 *Development of positive moment reinforcement*

R12.8.11.1 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.

R12.8.11.2 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.

R12.8.11.3 At simple supports and points of inflection such as "PI" in Fig. R12.8.10.2, 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 R12.8.11.3(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,

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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.

12.8.11.4 At simple supports of deep beams, positive moment tension reinforcement shall be anchored to develop f_y in tension at the face of support except that if design is carried out using Appendix B, the positive moment tension

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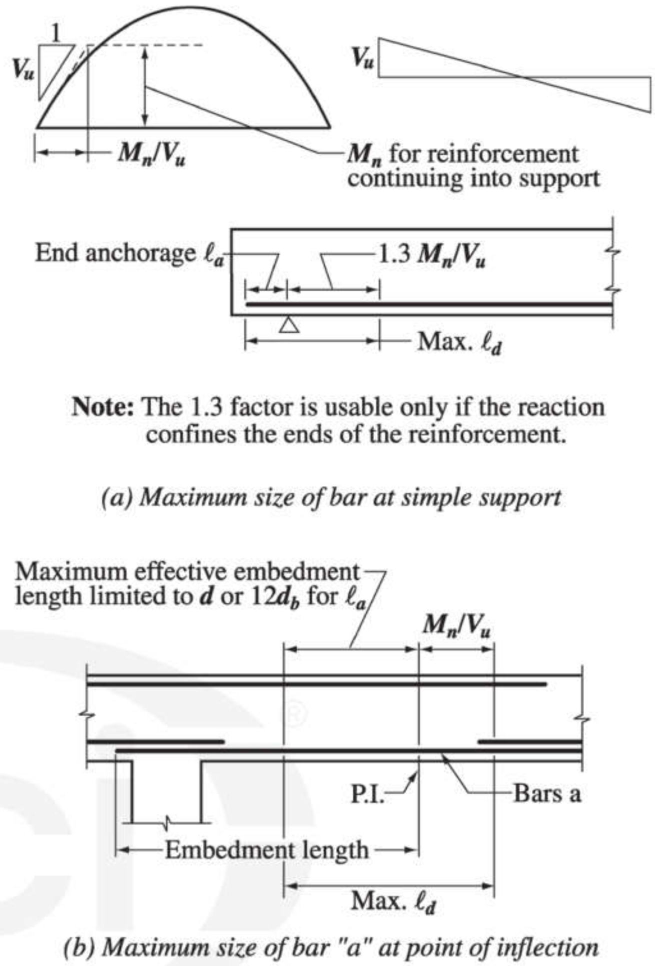

Fig. R12.8.11.3—Concept for determining maximum bar size per 12.8.11.3.

and jd is the moment arm. In ACI 318-71, this anchorage requirement was relaxed from previous 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 R12.8.11.3(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.

R12.8.11.4 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 reinforce-

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reinforcement shall be anchored in accordance with B.4.3. At interior supports of deep beams, positive moment tension reinforcement shall be continuous or be spliced with that of the adjacent spans.

12.8.12 *Development of negative moment reinforcement*

12.8.12.1 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.

12.8.12.2 Negative moment reinforcement shall have an embedment length into the span as required by 12.8.1 and 12.8.10.3.

12.8.12.3 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 d, $12d_b$, or $\ell_n/16$, whichever is greater.

12.8.12.4 At interior supports of deep flexural members, negative moment tension reinforcement shall be continuous with that of the adjacent spans.

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ment to be continuous or be developed through and beyond the support (Rogowsky and MacGregor 1986).

R12.8.12 *Development of negative moment reinforcement*

Figure R12.8.12 illustrates two methods of satisfying requirements for anchorage of tension reinforcement beyond the face of support. For anchorage of reinforcement with hooks, refer to R12.8.5.

Section 12.8.12.3 provides for possible shifting of the moment diagram at a point of inflection, as discussed under R12.8.12.3. This requirement may exceed that of 12.8.12.3, and the more restrictive of the two provisions governs.

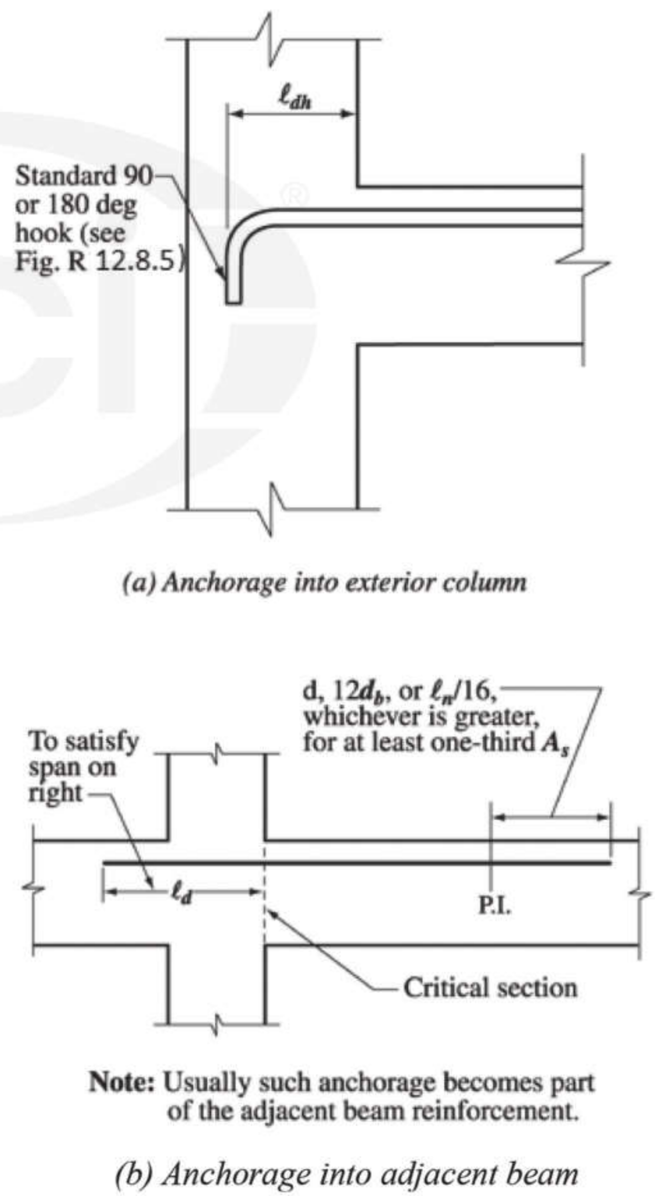

Fig. R12.8.12—Development of negative moment reinforcement.

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12.8.13 *Development of web reinforcement*

12.8.13.1 Web reinforcement shall be as close to the compression and tension surfaces of the member as cover requirements and proximity of other reinforcement permits.

12.8.13.2 Ends of single leg, simple U-, or multiple U-stirrups shall be anchored as required by 12.8.13.2.1 through 12.8.13.2.5:

12.8.13.2.1 For No. 5 bar and D31 wire and smaller, and for No. 6, No. 7, and No. 8 bars with f_{yt} of 40,000 psi or less, a standard hook around longitudinal reinforcement.

12.8.13.2.2 For No. 6, No. 7, and No. 8 stirrups with f_{yt} 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 $0.014 d_b f_{yt}/(\lambda\sqrt{f'_c})$.

12.8.13.2.3 For each leg of welded plain wire reinforcement 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

R12.8.13 *Development of web reinforcement*

R12.8.13.1 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.

R12.8.13.2 The anchorage or development requirements for stirrups composed of bars or deformed wire were changed in ACI 318-89 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.

R12.8.13.2.1 For a No. 5 bar or smaller, anchorage is provided by a standard stirrup hook, as defined in 12.1.3, hooked around a longitudinal bar. ACI 318-89 eliminated the need for a calculated straight embedment length in addition to the hook for these small bars, but 12.8.13.1 requires a full-depth stirrup. Likewise, larger stirrups with f_y equal to or less than 40,000 psi are sufficiently anchored with a standard stirrup hook around the longitudinal reinforcement.

R12.8.13.2.2 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 used in 12.8.5.2 has been adjusted to reflect cover and confinement around the stirrup hook.

For stirrups with f_{yt} of only 40,000 psi, a standard stirrup hook provides sufficient anchorage and these bars are covered in 12.8.13.2.1. For bars with higher strength, the embedment should be checked. A 135- or 180-degree hook is preferred, but a 90-degree hook may be used provided the free end of the 90-degree hook is extended the full 12 bar diameters as required in 12.1.3.

R12.8.13.2.3 The requirements for anchorage of welded plain wire reinforcement stirrups are illustrated in Fig. R12.8.13.2.3.

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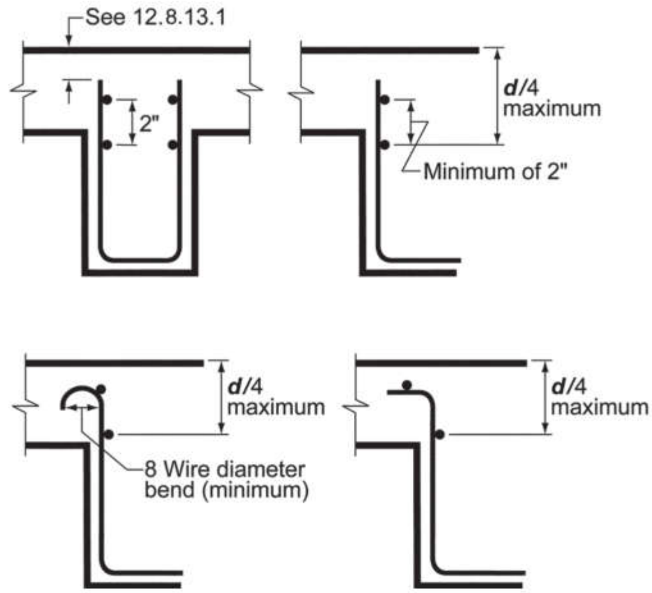

Fig. R12.8.13.2.3—Anchorage in compression zone of welded plain wire reinforcement U-stirrups.

12.8.13.2.4 For each end of a single leg stirrup of welded plain wire reinforcement, 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.

R12.8.13.2.4 Use of welded wire reinforcement for shear reinforcement has become commonplace in the precast, prestressed concrete industry. The rationale for acceptance of straight sheets of wire reinforcement as shear reinforcement is presented in a report by a joint PCI/WRI Ad Hoc Committee on Welded Wire Fabric for Shear Reinforcement (1980).

The provisions for anchorage of single-leg welded wire reinforcement 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 R12.8.13.2.4 illustrates the anchorage requirements for single-leg welded wire reinforcement. For anchorage of single-leg welded wire reinforcement, the Code has permitted hooks and embedment length in the compres-

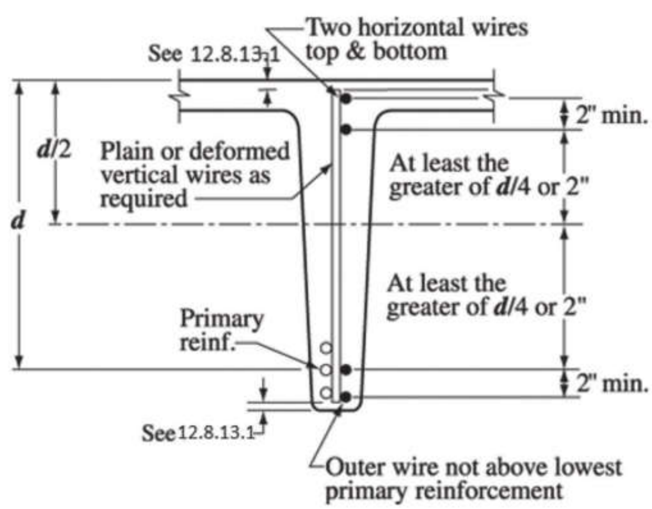

Fig. R12.8.13.2.4—Anchorage of single-leg welded wire reinforcement shear reinforcement.

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sion and tension faces of members (refer to 12.8.13.2.1 and 12.8.13.2.3), and embedment only in the compression face (refer to 12.8.13.2.2). Section 12.8.13.2.4 provides for anchorage of straight single-leg welded wire reinforcement using longitudinal wire anchorage with adequate embedment length in compression and tension faces of members.

12.8.13.2.5 In joist construction as defined in 8.13, for No. 4 bar and D20 wire and smaller, a standard hook.

R12.8.13.2.5 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.

12.8.13.3 Between anchored ends, each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar.

12.8.13.4 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 middepth $d/2$ as specified for development length in 12.8.2 for that part of f_{yt} required to satisfy Eq. (11-17).

12.8.13.5 Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when length of laps is $1.3\ell_d$. In members at least 18 in. deep, such splices with $A_b f_{yt}$ not more than 9000 lb per leg shall be considered adequate if stirrup legs extend the full available depth of member.

R12.8.13.5 These requirements for lapping of double U-stirrups to form closed stirrups control over the provisions of 12.9.2.

12.9—Splices

12.9.1 *Splices of reinforcement—General*

12.9.1.1 Splices of reinforcement shall be made only as required or permitted on design drawings, or in specifications, or as authorized by the licensed design professional.

R12.9—Splices

R12.9.1 *Splices of reinforcement—General*

Splices should, if possible, be located away from points of maximum tensile stress. The lap splice requirements of 12.9.2 encourage this practice.

12.9.1.2 *Lap splices*

R12.9.1.2 *Lap splices*

12.9.1.2.1 Lap splices shall not be used for bars larger than No. 11 except as provided in 12.9.3.2 and 16.8.2.3.

R12.9.1.2.1 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 12.9.3.2 and 16.8.2.3 for compression lap splices of No. 14 and No. 18 bars with smaller bars.

12.9.1.2.2 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 12.8.4. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced.

R12.9.1.2.2 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.

12.9.1.2.3 Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than the smaller of one-fifth the required lap splice length, and 6 in.

R12.9.1.2.3 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

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on the lap splicing of deformed bars was conducted with reinforcement within this spacing.

12.9.1.3 *Mechanical and welded splices*

R12.9.1.3 *Mechanical and welded splices*

12.9.1.3.1 Mechanical and welded splices shall be permitted.

12.9.1.3.2 A full mechanical splice shall develop in tension or compression, as required, at least $\mathbf{1.25}f_y$ of the bar.

R12.9.1.3.2 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.

12.9.1.3.3 Except as provided in this Code, all welding shall conform to "Structural Welding Code—Steel Reinforcing Bars" (AWS D1.4).

R12.9.1.3.3 Refer to R3.5.2 for discussion on welding.

12.9.1.3.4 A full welded splice shall develop at least $\mathbf{1.25}f_y$ of the bar.

R12.9.1.3.4 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. Refer to the discussion on strength in R12.9.1.3.2. ACI 318-95 eliminated a requirement that the bars be butted because indirect butt welds are permitted by AWS D1.4, although AWS D1.4 does indicate that wherever practical, direct butt splices are preferable for No. 7 bars and larger.

12.9.1.3.5 Mechanical or welded splices not meeting requirements of 12.9.1.3.2 or 12.9.1.3.4 shall be permitted only for No. 5 bars and smaller and in accordance with 12.9.2.4.

R12.9.1.3.5 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 12.9.2.4 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. ACI 318-95 limited these lower-strength welds and connections to No. 5 bars and smaller due to the potentially brittle nature of failure at these welds.

12.9.2 *Splices of deformed bars and deformed wire in tension*

R12.9.2 *Splices of deformed bars and deformed wire in tension*

12.9.2.1 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: $\mathbf{1.0}\ell_d$

Class B splice: $\mathbf{1.3}\ell_d$

where ℓ_d is calculated in accordance with 12.8.2 to develop f_y, but without the 12 in. minimum of 12.8.2.1 and without the modification factor of 12.8.2.5.

R12.9.2.1 Lap splices in tension are classified as Type A or B, with length of lap a multiple of the tensile development length ℓ_d calculated in accordance with 12.8.2.2 or 12.8.2.3. 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 12.8.2.5 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. R12.9.2.1(a) illus-

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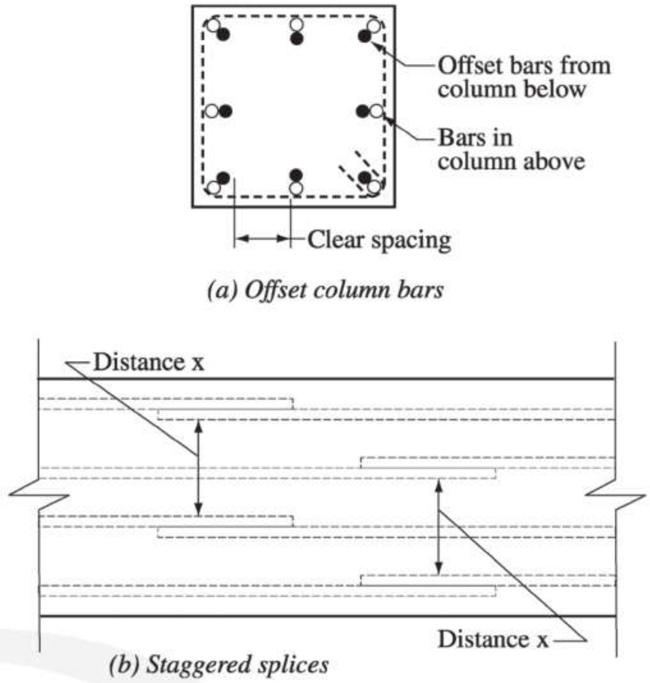

Fig. R12.9.2.1—Clear spacing of spliced bars.

trates the clear spacing to be used. For staggered splices, the clear spacing is the minimum distance between adjacent splices (distance ***x*** in Fig. R12.9.2.1(b)).

ACI 318-89 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 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 splicing bars at points of minimum stress and to stagger splices to improve behavior of critical details.

12.9.2.2 Lap splices of deformed bars and deformed wire in tension shall be Class B splices except that Class A splices are allowed when:

R12.9.2.2 The tension lap splice requirements of 12.9.2.1 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 R12.9.2.2

Table R12.9.2.2—Tension lap splices

A_s provided/A_s 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.

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(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.

12.9.2.3 When bars of different size are lap spliced in tension, splice length shall be the larger of ℓ_d of larger bar and tension lap splice length of smaller bar.

12.9.2.4 Mechanical or welded splices used where area of reinforcement provided is less than twice that required by analysis shall meet requirements of 12.9.1.3.2 or 12.9.1.3.4.

12.9.2.5 Mechanical or welded splices not meeting the requirements of 12.9.1.3.2 or 12.9.1.3.4 shall be permitted for No. 5 bars and smaller if the requirements of 12.9.2.5.1 through 12.9.2.5.3 are met:

12.9.2.5.1 Splices shall be staggered at least 24 in.

12.9.2.5.2 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.

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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 12.9.2.5. 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.

R12.9.2.3 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.

R12.9.2.4 Refer to R12.9.1.3.5. Section 12.9.2.4 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 AWS D1.4/D1.4M.

A full mechanical or welded splice conforming to 12.9.1.3.2 or 12.9.1.3.4 can be used without the stagger requirement instead of the lower-strength mechanical or welded splice.

R12.9.2.5 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.

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12.9.2.5.3 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.

12.9.2.6 Splices in tension tie members shall be made with a full mechanical or full welded splice in accordance with 12.9.1.3.2 or 12.9.1.3.4, and splices in adjacent bars shall be staggered at least 30 in.

12.9.3 *Splices of deformed bars in compression*

12.9.3.1 Compression lap splice length shall be $\mathbf{0.0005}\boldsymbol{f_y d_b}$, for f_y of 60,000 psi or less, or $\mathbf{(0.0009}\boldsymbol{f_y}\mathbf{ - 24)}\boldsymbol{d_b}$ 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.

12.9.3.2 When bars of different size are lap spliced in compression, splice length shall be the larger of ℓ_d of larger bar, and compression lap splice length of smaller bar. Lap splices of No. 14 and No. 18 bars to No. 11 and smaller bars shall be permitted.

12.9.3.3 Mechanical or welded splices used in compression shall meet requirements of 12.9.1.3.2 or 12.9.1.3.4.

12.9.3.4 *End-bearing splices*

12.9.3.4.1 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.

COMMENTARY

R12.9.3 *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 ACI 318-71.

R12.9.3.1 Essentially, lap requirements for compression splices have remained the same since the 1963 ACI Building Code.

The ACI 318-63 values were modified in ACI 318-71 code to recognize various degrees of confinement and to permit design with reinforcement up to 80,000 psi yield strength. Tests (ACI Committee 408 1966; Pfister and Mattock 1963) 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 approximately 10 percent for an increase in specified yield strength from 60,000 to 75,000 psi.

R12.9.3.2 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.

R12.9.3.4 *End-bearing splices*

R12.9.3.4.1 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.

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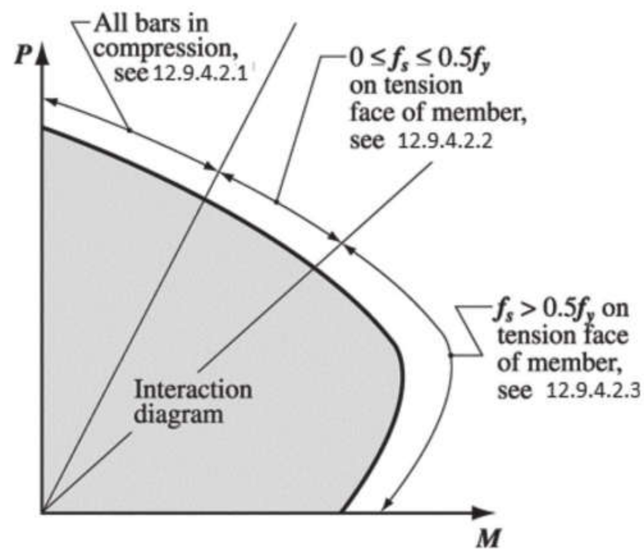

Fig. R12.9.4—Special splice requirements for columns.

12.9.3.4.2 Bar ends shall terminate in flat surfaces within 1.5 degrees of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly.

R12.9.3.4.2 These tolerances were added in ACI 318-71, representing practice based on tests of full-size members containing No. 18 bars.

12.9.3.4.3 End-bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

R12.9.3.4.3 This limitation was added in ACI 318-71 to ensure a minimum shear resistance in sections containing end-bearing splices.

12.9.4 *Special splice requirements for columns*

12.9.4.1 Lap splices, mechanical splices, butt-welded splices, and end-bearing splices shall be used with the limitations of 12.9.2.2 through 12.9.2.4. A splice shall satisfy requirements for all load combinations for the column.

R12.9.4 *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. R12.9.4. When such tensions occur, 12.9.4 requires tension splices to be used or an adequate tensile resistance to be provided. Furthermore, a minimum tensile strength is required in each face of all columns, even where analysis indicates compression only.

ACI 318-89 clarifies this section on the basis that a compressive lap splice has a tensile strength of at least one-fourth f_y, which simplifies the calculation requirements in previous codes.

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.

12.9.4.2 *Lap splices in columns*

12.9.4.2.1 Where the bar stress due to factored loads is compressive, lap splices shall conform to 12.9.3.1, 12.9.3.2, and, where applicable, to 12.9.4.2.4 or 12.9.4.2.5.

R12.9.4.2 *Lap splices in columns*

R12.9.4.2.1 ACI 318-89 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.

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12.9.4.2.2 Where the bar stress due to factored loads is tensile and does not exceed $\mathbf{0.5}\boldsymbol{f_y}$ 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 $\boldsymbol{\ell_d}$.

12.9.4.2.3 Where the bar stress due to factored loads is greater than $\mathbf{0.5}\boldsymbol{f_y}$ in tension, lap splices shall be Class B tension lap splices.

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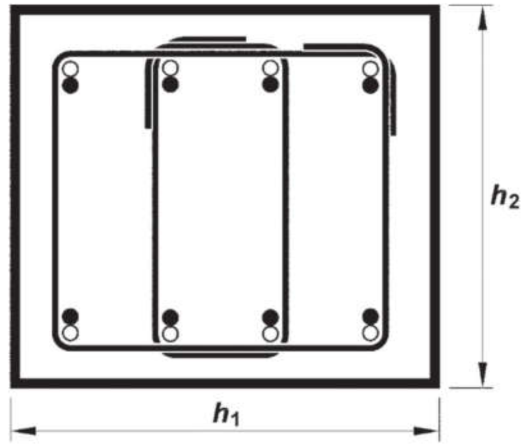

Fig. R12.9.4.2—Example application of 12.9.4.2.

12.9.4.2.4 In tied reinforced compression members, where ties throughout the lap splice length have an effective area not less than $\mathbf{0.0015}\boldsymbol{hs}$ in both directions, 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 $\boldsymbol{h}$ shall be used in determining effective area.

R12.9.4.2.4 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 satisfied in each direction to apply the 0.83 factor. This is illustrated in Fig. R12.9.4.2, where four legs are effective in one direction, and two legs in the other direction. This calculation is critical in one direction, which normally can be determined by inspection.

12.9.4.2.5 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.

R12.9.4.2.5 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 12.11.2.4 and 10.9.3.

12.9.4.3 *Mechanical or welded splices in columns*

Mechanical or welded splices in columns shall meet the requirements of 12.9.1.3.2 or 12.9.1.3.4.

R12.9.4.3 *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 $\boldsymbol{f_y}$ as required by 12.9.1.3.2 or 12.9.1.3.4. Splice strength 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 12.9.3.4 and 12.9.4.4.

12.9.4.4 *End-bearing splices in columns*

End-bearing splices complying with 12.9.3.4 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 $\boldsymbol{f_y}$, not less than $\mathbf{0.25}\boldsymbol{f_y}$ times the area of the vertical reinforcement in that face.

R12.9.4.4 *End-bearing splices in columns*

End-bearing splices used to splice column bars always in compression should have a tension strength 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 12.9.3.4.

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12.9.5 *Splices of welded deformed wire reinforcement in tension*

12.9.5.1 Minimum lap splice length of welded deformed wire reinforcement measured between the ends of each reinforcement sheet shall be not less than $\mathbf{1.3\ell_d}$ nor 8 in., and the overlap measured between outermost cross wires of each reinforcement sheet shall be not less than 2 in. where ℓ_d is calculated in accordance with 12.8.7 to develop f_y.

12.9.5.2 Lap splices of welded deformed wire reinforcement, with no cross wires within the lap splice length, shall be determined as for deformed wire.

12.9.5.3 When any plain wires are present in the deformed wire reinforcement in the direction of the lap splice or when deformed wire reinforcement is lap spliced to welded plain wire reinforcement, the reinforcement shall be lap spliced in accordance with 12.9.6.

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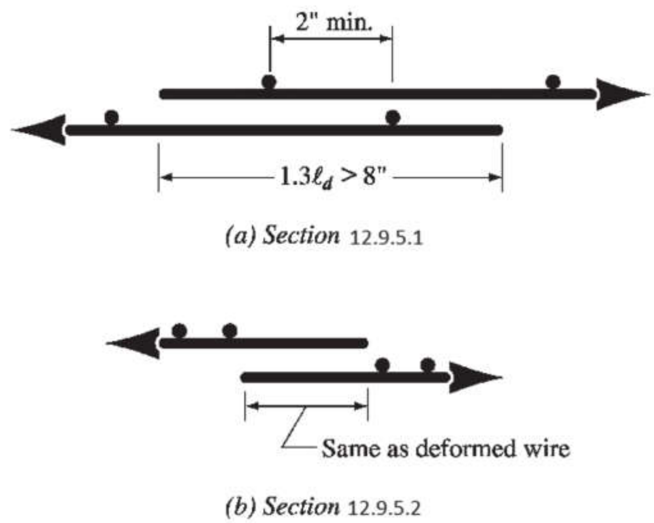

Fig. R12.9.5—Lap splices of welded deformed wire reinforcement.

R12.9.5 *Splices of welded deformed wire reinforcement in tension*

Splice provisions for welded deformed wire reinforcement are based on available tests (Lloyd and Kesler 1969). The requirements were simplified (ACI 318-76 supplement) from provisions of ACI 318-71 by assuming that only one crosswire of each welded wire reinforcement sheet is overlapped and by computing the splice length as $\mathbf{1.3\ell_d}$. The development length ℓ_d is that computed in accordance with the provisions of 12.8.7 without regard to the 8 in. minimum. The 8 in. applies to the overall splice length. Refer to Fig. R12.9.5. If no cross wires are within the lap length, the provisions for deformed wire apply.

12.9.6 *Splices of welded plain wire reinforcement in tension*

Minimum length of lap for lap splices of welded plain wire reinforcement shall be in accordance with 12.9.6.1 and 12.9.6.2.

12.9.6.1 Where A_s provided is less than twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each reinforcement sheet shall be not less than the largest of one spacing of cross wires plus 2 in., $\mathbf{1.5\ell_d}$, and 6 in., where ℓ_d is calculated in accordance with 12.8.8 to develop f_y.

12.9.6.2 Where A_s provided is at least twice that required by analysis at splice location, length of overlap measured

R12.9.6 *Splices of welded plain wire reinforcement in tension*

The strength of lap splices of welded plain wire reinforcement 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 (Lloyd 1971) has shown an increased splice length is required when welded wire reinforcement of large, closely spaced wires, is lapped and, as a consequence, additional splice length requirements are provided for this reinforcement, in addition to an absolute minimum of 6 in. The development length ℓ_d

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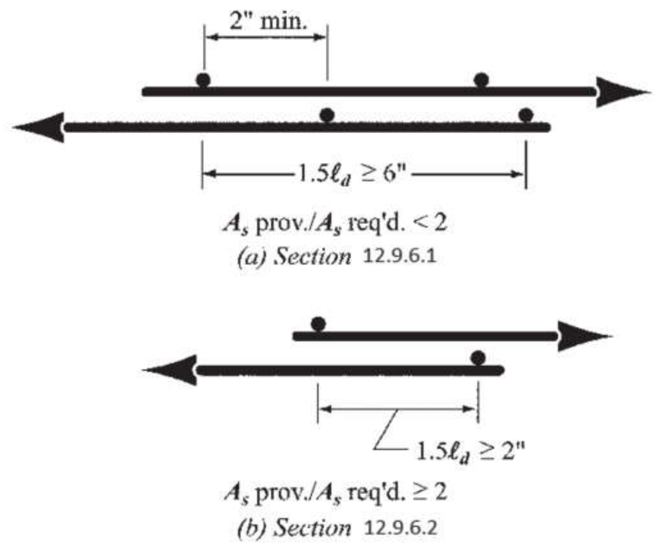

Fig. R12.9.6—Lap splices of welded plain wire reinforcement.

between outermost cross wires of each reinforcement sheet shall not be less than the larger of $1.5\ell_d$, and 2 in. where ℓ_d is calculated in accordance with 12.8.8 to develop f_y.

is that computed in accordance with the provisions of 12.8.8 without regard to the 6 in. minimum. Splice requirements are illustrated in Fig. R12.9.6.

12.10—Lateral reinforcement

12.10.1 *Lateral reinforcement for compression members*

12.10.1.1 Lateral reinforcement for compression members shall conform to the provisions of 12.10.1.4 and 12.10.1.5 and, where shear or torsion reinforcement is required, shall also conform to provisions of Chapter 11.

12.10.1.2 Lateral reinforcement requirements for composite compression members shall conform to 10.13. Lateral reinforcement requirements for tendons shall conform to 19.11.

12.10.1.3 It shall be permitted to waive the lateral reinforcement requirements of 12.10.1, 10.13, and 19.11 where tests and structural analysis show adequate strength and feasibility of construction.

R12.10—Transverse reinforcement

R12.10.1 *Transverse reinforcement for compression members*

R12.10.1.3 Precast columns with cover less than 1-1/2 in., prestressed columns without longitudinal bars, columns smaller than minimum dimensions prescribed in ACI 350-01, columns of concrete with small size coarse aggregate, wall-like columns, and other special cases may require special designs for transverse reinforcement. 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 volumetric reinforcement ratio for the spiral, ρ_s, is to conform to 10.9.3.

12.10.1.4 *Spirals*

Spiral reinforcement for compression members shall conform to 10.9.3 and to the following:

12.10.1.4.1 Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled to permit handling and placing without distortion from designed dimensions.

R12.10.1.4 *Spirals*

For practical considerations in cast-in-place construction, the minimum diameter of spiral reinforcement is 3/8 in. (3/8 in. round, No. 3 bar, or equivalent deformed or plain wire). This is the smallest size that can be used in a column with 1-1/2 in. or more cover and having concrete compressive strengths of 3000 psi or more if the minimum clear spacing for placing concrete is to be maintained.

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12.10.1.4.2 For cast-in-place construction, size of spirals shall not be less than 3/8 in. diameter.

12.10.1.4.3 Clear spacing between spirals shall not exceed 3 in., nor be less than 1 in. Refer also to 3.3.2.

12.10.1.4.4 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.

12.10.1.4.5 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 12.1.3 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 12.1.3 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 12.9.1.3.

12.10.1.4.6 Spirals shall extend from top of footing or slab in any story to level of lowest horizontal reinforcement in members supported above.

12.10.1.4.7 Where beams or brackets do not frame into all sides of a column, ties shall extend above termination of spiral to bottom of slab, drop panel, or shear cap.

12.10.1.4.8 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.

12.10.1.4.9 Spirals shall be held firmly in place and true to line.

12.10.1.5 *Ties*

Tie reinforcement for compression members shall conform to the following:

COMMENTARY

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. However, if one or more sides of the column are not enclosed by beams or brackets, ties are required from the termination of the spiral to the bottom of the slab, drop panel, or shear cap. 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. Refer also to 12.14.

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 should be clearly written to cover the supply of spacers or field tying of spiral reinforcement. In ACI 350-06, splice requirements were modified for epoxy-coated and plain spirals and to allow mechanical splices.

R12.10.1.5 *Ties*

All longitudinal bars in compression should be enclosed within transverse ties. Where longitudinal bars are arranged in a 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.

ACI 318-56 required "lateral support equivalent to that provided by a 90-degree corner of a tie" for every vertical

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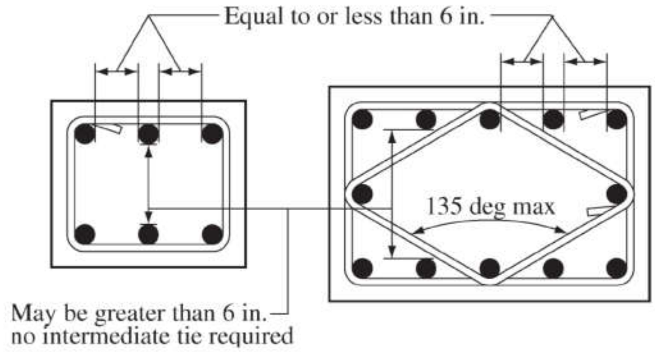

Fig. R12.10.1.5—Sketch to clarify measurements between laterally supported column bars.

bar. Tie requirements were liberalized in 1963 by increasing the permissible included angle from 90 to 135 degrees and exempting bars that are located within 6 in. clear on each side along the tie from adequately tied bars (refer to Fig. R12.10.1.5). Limited tests (Pfister 1964) 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 Pfister (1964), 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. Refer also to 12.14.

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 12.10.1.4; otherwise, it is considered a tie.

12.10.1.5.1 All nonprestressed bars shall be enclosed by transverse 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 reinforcement of equivalent area shall be permitted.

12.10.1.5.2 Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie bar or wire diameters, or least dimension of the compression member.

12.10.1.5.3 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 degrees and no bar shall be farther than

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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.

12.10.1.5.4 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.

R12.10.1.5.4 Vertical splitting and loss of tie restraint are possible where the overlapped ends of adjacent circular ties are anchored at a single longitudinal bar. Adjacent circular ties should not engage the same longitudinal bar with end hook anchorages. While the transverse reinforcement in members with longitudinal bars located around the periphery of a circle can be either spirals or circular ties, spirals are usually more effective.

12.10.1.5.5 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.

12.10.1.5.6 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 transverse 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.

R12.10.1.5.6 With ACI 318-83, 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.

R12.10.1.5.7 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.

12.10.2 *Lateral reinforcement for flexural members*

R12.10.2 *Transverse reinforcement for flexural members*

12.10.2.1 Compression reinforcement in beams shall be enclosed by ties or stirrups satisfying the size and spacing limitations in 12.10.1.5 or by welded wire reinforcement of equivalent area. Such ties or stirrups shall be provided throughout the distance where compression reinforcement is required.

R12.10.2.1 Compression reinforcement in beams and girders should be enclosed to prevent buckling; similar requirements for such enclosure have remained essentially unchanged through several ACI 318 code editions, except for minor clarification.

12.10.2.2 Transverse 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.

12.10.2.3 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 $\mathbf{1.3}\boldsymbol{\ell_d}$) or anchored in accordance with 12.8.13.

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12.11—Reinforcement details for columns

12.11.1 *Offset bars*

Offset bent longitudinal bars shall conform to the following:

12.11.1.1 Slope of inclined portion of an offset bar with axis of column shall not exceed 1 in 6.

12.11.1.2 Portions of bar above and below an offset shall be parallel to axis of column.

12.11.1.3 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.

12.11.1.4 Offset bars shall be bent before placement in the forms. Refer to 12.3.

12.11.1.5 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 12.9.4.

12.11.2 *Steel cores*

Load transfer in structural steel cores of composite compression members shall be provided by the following:

12.11.2.1 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.

12.11.2.2 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.

12.11.2.3 Transfer of stress between column base and footing shall be designed in accordance with 16.8.

12.11.2.4 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.

12.12—Connections

12.12.1 At connections of principal framing elements (such as beams and columns), enclosure shall be provided

COMMENTARY

R12.11—Reinforcement details for columns

R12.11.2 *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, and welds. This provision should ensure that splices in composite compression members meet essentially the same tensile capacity as required for conventionally reinforced concrete compression members.

R12.12—Connections

Confinement is essential at connections to ensure that the flexural strength of the members can be developed without

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for splices of continuing reinforcement and for anchorage of reinforcement terminating in such connections.

12.12.2 Enclosure at connections shall consist of external concrete or internal closed ties, spirals, or stirrups.

12.13—Shrinkage and temperature reinforcement

12.13.1 Reinforcement for shrinkage and temperature stresses shall be provided in slabs-on-ground, walls, suspended slabs, and domes where structural reinforcement is not required or is less than the required shrinkage and temperature reinforcement. The minimum reinforcement provided in any direction and on any face shall be the greater of required shrinkage and temperature reinforcement or structural reinforcement.

12.13.1.1 Shrinkage and temperature reinforcement shall be provided in accordance with either 12.13.2 or 12.13.3.

12.13.1.2 Where shrinkage and temperature movements are significantly restrained, the requirements of 8.2.4 and 9.2.3 shall be considered.

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deterioration of the joint under repeated loadings (Hanson and Conner 1967; ACI 352R).

R12.13—Shrinkage and temperature reinforcement

R12.13.1 In slabs-on-ground, walls, and suspended slabs, where little or no structural reinforcement is required, shrinkage and temperature reinforcement is required to control the width of cracks due to restraint of thermal and shrinkage strains, increase liquid-tightness, reduce the potential for corrosion, and maintain structural continuity.

R12.13.1.1 The shrinkage and temperature reinforcement required by 12.13.2 or 12.13.3 helps prevent cracks wide enough to permit seepage in liquid-containing structures. Refer to Kianoush et al. (2006) for more information on the background for these requirements.

R12.13.1.2 Where walls, foundations, slabs, or large columns cause significant restraint of shrinkage and temperature strains in the same direction as those due to external loads, the reinforcement must be provided to resist the combined stresses to control cracking (refer to PCI [2004] and Gilbert [1992]). Depending on the geometry and the level of restraint, the restrained shrinkage and temperature strains may also cause displacements, shear forces, and flexural moments in these sections.

Consideration should also be given to increasing shrinkage and temperature reinforcement in slabs-on-ground between large pipe, vault, or other penetrations (Fig. R12.13.1.2),

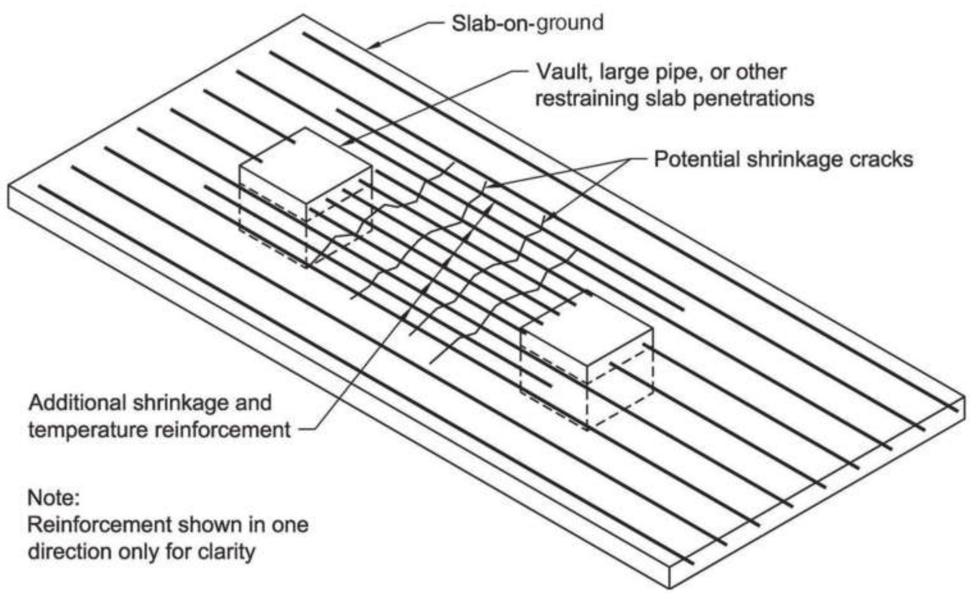

Fig. R12.13.1.2—Concept of shrinkage and temperature reinforcement at locations of increased restraint.

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or reducing the restraint by these items (as with expansion joints or isolation joints, or later connections during the construction period that permit initial shrinkage to occur without causing increased stresses).

12.13.2 Deformed bar reinforcement conforming to 3.5.3 used for shrinkage and temperature reinforcement shall be provided in accordance with 12.13.2.1 through 12.13.2.8 where members are required to be liquid-tight or gas-tight or where enhanced crack control is required for corrosion resistance. Otherwise, shrinkage and temperature reinforcement shall be in accordance with 7.12.2.1 of ACI 318-11.

R12.13.2 The ratios specified for deformed shrinkage and temperature reinforcement are expected to improve performance over prior requirements in areas of maximum restraint. However, there are many other complicating factors that affect the width and spacing of shrinkage cracks in concrete. These include the size and spacing of the reinforcing bars, the amounts and types of cementitious materials, the maximum size and gradation of the aggregates, aggregate type, water content, the type and duration of the curing, and the environmental conditions prevalent during the placing and curing periods. For example, for the same area of reinforcement, a larger number of smaller bars at a closer spacing is believed to improve performance, versus a smaller number of larger bars.

The code provisions are for Grade 60 reinforcement. Lower percentages of shrinkage and temperature reinforcement should not be used for reinforcing steel graders higher than Grade 60. Grade 40 reinforcement is no longer commonly used in the United States for environmental engineering concrete structures, but where used, the percentages of reinforcement should be increased as determined by the licensed design professional.

Welded wire reinforcement, if used for shrinkage and temperature reinforcement in environmental structures, should be specified in flat sheets and properly supported.

Use of the ratios in Section 12.13.2 will not ensure liquid-tightness but should be combined with the knowledge and experience of the licensed design professional and used as one of several important factors for increasing tightness of environmental engineering concrete structures.

12.13.2.1 The area of Grade 60 shrinkage and temperature reinforcement shall provide at least the ratio of reinforcement area to gross concrete area in accordance with Table 12.13.2.1.

R12.13.2.1 In prior ACI 350 reports and codes, the amount of shrinkage and temperature reinforcement varied with distance between "movement dissipating joints." While this generally resulted in satisfactory performance, there were notable exceptions, particularly in what are now categorized as "maximum restraint" zones. There were also concerns that there were large differences in the reinforcement ratio required for small differences in movement joint

Table 12.13.2.1—Minimum shrinkage and temperature reinforcement

	Minimum shrinkage and temperature reinforcement, percent		
Section length*	**Reduced restraint**	**Normal restraint**	**Maximum restraint**
20 ft or less	0.25	0.25	0.50
20 ft to 30 ft	0.25	Linearly interpolate between 0.25 and 0.50	Linearly interpolate between 0.50 and 1.00
30 ft or more	0.25	0.50	1.00

*Section length is either: 1) the distance between any combination of adjacent expansion joints, isolation joints, full contraction joints, or an unrestrained end of element; or 2) for partial contraction joints and crack-inducing joints, based on 1.50 times the distance between the joint and adjacent movement joint (as defined in Section 7.4.1) of any type, or an unrestrained element end.

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spacing. These provisions address this by requiring less or more reinforcement, depending upon the degree of restraint, and independent of the spacing of movement joints (except when 30 ft or less as described in the code provisions).

Many factors affect the width and spacing of shrinkage cracks in liquid-containing structures. Reducing them to levels that minimize seepage goes beyond the calculation of reinforcement values and is related to many factors, such as those mentioned in R12.13.2. The reinforcement ratios indicated in 12.13.2.1 should be considered as minimums and may be increased per the judgment of the licensed design professional.

High shrinkage levels in concrete resulting from mixture properties such as excessive fines content, high water content, and inclusion of certain aggregates can increase the tensile stress in restrained sections. When concrete drying shrinkage measured with ASTM C157 (modified for 7 days of cure followed by 28 days of drying) exceeds 0.045 percent (refer to SEAOC [1965, 1979]), reinforcement ratios above the requirements of 12.13.2.1 should be considered. Many designers use shrinkage limits of 0.045 and less to help moderate tensile stresses and control cracking in restrained sections of environmental concrete structures. Where local aggregates make such a limit impractical, the designer should consider alternative means to limit shrinkage cracking.

When shrinkage-compensating concrete is used in accordance with the shrinkage-compensating cement or admixture manufacturer's recommendations, the reinforcement should be at least 0.30 percent for all restraint conditions.

The normal and maximum amount of restraint provisions are related to the distance between adjacent, parallel movement joints and unrestrained ends of elements. Monolithic construction joints are not considered to be movement joints.

12.13.2.2 Reduced restraint locations are as follows:

(a) Wall reinforcement in the vertical direction

(b) Dome shells, except for local regions defined under maximum restraint, constructed monolithically or placed sequentially in segments over a 7-day period with continuous water curing

(c) Any member where effective means to prevent restraint is provided as determined by the licensed design professional

R12.13.2.2 *Reduced restraint*

Walls are considered to have reduced restraint in the vertical direction because their own weight helps to reduce shrinkage and temperature stresses.

Means of reducing restraint for walls in the horizontal (longitudinal or circumferential) direction can include bearing pads or multiple layers of plastic placed on previously placed concrete without dowels. Figure R12.13.2.2 shows an example of one way to temporarily reduce restraint longitudinally or circumferentially in areas of lower seismicity. However, care should be taken to provide resistance to lateral and longitudinal forces, as required, after the initial period of reducing horizontal restraint. The longitudinal or circumferential restraint should remain "reduced" for initial shrinkage and temperature shortening for 28 days or longer, depending on thickness and local environmental conditions such as relative humidity.

Domes without gas pressure are in compression and have reduced potential for temperature and shrinkage strains in areas away from where restraint exists. Where dome segments are kept continuously wet and placed within

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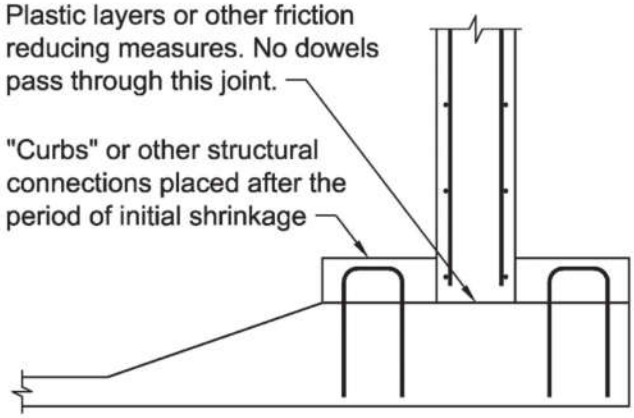

Fig. R12.13.2.2—Concept of longitudinal or circumferential restraint reduction (lower seismicity regions).

a 7-day curing period, shrinkage is limited and restraint between segments is minor.

Where domes are to be gas-tight, the tension from gas pressure reduces or can exceed the gravity dome compression and increased crack control is needed to prevent gas leakage. Therefore, increasing shrinkage and temperature reinforcement in the dome shell should be considered.

"Keying" or downward "haunching" of slabs-on-ground beneath walls and columns increases restraint and should not be used for the reduced restraint condition. Upturned footings are recommended to avoid increasing restraint. Elements that can produce restraint include pipes, sumps, and similar items that penetrate the slab. Restraint can be avoided by providing blockouts, which are placed after significant shrinkage has occurred. A period for reduced restraint for initial shrinkage and temperature shortening of 28 days or longer, taking into account local environmental conditions, should be provided.

12.13.2.3 Normal restraint locations are where neither reduced nor maximum restraint applies.

12.13.2.4 Maximum restraint locations are as follows:

(a) Walls with more than 20 feet between movement joints (as defined in 7.4.1) or to an unrestrained end, for horizontal reinforcement: The first 6 ft above a horizontal monolithic construction joint

(b) Suspended slabs doweled into walls with more than 20 feet between movement joints (as defined in 7.4.1) or to an unrestrained end, for reinforcement parallel to the wall: The first portion of the slab, to 6 ft from the inside face of the wall and from each face of an interior wall

(c) Domes with nonprestressed tension rings, when doweled into walls: Circumferential reinforcement within

R12.13.2.4 *Maximum restraint*

These areas are zones of maximum restraint and require significantly more shrinkage and temperature reinforcement to control crack widths.

When doweled into the circumferential wall, a dome with a nonprestressed ring will require maximum restraint reinforcement within the region of dome tension ring. However, in most cases, the large amount of circumferential reinforcement required to resist the ring tension will exceed the required maximum restraint reinforcement. For a dome with a prestressed compression ring, the potential for restrained shrinkage only occurs while the dome is supported on

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the tension ring, and extending 6 ft from the inside face of wall

(d) Domes with prestressed compression rings, when doweled into walls and the dome is not continuously wet cured until after prestressing is complete: Circumferential reinforcement within the compression ring and extending 6 ft from the inside face of wall

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formwork and left to dry for an extended period before prestressing. By keeping the dome continuously wet until prestressing is applied, the dome compression ring region should not experience maximum restraint shrinkage.

Some designers believe that maximum restraint reinforcement in the area parallel to monolithic construction joints in slabs-on-ground and suspended slabs is desirable to help control cracking and increase liquid tightness.

Slab portions placed between (and doweled into) previously placed portions of the slab ("checkerboard style") are restrained from shrinkage at the monolithic construction joints on multiple sides. It is required to reinforce at least the first 6 ft parallel to previously placed concrete in accordance with the maximum restraint requirements of the code, as illustrated by Fig. R12.13.2.4a.

It should be noted that checkerboard style placement is not recommended. If the designer wishes to reinforce a slab placed checkerboard style, as shown in Fig. R12.13.2.4a, significant coordination will be required between the designer and contractor to get the reinforcement in the correct places.

Walls greater than 20 ft in length or greater than 20 ft between full contraction or expansion joints have maximum restraint when doweled into previously placed concrete. The zone adjacent to the bottom construction joint needs significantly higher shrinkage and temperature reinforcement when reduced crack width is desired. Walls above the maximum restraint zones are considered to have normal restraint.

The bottom 6 ft of walls is required to have twice the required shrinkage and temperature reinforcement.

Suspended slabs have maximum restraint when doweled into previously placed concrete in walls. The zone adjacent to these construction joints needs significantly greater shrinkage and temperature reinforcement when reduced crack width is desired. Refer to Fig. R12.13.2.4b.

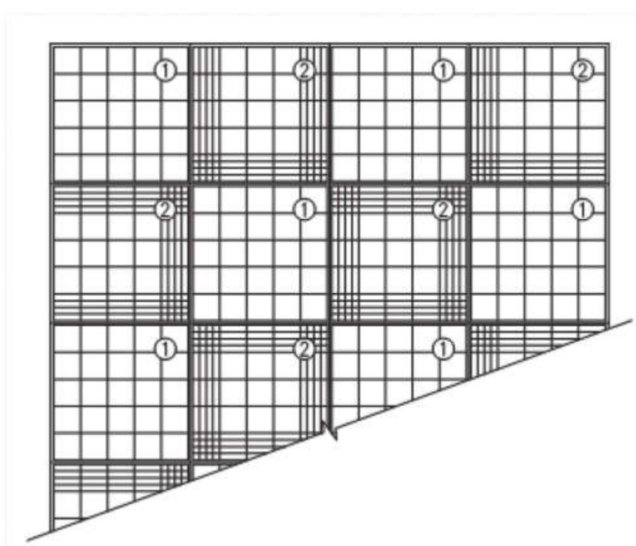

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Fig. R12.13.2.4a—Slab reinforcement at restrained edges.

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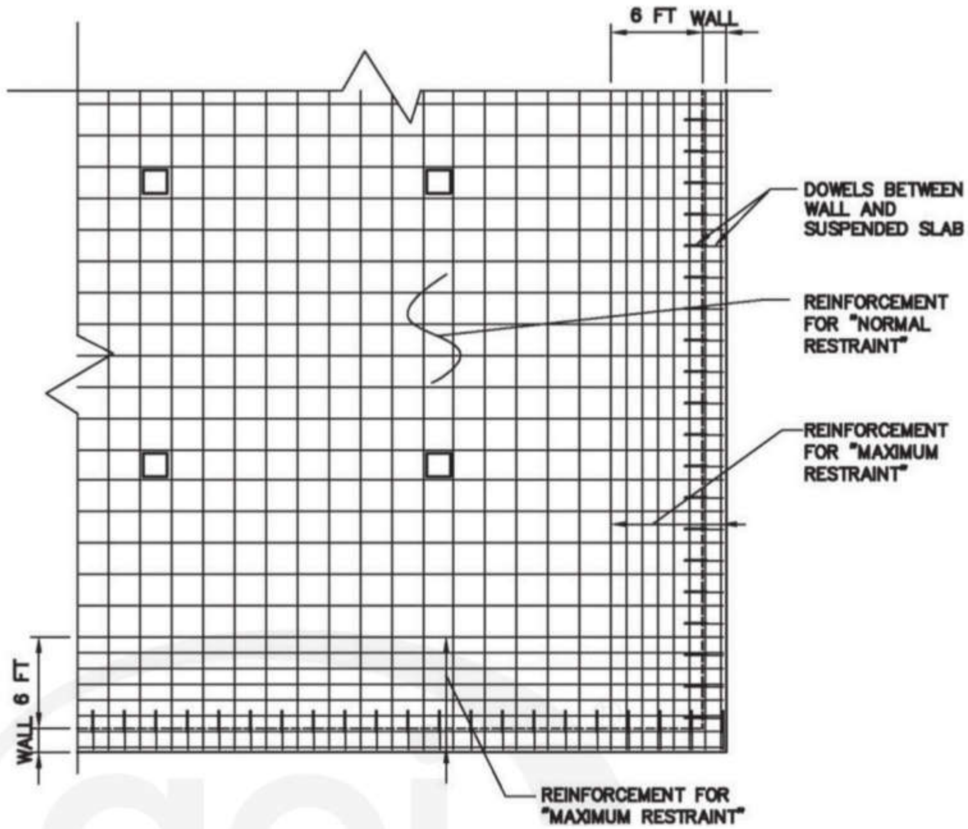

Fig. R12.13.2.4b—Reinforcement for maximum restraint at suspended slabs.

The 6 ft distance for additional reinforcement (as indicated for slabs and walls) is empirical, from observations of seeping cracks on existing structures. The designer should use this amount as a minimum and may want to provide more if past observations of local structures so warrants.

For domes, the transition area of the dome to the perimeter ring beam or wall acting as the tension element causes significant restraint.

Domes that are required to be gas-tight, with internal gas pressure that significantly reduces the dome compression, should be provided with reinforcement for maximum restraint.

12.13.2.5 Shrinkage and temperature reinforcement shall not be spaced farther apart than 12 in.

R12.13.2.5 Smaller bar sizes at closer spacing are considered more effective in controlling the widths of shrinkage cracks than the same ratio of reinforcement.

12.13.2.6 In members with more than one layer of reinforcement, no less than one-third of the total required shrinkage and temperature reinforcement area in each direction shall be distributed to either face.

R12.13.2.6 In members with more than one layer of reinforcement, the shrinkage and temperature reinforcement is normally divided equally between both concrete faces. However, in some cases, one face of the member could be subjected to higher temperature or moisture variations compared to the other face. In such cases, up to two-thirds of the total shrinkage and temperature reinforcement required may be placed on the colder or dryer face of the member.

In prior ACI 350 reports and codes, the reinforcement in the bottom of base slabs in contact with soil was permitted

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to be reduced to 50 percent of that otherwise required. This is no longer permitted. However, distributing the required shrinkage and temperature reinforcement up to two-thirds to the top face and one-third to the bottom face is permitted.

12.13.2.7 Shrinkage and temperature reinforcement for slabs-on-soil, walls, and suspended slabs, more than 24 in. thick need not be greater than that calculated for a 24 in. thick member.

R12.13.2.7 The provision that members greater than 24 in. thick are not required to have more shrinkage and temperature reinforcement than that calculated for a 24 in.-thick member has been a long-standing code provision. Experience has generally shown that basing the shrinkage reinforcement on the outer 12 in. of thicker members is sufficient to control shrinkage and temperature cracking.

If there is concern that a thick member may dry and cool through the interior before sufficient concrete strength has been reached or if higher than recommended thermal gradients during curing are likely, shrinkage and temperature reinforcement may be increased, based on engineering judgment.

12.13.2.8 Shrinkage and temperature reinforcement shall develop the specified yield strength f_y in tension in accordance with 12.8.

12.13.3 Prestressing steel conforming to 3.5.5 used for shrinkage and temperature reinforcement shall be provided in accordance with the following:

12.13.3.1 Tendons shall be proportioned to provide a minimum average compressive stress of 125 psi on gross concrete area using effective prestress, after losses, in accordance with 19.6.

12.13.3.2 For monolithic cast-in-place post-tensioned beam-and-slab construction, gross concrete area of a beam and tributary slab shall consist of the total beam area including the slab thickness and the slab within half the clear distance to adjacent beam webs. It shall be permitted to

R12.13.3 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. This amount of prestressing—125 psi on the gross concrete area—has been successfully used on numerous projects. In monolithic beam-and-slab construction, a minimum of one shrinkage and temperature tendon is required between beams, even if the beam tendons alone provide at least 125 psi average compression stress on the gross concrete area as defined in 12.13.3.2. Any size tendon is permissible as long as all other requirements of 12.13.3 are satisfied. Application of the provisions of 12.13.3.2 to monolithic cast-in-place post-tensioned beam-and-slab construction is illustrated in Fig. R12.13.3a.

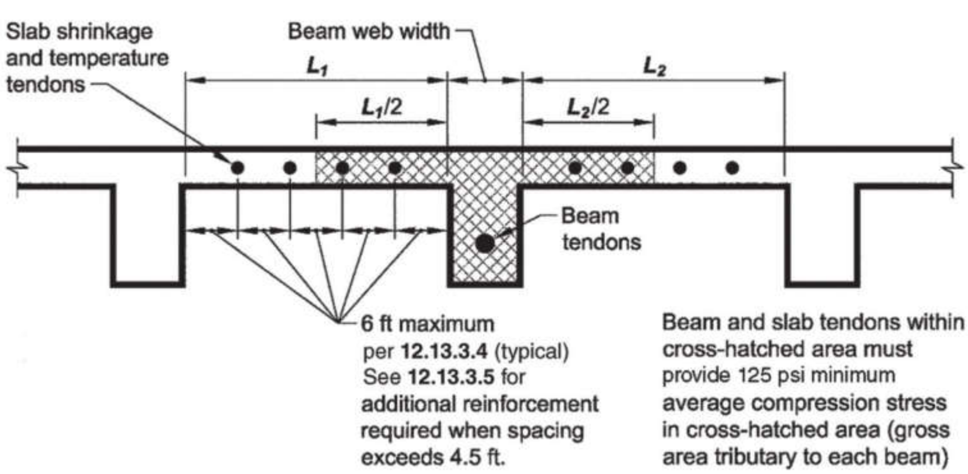

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Fig. R12.13.3a—Section through-beams cast monolithically with slab.

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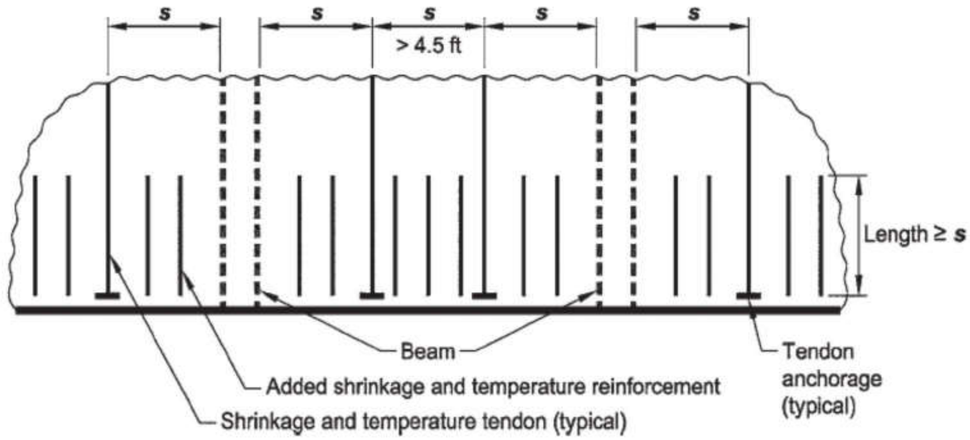

Fig. R12.13.3b—Plan at slab edge showing added shrinkage and temperature reinforcement (refer to 12.13.3.5).

include the effective force in beam tendons in the calculation of total prestress force acting on gross concrete area.

12.13.3.3 Where slabs are supported on walls or not cast monolithically with beams, gross concrete area is the slab section tributary to the tendon or tendon group.

12.13.3.4 In all cases, a minimum of one slab tendon is required between faces of beams or walls. Spacing of slab tendons, and the distance between face of beam or wall to the nearest slab tendon, shall not exceed 6 ft.

12.13.3.5 Where spacing of slab tendons exceeds 4.5 ft, additional nonprestressed shrinkage and temperature reinforcement conforming to 12.13.2 shall be provided between faces of beams or walls, parallel to the slab shrinkage and temperature tendons. This additional shrinkage and temperature reinforcement shall extend from the slab edges for a distance greater than or equal to the tendon spacing, except 12.13.2.8 shall not apply.

Where the spacing of slab tendons used for shrinkage and temperature reinforcement exceeds 4.5 ft, additional nonprestressed reinforcement is required to extend from slab edges where the prestressing forces are applied to adequately reinforce the area between the slab edge and the point where compressive stresses behind individual anchorages have spread sufficiently such that the slab is uniformly in compression (refer to Fig. R12.13.3b).

Tendons used for shrinkage and temperature reinforcement should be positioned as closely as practicable to middepth of the slab. In cases where the shrinkage and temperature tendons are used for supporting the principal tendons, variations from the slab centroid are permissible; however, the resultant of the shrinkage and temperature tendons should not fall outside the kern area of the slab.

The effects of slab shortening should be evaluated to ensure proper action. In most cases, the low level of prestressing recommended should not cause difficulties in a properly detailed member. Additional attention may be required where thermal effects become significant.

12.14—Requirements for structural integrity

R12.14—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.

12.14.1 In the detailing of reinforcement and connections, members of a structure shall be effectively tied together to improve integrity of the overall structure.

R12.14.1 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.

12.14.2 For cast-in-place construction, the following shall constitute minimum requirements:

R12.14.2 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.

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12.14.2.1 In joist construction, as defined in 8.13.1 through 8.13.5, at least one bottom bar shall be continuous or shall be spliced with a Class B tension splice or a mechanical or welded splice satisfying 12.9.1.3 and at noncontinuous supports shall be anchored to develop f_y at the face of the support using a standard hook satisfying 12.8.5 or headed deformed bar satisfying 12.8.6.

12.14.2.2 Beams along the perimeter of the structure shall have continuous reinforcement over the span length passing through the region bounded by the longitudinal reinforcement of the column 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-fourth of the tension reinforcement required for positive moment at midspan, but not less than two bars.

At noncontinuous supports, the reinforcement shall be anchored to develop f_y at the face of the support using a standard hook satisfying 12.8.5 or headed deformed bar satisfying 12.8.6.

12.14.2.3 The continuous reinforcement required in 12.14.2.2 shall be enclosed by transverse reinforcement of the type specified in 11.5.4.1. The transverse reinforcement shall be anchored as specified in 11.5.4.2. The transverse reinforcement need not extend through the column.

12.14.2.4 Where splices are required to satisfy 12.14.2.2, 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 B tension splices, or mechanical or welded splices satisfying 12.9.1.3.

12.14.2.5 In other than perimeter beams, where transverse reinforcement as defined in 12.14.2.3 is provided, there are no additional requirements for longitudinal integrity reinforcement. Where such transverse reinforcement is not provided, at least one-fourth of the positive moment reinforcement required at midspan, but not less than two bars, shall pass through the region bounded by the longitudinal reinforcement of the column and shall be continuous or shall be spliced over or near the support with a Class B tension splice, or a mechanical or welded splice satisfying 12.9.1.3. At noncontinuous supports, the reinforcement shall be anchored to develop f_y at the face of the support using a standard hook satisfying 12.8.5 or a headed deformed bar satisfying 12.8.6.

12.14.2.6 For nonprestressed two-way slab construction, refer to 14.3.8.5.

12.14.2.7 For prestressed two-way slab construction, refer to 19.12.6 and 19.12.7.

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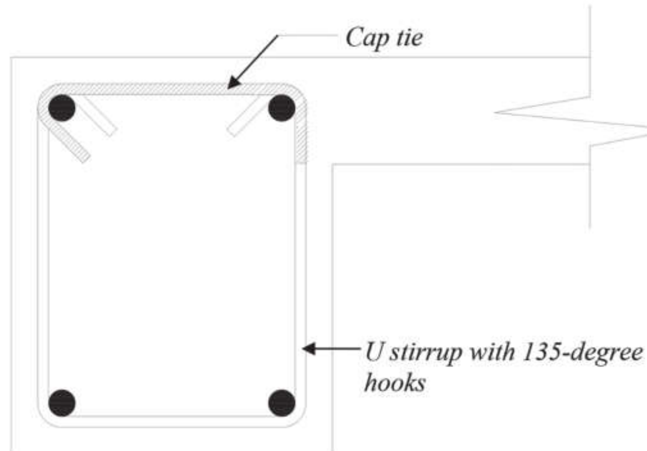

Fig. R12.14.2—Example of a two-piece stirrup that complies with the requirements of 12.14.2.3.

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 12.8.12.3 be further extended and spliced at or near midspan. Similarly, the bottom reinforcement required to extend into the support by 12.8.11.1 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 ACI 350-06 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 12.14.2.3 was revised to require U-stirrups with not less than 135-degree hooks around the continuous bars, or one-piece closed stirrups to prevent the top continuous bars from tearing out of the top of the beam. Section 12.14.2 was revised in ACI 350-06 to require that the transverse reinforcement used to enclose the continuous reinforcement be of the type specified in 11.6.4.1 and anchored according to 11.6.4.2. Figure R12.14.2 shows an example of a two-piece stirrup that satisfies these requirements. Pairs of U-stirrups lapping one another as defined in 12.8.13.5 are not permitted in perimeter or spandrel beams. In the event of damage to the side concrete cover, the stirrups and top longitudinal reinforcement may tend to tear out of the concrete. Thus, the top longitudinal reinforcement will not provide the catenary action needed to bridge over a damaged region. Further, lapped U-stirrups will not be effective at high torque; refer to R11.6.4.1.

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Lap splices were changed from Class A to Class B to provide similar strength to that provided by mechanical and welded splices satisfying 12.9.1.3. Class B lap splices provide a higher level of reliability for abnormal loading events.

12.14.3 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 17.5 shall apply.

R12.14.3 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 by 17.5.1.4.

Connection details should be arranged to minimize the potential for cracking due to restrained creep, shrinkage, and temperature movements. For information on connections and detailing requirements, refer to PCI (1988).

PCI Building Code Committee (1986) recommends minimum tie requirements for precast concrete bearing wall buildings.

12.14.4 For lift-slab construction, refer to 14.3.8.6 and 19.12.8.

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CHAPTER 13—EARTHQUAKE-RESISTANT STRUCTURES

13.1—General requirements

13.1.1 *Scope*

13.1.1.1 Chapter 13 contains requirements for design and construction of reinforced concrete members of a structure to resist earthquake forces.

COMMENTARY

CHAPTER R13—EARTHQUAKE-RESISTANT STRUCTURES

R13.1—General requirements

R13.1.1 *Scope*

Chapter 13 contains 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.

These provisions are 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 or serviceability. The integrity of the structure in the inelastic range of response should be maintained because the design earthquake forces defined in documents such as ASCE/SEI 7, the IBC 2006, the UBC 1997, and the FEMA P-749 provisions are considered less than those corresponding to linear response at the anticipated earthquake intensity (Blume et al. 1961; Clough 1960; Gulkan and Sozen 1974).

For liquid-containing structures, lateral design forces are normally calculated based on a linearly elastic model of the uncracked section. In reality, these models are not always completely accurate, as most reinforced concrete 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. For liquid-containing structures, serviceability considerations preclude significant excursions into the nonlinear range under unfactored loads. Accordingly, response modification coefficient (***R***) factors must be selected that, in combination with the load and strength reduction factors, should keep the structure close to the elastic range and maintain its serviceability under actual seismic loads (Hirosawa 1977).

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 (Nilsson and Losberg 1976). 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.

As a properly detailed cast-in-place or precast concrete structure responds to strong ground motion, its effective stiffness decreases and its energy dissipation increases. These changes tend to reduce the response accelerations and lateral inertia forces relative to values that would occur were the structure to remain linearly elastic and lightly damped (Gulkan and Sozen 1974). Thus, the use of design forces

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representing earthquake effects such as those in ASCE/SEI 7 requires that the seismic-force-resisting system retain a substantial portion of its strength into the inelastic range under displacement reversals.

The provisions of Chapter 13 relate detailing requirements to type of structural framing and Seismic Design Category (SDC). SDCs are adopted directly from ASCE/SEI 7 and relate to considerations of seismic hazard level, soil type, occupancy, and use. Before ACI 318-08, low, intermediate, and high seismic risk designations were used to delineate detailing requirements. For a qualitative comparison of SDCs and seismic risk designations, refer to Table R1.1.9.1. The assignment of a structure to an SDC is regulated by the legally adopted general building code of which this Code forms a part (1.1.9).

The design and detailing requirements should be compatible with the level of energy dissipation (or toughness) assumed in the computation of the design earthquake forces. 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 assigned to higher SDCs possess a higher degree of toughness. It is permitted, however, to design for higher toughness in the lower SDCs and take advantage of the lower design force levels.

The provisions of Chapters 1 to 12 and 14 to 22 are considered adequate for most structures assigned to SDC A (corresponding to lowest seismic hazard). For structures assigned to SDC B, additional requirements apply.

Structures assigned to SDC C may be subjected to moderately strong ground shaking. The designated seismic-force-resisting system typically comprises some combination of ordinary cast-in-place structural walls, intermediate precast structural walls, and intermediate moment frames. The legally adopted general building code of which this Code forms a part also may contain provisions for use of other seismic-force-resisting systems in SDC C. Section 13.1.1.7 defines requirements for whatever system is selected.

Structures assigned to SDC D, E, or F may be subjected to strong ground shaking. It is the intent of ACI Committee 350 that the seismic-force-resisting system of environmental engineering concrete structures assigned to SDC D, E, or F be provided by special moment frames, special structural walls, or a combination of the two. In addition to Sections 13.1.2 through 13.1.8, these structures also are required to satisfy requirements for continuous inspection (13.5), diaphragms and trusses (13.11), foundations (13.12), and gravity-load-resisting elements that are not designated as part of the seismic-force-resisting system (13.13). These provisions have been developed to provide the structure with adequate toughness for the high demands expected for these SDCs.

The legally adopted general building code of which this Code forms a part may also permit the use of intermediate moment frames as part of dual systems for some build-

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ings assigned to SDC D, E, or F. It is not the intention of ACI Committee 350 to recommend the use of intermediate moment frames as part of moment-resisting frame or dual systems in SDC D, E, or F. The legally adopted general building code may also permit substantiated alternative or non-prescriptive designs, or, with various supplementary provisions, the use of ordinary or intermediate systems for nonbuilding structures in the higher SDCs. These are not the typical applications around which this chapter is written, but wherever the term ordinary or intermediate moment frame is used in reference to reinforced concrete, 13.2 or 13.3 apply.

Table R13.1.1 summarizes the applicability of the provisions of Chapter 13 as they are typically applied where using minimum requirements in the various SDCs. Where special systems are used for structures in SDC B or C, it is not required to satisfy the requirements of 13.13, although it should be verified that members not designated as part of the seismic-force-resisting system will be stable under design displacements.

The proportioning and detailing requirements in Chapter 13 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. The acceptance criteria for moment frames given in ACI 374.1 can be used in conjunction with Chapter 13 to demonstrate that the strength and toughness of a proposed frame system equals or exceeds that provided by a comparable monolithic concrete system. ACI ITG-5.1 provides similar information for precast wall systems.

The toughness requirements in 13.1.1.8 refer to the concern for the structural integrity of the entire seismic-force-resisting system at lateral displacements anticipated for ground motions corresponding to the design earthquake.

Table R13.1.1—Sections of Chapter 13 to be satisfied in typical applications*

Component resisting earthquake effect, unless otherwise noted	Seismic Design Category			
	A (None)	B (13.1.1.4)	C (13.1.1.5)	D, E, F (13.1.1.6)
Analysis and design requirements	None	13.1.2	13.1.2	13.1.2, 13.1.3
Materials		None	None	13.1.4 to 13.1.7
Moment frames		13.2	13.3	13.5, 13.6, 13.7, 13.8
Structural walls and coupling beams		None	None	13.9
Precast structural walls		None	13.4	13.4†, 13.10
Structural diaphragms and trusses		None	None	13.11
Foundations		None	None	13.12
Frame members not proportioned to resist forces induced by earthquake motions		None	None	13.13
Anchors		None	13.1.8	13.1.8

*The requirements of Chapters 1 through 12 and 14 through 20, except as modified by Chapter 13 also applies in SDC D, E, and F.

†As permitted by the legally adopted general building code of which this Code forms a part.

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13.1.1.2 All structures shall be assigned to a Seismic Design Category (SDC) in accordance with 1.1.9.1. Liquid-containing structures assigned to SDC C, D, E, or F shall be designed by lateral force procedures prescribed in ACI 350.3. Liquid-containing structures with an L_T/H_L or D_T/H_L ratio less than 2.0 assigned to SDC B and all pedestal-mounted tanks assigned to SDC B shall be designed by lateral force procedures prescribed in ACI 350.3.

13.1.1.3 All members shall satisfy requirements of Chapters 1 to 12 and 14 to 20. Structures assigned to SDC B, C, D, E, or F also shall satisfy 13.1.1.4 through 13.1.1.8, as applicable.

13.1.1.4 Structures assigned to SDC B shall satisfy 13.1.2.

13.1.1.5 Structures assigned to SDC C shall satisfy 13.1.2 and 13.1.8.

13.1.1.6 Structures assigned to SDC D, E, or F shall satisfy 13.1.2, 13.1.8, and 13.11 through 13.13.

13.1.1.7 Structural systems designated as part of the seismic-force-resisting system shall be restricted to those designated by the legally adopted general building code of which this Code forms a part, or determined by other authority having jurisdiction in areas without a legally adopted building code. Except for SDC A, for which the following provisions do not apply, the following provisions shall be satisfied for each structural system designated as part of the seismic-force-resisting system, regardless of the SDC:

(a) Ordinary moment frames shall satisfy 13.2
(b) Intermediate moment frames shall satisfy 13.3
(c) Intermediate precast walls shall satisfy 13.4
(d) Special moment frames shall satisfy 13.5 through 13.8
(e) Special structural walls shall satisfy 13.9
(f) Special precast structural walls shall satisfy 13.10

All special moment frames and special structural walls shall also satisfy 13.1.3 through 13.1.7. Horizontal frames and trusses providing lateral support for walls of liquid-containing structures are not required to meet the provisions for special moment resisting frames. Walls of liquid-containing structures with in-plane factored shear force less than $2A_{cv}\sqrt{f_c'}$ are not required to meet the provisions for special structural walls.

13.1.1.8 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, toughness, and liquid-tightness equal to or exceeding those provided by a

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Depending on the energy-dissipation characteristics of the structural system used, such displacements may be larger than for a monolithic reinforced concrete structure.

R13.1.1.2 Lateral-force procedures in general building 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 greater than 2.0 for SDC B.

R13.1.1.7 Horizontal frames and trusses providing lateral support for walls of liquid-containing structures are not required to meet the provisions for special moment resisting frames for the following reasons:

(a) Because of the relatively low value of R (response modification coefficient) 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 walls of liquid-containing structures
(c) Because the horizontal frame or truss is designed to resist the full lateral load from 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

Walls of liquid-containing structures 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 walls of liquid-containing structures. For in-plane factored shear forces below this level, shear is resisted by concrete alone. Therefore, additional reinforcement above Section 12.13 requirements and special development and splices are not required

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comparable monolithic reinforced concrete structure satisfying this chapter.

13.1.1.9 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.

13.1.1.10 Liquid-containing structures subjected to earthquake-induced forces shall be designed in accordance with the strength design method or the alternate design method (Appendix A) 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.

(b) When a liquid-containing structure is designed in accordance with the alternate design method, allowable stresses in Appendix A shall be permitted to be increased by one-third, when permitted by the legally adopted general building code.

13.1.2 *Analysis and proportioning of structural members*

13.1.2.1 The interaction of all structural and nonstructural members that affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

13.1.2.2 Rigid members assumed not to be a part of the seismic-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 that are not a part of the seismic-force-resisting system shall be considered.

COMMENTARY

R13.1.1.10 For a classification of liquid-containing structures, refer to ACI 350.3.

Liquid-containing structures founded at or below grade are nonbuilding structures as defined in Park and Thompson (1977) 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 $\boldsymbol{S_d}$ may be taken as 1.0 for load combinations with earthquake effect $\boldsymbol{E}$ in accordance with 9.2.7. Earthquake loading is 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.

Alternatively, if the potential for even minor damage to the structure is to be minimized, the licensed design professional may select the applicable environmental durability factor $\boldsymbol{S_d}$ based on the definitions in 9.2.6.

R13.1.2 *Analysis and proportioning of structural members*

It is assumed that the distribution of required strength to the various components of a seismic-force-resisting system will be guided by the analysis of a linearly elastic model of the system acted upon by the factored forces required by the legally adopted general building code. If nonlinear response history analyses 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 earthquake admits nonlinear response, it is necessary to investigate the stability of the seismic-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

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13.1.2.3 Structural members extending below the base of structure that are required to transmit forces resulting from earthquake effects to the foundation shall comply with the requirements of Chapter 13 that are consistent with the seismic-force-resisting system above the base of structure.

13.1.2.4 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 earthquake forces, shears, and moments shall take into account the seismicity of the area, the overall ductility and energy-dissipating capacity of the structure (response modification coefficient $\boldsymbol{R}$), the importance of the structure, and the dynamic amplification factors.

13.1.3 *Strength reduction factors for special moment frames and special structural walls*

Strength reduction factors shall be as given in 9.3.4.

13.1.4 *Concrete in special moment frames and special structural walls*

13.1.4.1 Requirements of 13.1.4 apply to special moment frames, special structural walls, and all components of special structural walls including coupling beams and wall piers.

13.1.4.2 Specified compressive strength of concrete, f_c', shall be governed by Chapter 4 as applicable.

13.1.4.3 Specified compressive strength of lightweight concrete, f_c', shall not exceed 5000 psi unless demonstrated by experimental evidence that structural members made with that lightweight concrete provide strength and toughness equal to or exceeding those of comparable members made with normalweight concrete of the same strength. Modification factor λ for lightweight concrete in this chapter shall be in accordance with 8.6.1 unless specifically noted otherwise.

13.1.5 *Reinforcement in special moment frames and special structural walls*

COMMENTARY

analysis, one option is to multiply by a factor of at least two the displacements from linear analysis by using the factored lateral forces, unless the legally adopted general building code specifies the factors to be used as in the IBC or the UBC. 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 analysis assumptions described in 8.8 also may be used to estimate lateral deflections of reinforced concrete structural systems.

The main objective of Chapter 13 is the safety of the structure. The intent of 13.1.2.1 and 13.1.2.2 is to draw attention to the influence of nonstructural members on structural response and to hazards from falling objects.

Section 13.1.2.3 serves as an alert that the base of structure as defined in analysis may not necessarily correspond to the foundation or ground level. Details of columns and walls extending below the base of structure to the foundation are required to be consistent with those above the base of structure.

In selecting member sizes for earthquake-resistant structures, it is important to consider constructibility problems related to congestion of reinforcement. The design should be such that all reinforcement can be assembled and placed in the proper location 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.

R13.1.4 *Concrete in special moment frames and special structural walls*

Requirements of this section refer to concrete quality of structures that resist earthquake-induced forces. The maximum specified compressive strength of lightweight 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 members made with lightweight concrete subjected to displacement reversals in the nonlinear range. If convincing evidence is developed for a specific application, the limit on maximum specified compressive strength of lightweight concrete may be increased to a level justified by the evidence.

R13.1.5 *Reinforcement in special moment frames and special structural walls*

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13.1.5.1 Requirements of 13.1.5 apply to special moment frames, special structural walls, and all components of special structural walls including coupling beams and wall piers.

13.1.5.2 Deformed reinforcement resisting earthquake-induced flexure, axial force, or both, shall comply with ASTM A706 Grade 60. ASTM A615 Grades 40 and 60 reinforcement shall be permitted in these members if:

(a) The actual yield strength based on mill tests does not exceed f_y by more than 18,000 psi; and

(b) The ratio of the actual tensile strength to the actual yield strength is not less than 1.25.

13.1.5.3 Prestressing steel resisting earthquake-induced flexural and axial loads in frame members and in precast structural walls shall comply with ASTM A416 or A722.

13.1.5.4 The value of f_{yt} used to compute the amount of confinement reinforcement in 13.6.4.4 shall not exceed 100,000 psi.

13.1.5.5 The values of f_y or f_{yt} used in design of shear reinforcement shall conform to 11.4.2.

13.1.6 *Mechanical splices in special moment frames and special structural walls*

13.1.6.1 Mechanical splices shall be classified as either Type 1 or Type 2 mechanical splices, as follows:

(a) Type 1 mechanical splices shall conform to 12.9.1.3.2

(b) Type 2 mechanical splices shall conform to 12.9.1.3.2 and shall develop the specified tensile strength of the spliced bar

13.1.6.2 Type 1 mechanical splices shall not be used within a distance equal to twice the member depth from the column or beam face for special moment frames or from sections where yielding of the reinforcement is likely to occur as a

COMMENTARY

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 (refer to 13.1.5.2(a)). ASTM A706 for low-alloy steel reinforcing bars now includes both Grade 60 and Grade 80; however, only Grade 60 is generally permitted because of insufficient data to confirm applicability of existing Code provisions for structures using the higher grade. Section 13.1.1.8 permits alternative material such as ASTM A706 Grade 80 if results of tests and analytical studies are presented in support of its use.

The requirement for a tensile strength larger than the yield strength of the reinforcement (13.1.5.2(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 nominal and yield moments (ACI 352R). According to this interpretation, the larger the ratio of nominal to yield moment, the longer the yield region. Chapter 13 requires that the ratio of 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.

The restrictions on the values of f_y and f_{yt} apply to all types of transverse reinforcement, including spirals, circular hoops, rectilinear hoops, and crossties. The restrictions on the values of f_y and f_{yt} in 11.4.2 for computing nominal shear strength are intended to limit the width of shear cracks. Research results (Budek et al. 2002; Muguruma and Watanabe 1990; Sugano et al. 1990) indicate that higher yield strengths can be used effectively as confinement reinforcement as specified in 13.6.4.4.

R13.1.6 *Mechanical splices in special moment frames and special structural walls*

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 12.9.1.3.2. The restriction on Type 1 splices applies to all reinforcement resisting earthquake effects, including

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result of inelastic lateral displacements. Type 2 mechanical splices shall be permitted to be used at any location.

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transverse reinforcement. 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, there should be 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.

13.1.7 *Welded splices in special moment frames and special structural walls*

R13.1.7 *Welded splices in special moment frames and special structural walls*

13.1.7.1 Welded splices in reinforcement resisting earthquake-induced forces shall conform to 12.9.1.3.4 and shall not be used within a distance equal to twice the member depth from the column or beam face for special moment frames or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements.

R13.1.7.1 Welding of reinforcement should be according to 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 12.9.1.3.4. The restriction on welded splices applies to all reinforcement resisting earthquake effects, including transverse reinforcement.

13.1.7.2 Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement that is required by design shall not be permitted.

R13.1.7.2 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.

13.1.8 *Anchoring to concrete*

Anchors resisting earthquake-induced forces in structures assigned to SDC C, D, E, or F shall conform to the requirements of E.3.3.

13.2—Ordinary moment frames

13.2.1 *Scope*

Requirements of 13.2 apply to ordinary moment frames forming part of the seismic-force-resisting system.

13.2.2 Beams shall have at least two of the longitudinal bars continuous along both the top and bottom faces. These bars shall be developed at the face of support.

13.2.3 Columns having clear height less than or equal to five times the dimension c_1 shall be designed for shear in accordance with 13.3.3.2.

R13.2—Ordinary moment frames

These provisions were introduced in ACI 318-08 and apply only to ordinary moment frames assigned to SDC B. The provisions for beam reinforcement are intended to improve continuity in the framing members as compared with the provisions of Chapters 1 through 12 and 14 through 19 and thereby improve lateral force resistance and structural integrity; these provisions do not apply to slab-column moment frames. The provisions for columns are intended to provide additional toughness to resist shear for columns with proportions that would otherwise make them more susceptible to shear failure under earthquake loading.

13.3—Intermediate moment frames

13.3.1 *Scope*

Requirements of 13.3 apply to intermediate moment frames forming part of the seismic-force-resisting system.

13.3.2 Reinforcement details in a frame member shall satisfy 13.3.4 if the factored axial compressive load, P_u, for the member does not exceed $A_g f_c'/10$. If P_u is larger

R13.3—Intermediate moment frames

The objective of the requirements in 13.3.3 is to reduce the risk of failure in shear in beams and columns during an earthquake. Two options are provided to determine the factored shear force.

According to 13.3.3.1(a) and 13.3.3.2(a), the factored shear force is determined from a free-body diagram obtained by cutting through the member ends, with end moments

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reinforcement details shall satisfy 13.3.5. Where a two-way slab system without beams forms a part of the seismic-force-resisting system, reinforcement details in any span resisting moments caused by $\boldsymbol{E}$ shall satisfy 13.3.6.

13.3.3 *Shear strength*

13.3.3.1 ϕV_n of beams and columns resisting earthquake effect $\boldsymbol{E}$ shall not be less than the smaller of (a) and (b):

(a) The sum of the shear associated with development of M_n of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity loads

(b) The maximum shear obtained from design load combinations that include $\boldsymbol{E}$, with $\boldsymbol{E}$ assumed to be twice that prescribed by the legally adopted general building code for earthquake-resistant design.

13.3.3.2 ϕV_n of columns resisting earthquake effect $\boldsymbol{E}$ shall not be less than the smaller of (a) and (b):

(a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength

(b) The maximum shear obtained from design load combinations that include $\boldsymbol{E}$, with $\boldsymbol{E}$ increased by Ω_o.

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assumed equal to the nominal moment strengths acting in reverse curvature bending. Examples for a beam and a column are illustrated in Fig. R13.3.3. In all applications of 13.3.3.1(a) and 13.3.3.2(a), shears are required to be calculated for moments due to reverse curvature bending, acting both clockwise and counterclockwise. Figure R13.3.3 demonstrates only one of the two options that are to be considered for every member. The factored axial force P_u

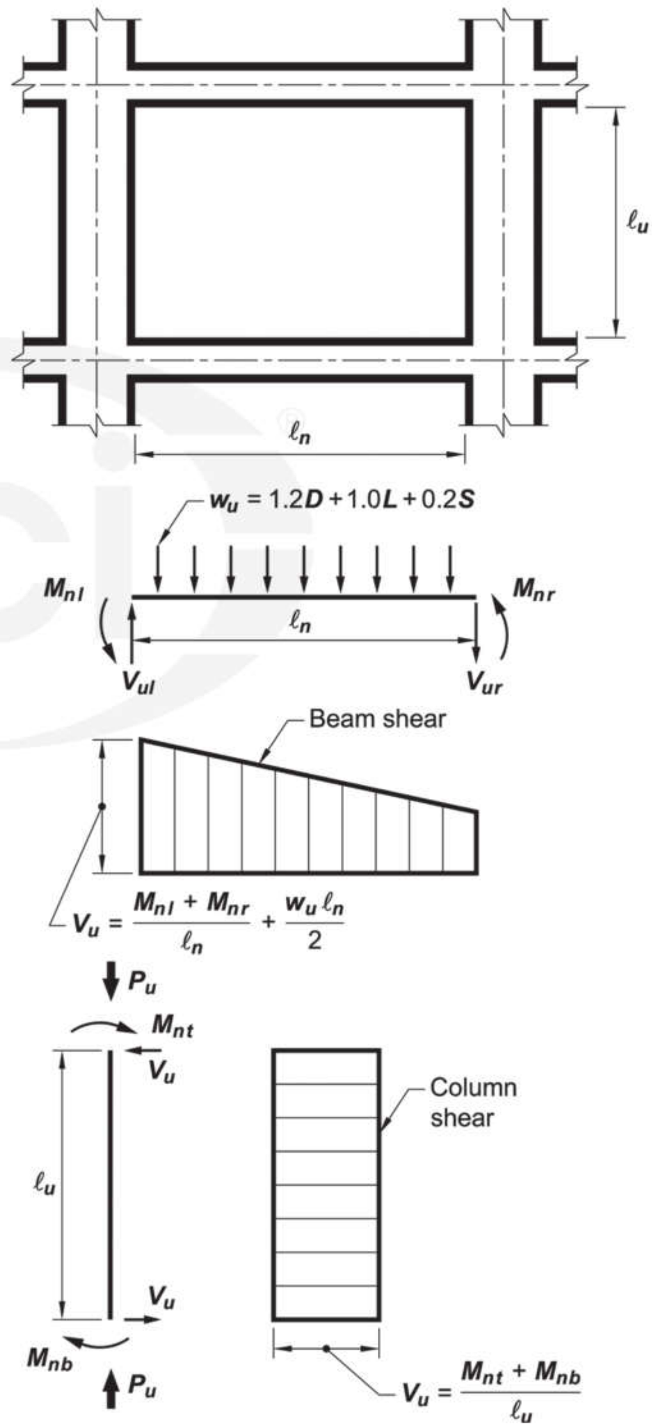

Fig. R13.3.3—Design shears for intermediate moment frames.

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should be chosen to develop the largest moment strength of the column.

To determine the maximum beam shear, it is assumed that its nominal moment strengths (**ϕ = 1.0**) are developed simultaneously at both ends of its clear span. As indicated in Fig. R13.3.3, the shear associated with this condition $[(M_{nl} + M_{nr})/\ell_n]$ is added algebraically to the shear due to the factored gravity loads to obtain the design shear for the beam. For the example shown, both the dead load w_D and the live load w_L have been assumed to be uniformly distributed. Effects of E acting vertically are to be included if required by the general building code.

Option 13.3.3.1(b) for beams 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

$$U = 1.2D + 1.2\mathrm{F} + 2.0E + 1.6\mathrm{H} + 1.0L + 0.2S$$

where E is the value specified by the governing code. The factor 1.0 applied to L is allowed to be reduced to 0.5 in accordance to 9.2.1(a).

Option 13.3.3.2(b) for columns is similar to that for beams except that it bases V_u on load combinations including the earthquake effect E, with E increased by the overstrength factor Ω_o rather than the factor 2.0. In ASCE/SEI 7, Ω_o = **3.0** for intermediate moment frames. The higher factor for columns relative to beams is because of greater concerns about shear failure in columns.

Section 13.3.4 contains requirements for providing beams with a threshold level of toughness. Transverse reinforcement at the ends of the beam is required to be hoops. In most cases, stirrups required by 13.3.3 for design shear force will be more than those required by 13.3.4. Requirements of 13.3.5 serve the same purpose for columns.

Discontinuous structural walls and other stiff members can impose large axial forces on supporting columns during earthquakes. The required transverse reinforcement in 13.3.5.6 is to improve column toughness under anticipated demands. The factored axial compressive force related to earthquake effect should include the factor Ω_o if required by the legally adopted general building code of which this Code forms a part.

Section 13.3.6 applies to two-way slabs without beams, such as flat plates.

Using load combinations of Eq. (9-5) and (9-7) may result in moments requiring top and bottom reinforcement at the supports.

The moment M_{slab} 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 14.5.3.2, only a fraction of the moment M_{slab} is assigned to the slab effective width. For edge and corner connections, flexural

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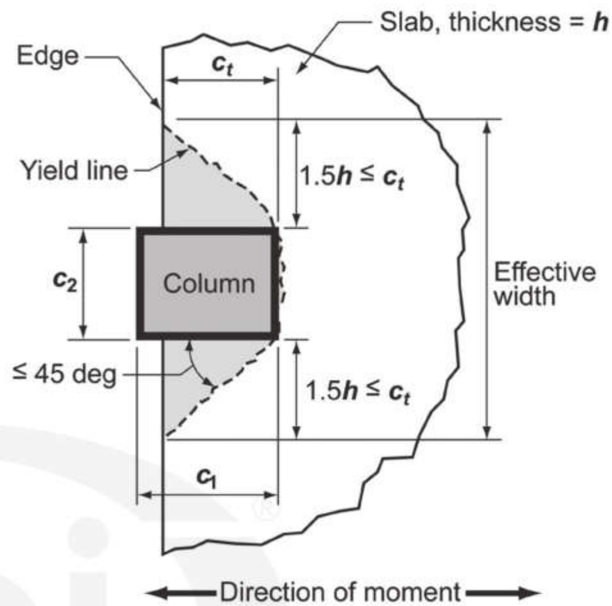
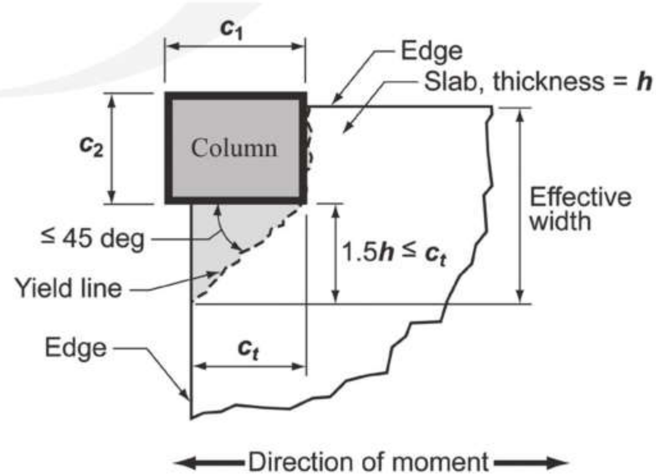
reinforcement perpendicular to the edge is not considered fully effective unless it is placed within the effective slab width (ACI 352.1R; Pan and Moehle 1989). Refer to Fig. R13.3.6.1.

Application of the provisions of 13.3.6 is illustrated in Fig. R13.3.6.2 and R13.3.6.3.



(a) Edge connection



(b) Corner connection

Fig. R13.3.6.1—Effective width for reinforcement placement in edge and corner connections.

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13.3.4 *Beams*

13.3.4.1 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 beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

13.3.4.2 At both ends of the beam, hoops shall be provided over lengths not less than $2h$ measured from the face of the supporting member toward midspan. The first hoop shall be located not more than 2 in. from the face of the supporting member. Spacing of hoops shall not exceed the smallest of (a), (b), (c), and (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.

13.3.4.3 Stirrups shall be spaced not more than $d/2$ throughout the length of the beam.

13.3.5 *Columns*

13.3.5.1 Columns shall be spirally reinforced in accordance with 12.10.1.4 or shall conform with 13.3.5.2 through 13.3.5.4. Section 13.3.5.5 shall apply to all columns, and 13.3.5.6 shall apply to all columns supporting discontinuous stiff members.

13.3.5.2 At both ends of the column, 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 column

(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 column

(f) Maximum cross-sectional dimension of the column

(g) 18 in.

13.3.5.3 The first hoop shall be located not more than $s_o/2$ from the joint face.

13.3.5.4 Outside the length ℓ_o, spacing of transverse reinforcement shall conform to 12.10.1 and 11.4.5.1.

13.3.5.5 Joint transverse reinforcement shall conform to 11.10.

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13.3.5.6 Columns supporting reactions from discontinuous stiff members, such as walls, shall be provided with transverse reinforcement at the spacing s_o as defined in 13.3.5.2 over the full height beneath the level at which the discontinuity occurs if the portion of factored axial compressive force in these members related to earthquake effects exceeds $A_g f_c'/10$. Where design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $A_g f_c'/10$ shall be increased to $A_g f_c'/4$. This transverse reinforcement shall extend above and below the columns as required in 13.6.4.6(b).

13.3.6 *Two-way slabs without beams*

13.3.6.1 Factored slab moment at support including earthquake effects $\boldsymbol{E}$ shall be determined for load combinations given in Eq. (9-5) and (9-7). Reinforcement provided to resist M_{slab} shall be placed within the column strip defined in 14.2.1.

13.3.6.2 Reinforcement placed within the effective width specified in 14.5.3.2 shall be proportioned to resist $\gamma_f M_{slab}$. 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.

13.3.6.3 Not less than one-half of the reinforcement in the column strip at support shall be placed within the effective slab width given in 14.5.3.2.

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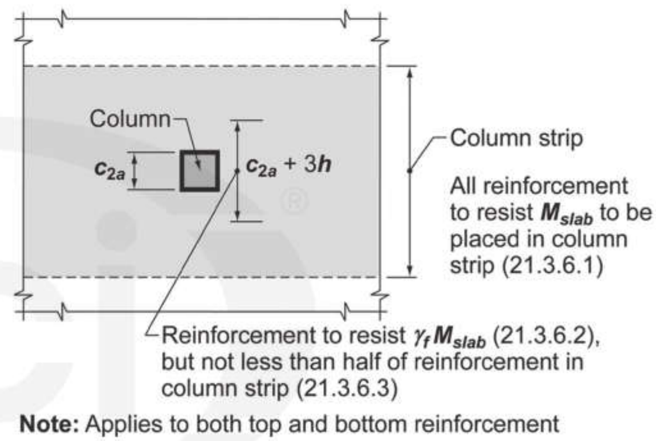

Fig. R13.3.6.2—Location of reinforcement in slabs.

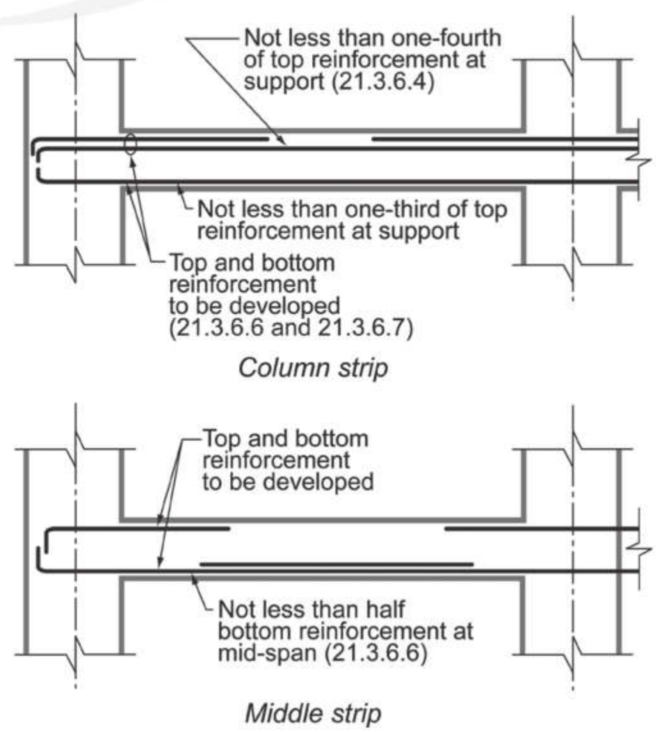

Fig. R13.3.6.3—Arrangement of reinforcement in slabs.

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13.3.6.4 Not less than one-fourth of the top reinforcement at the support in the column strip shall be continuous throughout the span.

13.3.6.5 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.

13.3.6.6 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 f_y at face of support as defined in 14.6.2.5.

13.3.6.7 At discontinuous edges of the slab all top and bottom reinforcement at support shall be developed at the face of support as defined in 14.6.2.5.

13.3.6.8 At the critical sections for columns defined in 11.11.1.2, 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 11.11.2.1 for nonprestressed slabs and in 11.11.2.2 for prestressed slabs. It shall be permitted to waive this requirement if the slab design satisfies requirements of 13.13.6.

13.4—Intermediate precast structural walls

13.4.1 *Scope*

Requirements of 13.4 apply to intermediate precast structural walls forming part of the seismic-force-resisting system.

13.4.2 In connections between wall panels, or between wall panels and the foundation, yielding shall be restricted to steel elements or reinforcement.

13.4.3 Elements of the connection that are not designed to yield shall develop at least $1.5S_y$.

13.4.4 In structures assigned to SDC D, E, or F, wall piers shall be designed in accordance with 13.9 or 13.13.

13.5—Flexural members of special moment frames

13.5.1 *Scope*

Requirements of 13.5 apply to special moment frame members that form part of the seismic-force-resisting system and are proportioned primarily to resist flexure. These frame members shall also satisfy the conditions of 13.5.1.1 through 13.5.1.4.

13.5.1.1 Factored axial compressive force on the member, P_u, shall not exceed $A_g f_c'/10$.

13.5.1.2 Clear span for member, ℓ_n, shall not be less than four times its effective depth.

COMMENTARY

R13.3.6.8 The requirements apply to two-way slabs that are designated part of the seismic-force-resisting system. Slab-column connections in laboratory tests (Pan and Moehle 1989) exhibited reduced lateral displacement ductility when the shear at the column connection exceeded the recommended limit. Slab-column connections also must satisfy shear and moment strength requirements of Chapters 11 and 14 under load combinations including earthquake effect.

13.4—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.

R13.5—Flexural members of special moment frames

R13.5.1 *Scope*

This section refers to beams of special moment frames resisting lateral loads induced by earthquake motions. Any frame member subjected to a factored axial compressive force exceeding ($A_g f_c'/10$) under any load combination is to be proportioned and detailed as described in 13.6.

Experimental evidence (Hirosawa 1977) indicates that, under reversals of displacement into the nonlinear range, behavior of continuous members having length-to-depth ratios of less than 4 is significantly different from the behavior of relatively slender members. Design rules derived from experience with relatively slender members do not apply

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13.5.1.3 Width of member, b_w, shall not be less than the smaller of $0.3h$ and 10 in.

13.5.1.4 Width of member, b_w, shall not exceed width of supporting member, c_2, plus a distance on each side of supporting member equal to the smaller of (a) and (b):

(a) Width of supporting member, c_2; and

(b) 0.75 times the overall dimension of supporting member, c_1.

COMMENTARY

directly to members with length-to-depth ratios less than 4, especially with respect to shear strength.

Geometric constraints indicated in 13.5.1.3 and 13.5.1.4 were derived from practice and research (ACI 352R) on reinforced concrete frames resisting earthquake-induced forces. The limits in 13.5.1.4 recognize that the maximum effective beam width depends principally on the column dimensions rather than on the depth of the beam, as suggested in the ACI 318-05 and earlier versions of the code. An example of maximum effective beam width is shown in Fig. R13.5.1.

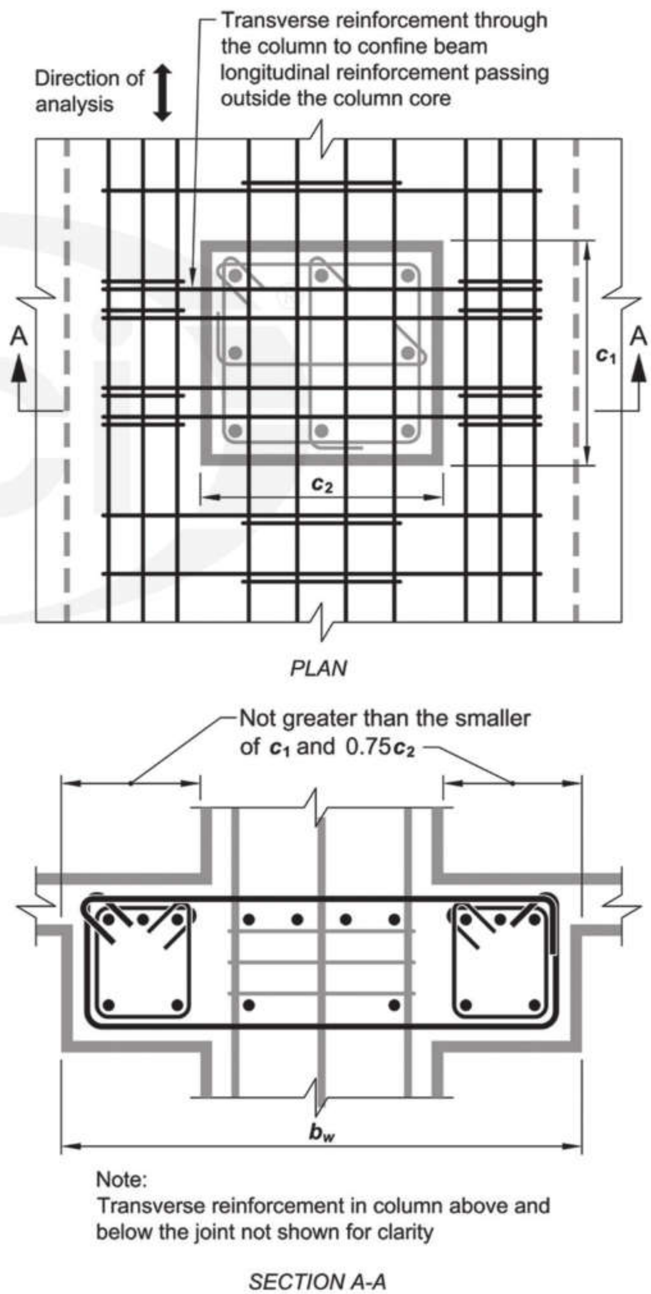

Fig. R13.5.1—Maximum effective width of wide beam and required transverse reinforcement.

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13.5.2 *Longitudinal reinforcement*

13.5.2.1 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 $200b_w d/f_y$, and the reinforcement ratio ρ shall not exceed 0.025. At least two bars shall be provided continuously both top and bottom.

13.5.2.2 Positive moment strength at joint face shall be not less than one-half the negative moment strength provided at that face of the joint. Neither the 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.

13.5.2.3 Lap splices of flexural reinforcement shall be permitted only if hoop or spiral reinforcement is provided over the lap length. Spacing of the transverse reinforcement enclosing the lap-spliced bars shall not exceed the smaller of $d/4$ and 4 in. Lap splices shall not be used:

(a) Within the joints

(b) Within a distance twice the member depth from the face of the joint

(c) Where analysis indicates flexural yielding is caused by inelastic lateral displacements of the frame

13.5.2.4 Mechanical splices shall conform to 13.1.6 and welded splices shall conform to 13.1.7.

13.5.2.5 Prestressing, where used, shall satisfy (a) through (d), unless used in a special moment frame as permitted by 13.8.3:

(a) The average prestress, f_{pc}, calculated for an area equal to the smallest cross-sectional dimension of the member multiplied by the perpendicular cross-sectional dimension shall not exceed the smaller of 500 psi and $f_c'/10$.

(b) Prestressing steel shall be unbonded in potential plastic hinge regions, and the calculated strains in prest

COMMENTARY

R13.5.2 *Longitudinal reinforcement*

Section 10.3.5 limits the 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) does not 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.

R13.5.2.1 The limiting reinforcement ratio of 0.025 is based primarily on considerations of steel congestion and, indirectly, on limiting shear stresses in beams of typical proportions. The requirement of at least two bars, top and bottom, refers again to construction rather than behavioral requirements.

R13.5.2.3 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.

R13.5.2.5 These provisions were developed, in part, based on observations of building performance in earthquakes (ACI 423.3R). For calculating the average prestress, the smallest cross-sectional dimension in a beam normally is the web dimension and is not intended to refer to the flange thickness. In a potential plastic hinge region, the limitation on strain and the requirement for unbonded tendons are intended to prevent fracture of tendons under inelastic earthquake defor-

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mation. Calculation of the strain in the prestressing steel is

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steel under the design displacement shall be less than 1 percent.

(c) Prestressing steel shall not contribute to more than one-fourth of the positive or negative flexural strength at the critical section in a plastic hinge region and shall be anchored at or beyond the exterior face of the joint.

(d) Anchorages of the post-tensioning tendons resisting earthquake-induced forces shall be capable of allowing tendons to withstand 50 cycles of loading, bounded by 40 and 85 percent of the specified tensile strength of the prestressing steel.

13.5.3 *Transverse reinforcement*

13.5.3.1 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

13.5.3.2 The first hoop shall be located not more than 2 in. from the face of a supporting member. Spacing of the hoops shall not exceed the smallest of (a), (b), and (c):

(a) $d/4$

(b) Six times the diameter of the smallest primary flexural reinforcing bars excluding longitudinal skin reinforcement required by 10.6.7

(c) 6 in.

13.5.3.3 Where hoops are required, primary flexural reinforcing bars closest to the tension and compression faces shall have lateral support conforming to 12.10.1.5.3. The spacing of transversely supported flexural reinforcing bars

COMMENTARY

required considering the anticipated inelastic mechanism of the structure. For prestressing steel unbonded along the full beam span, strains generally will be well below the specified limit. For prestressing steel with short unbonded length through or adjacent to the joint, the additional strain due to earthquake deformation is calculated as the product of the depth to the neutral axis and the sum of plastic hinge rotations at the joint, divided by the unbonded length.

The restrictions on the flexural strength provided by the tendons are based on the results of analytical and experimental studies (Ishizuka and Hawkins 1987; Park and Thompson 1977; Thompson and Park 1980). Although satisfactory seismic performance can be obtained with greater amounts of prestressing steel, this restriction is needed to allow the use of the same response modification and deflection amplification factors as those specified in model codes for special moment frames without prestressing steel. Prestressed special moment frames will generally contain continuous prestressing steel that is anchored with adequate cover at or beyond the exterior face of each beam-column connection located at the ends of the moment frame.

Fatigue testing for 50 cycles of loading between 40 and 80 percent of the specified tensile strength of the prestressing steel has been an industry practice of long standing (ACI 423.3R; ACI 423.7). The 80 percent limit was increased to 85 percent to correspond to the 1 percent limit on the strain in prestressing steel. Testing over this range of stress is intended to conservatively simulate the effect of a severe earthquake. Additional details on testing procedures, but to different stress levels, are provided in ACI 423.7.

R13.5.3 *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. R13.5.3.

For many years, the upper limit on hoop spacing was the smallest of $d/4$, eight longitudinal bar diameters, 24 tie bar diameters, and 12 in. The upper limits were changed because of concerns about adequacy of longitudinal bar buckling restraint and confinement of large beams.

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 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 13.5.3.5.

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shall not exceed 12 in. Skin reinforcement required by 10.6.7 need not be laterally supported.

13.5.3.4 Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than $\boldsymbol{d/2}$ throughout the length of the member.

13.5.3.5 Stirrups or ties required to resist shear shall be hoops over lengths of members in 13.5.3.1.

13.5.3.6 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-degree 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-degree hooks of the crossties shall be placed on that side.

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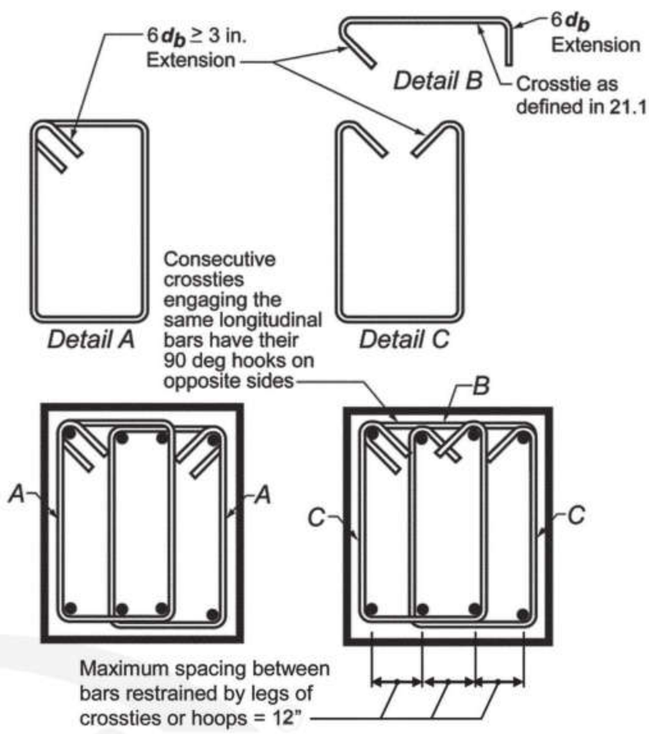

Fig. R13.5.3—Examples of overlapping hoops.

13.5.4 *Shear strength requirements*

13.5.4.1 *Design forces*

The design shear force $\boldsymbol{V_e}$ 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 $\boldsymbol{M_{pr}}$ act at the joint faces and that the member is loaded with the factored tributary gravity load along its span.

R13.5.4 *Shear strength requirements*

R13.5.4.1 *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 on the order of three or four 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 13.5.4.1 are illustrated in Fig. R13.5.4.

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 $\boldsymbol{1.25f_y}$ in the longitudinal reinforcement.

13.5.4.2 *Transverse reinforcement*

Transverse reinforcement over the lengths identified in 13.5.3.1 shall be proportioned to resist shear assuming $\boldsymbol{V_c = 0}$ when both (a) and (b) occur:

(a) The earthquake-induced shear force calculated in accordance with 13.5.4.1 represents one-half or more of the maximum required shear strength within those lengths

R13.5.4.2 *Transverse reinforcement*

Experimental studies (Popov et al. 1972; Wight and Sozen 1975) 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

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(b) The factored axial compressive force P_u including earthquake effects is less than $A_g f_c'/20$.

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increase of shear reinforcement being higher in the case of no axial load. This observation is reflected in the code (refer to 13.5.4.2) 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. However, this stratagem, 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 the shear with the shear (transverse) reinforcement confining and 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.

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Notes:

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.

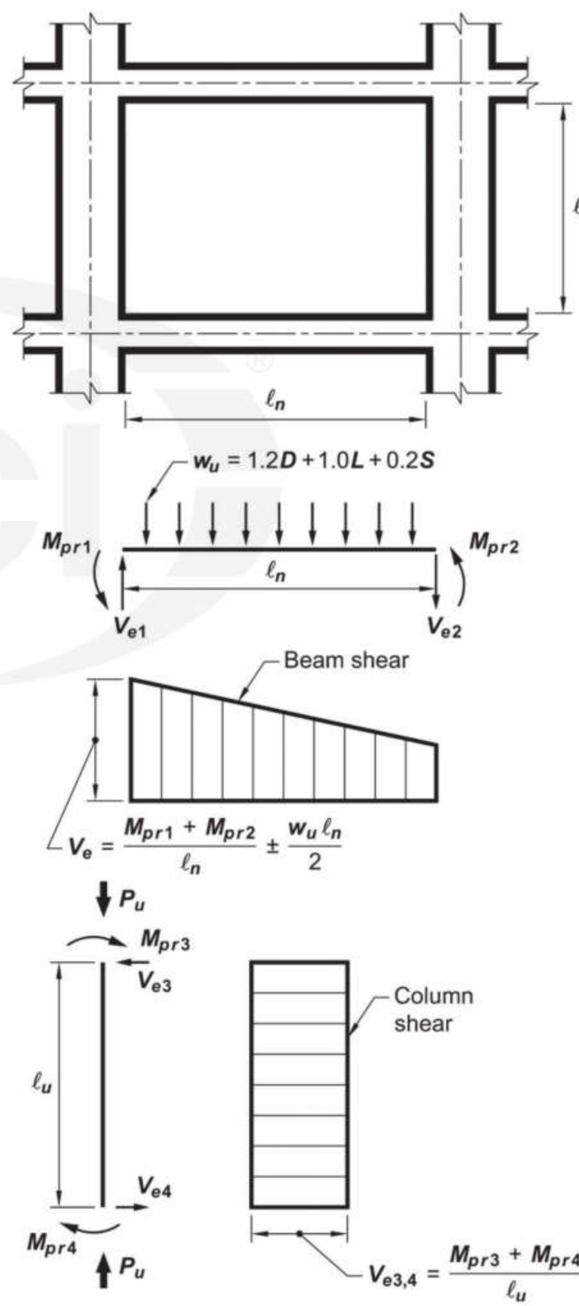

Fig. R13.5.4—Design shears for beams and columns.

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13.6—Special moment frame members subjected to bending and axial load

13.6.1 *Scope*

Requirements of this section apply to special moment frame members that form part of the seismic-force-resisting system and that resist a factored axial compressive force P_u under any load combination exceeding $A_g f_c'/10$. These frame members shall also satisfy the conditions of 13.6.1.1 and 13.6.1.2.

13.6.1.1 The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 12 in.

13.6.1.2 The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

13.6.2 *Minimum flexural strength of columns*

13.6.2.1 Columns shall satisfy 13.6.2.2 or 13.6.2.3.

13.6.2.2 The flexural strengths of the columns shall satisfy Eq. (13-1)

$$\sum M_{nc} \geq (6/5)\sum M_{nb} \tag{13-1}$$

$\sum M_{nc}$ is the sum of nominal flexural strengths of columns framing into the joint, evaluated at the faces of the 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.

$\sum M_{nb}$ is sum of nominal flexural strengths of the beams framing into the joint, evaluated at the faces of the 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 8.12 shall be assumed to contribute to M_{nb} 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 (13-1) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

13.6.2.3 If 13.6.2.2 is not satisfied at a joint, the lateral strength and stiffness of the columns framing into that joint shall be ignored when determining the calculated strength and stiffness of the structure. These columns shall conform to 13.13.

COMMENTARY

R13.6—Special moment frame members subjected to bending and axial load

R13.6.1 *Scope*

Section 13.6.1 is intended primarily for columns of special moment frames. Frame members, other than columns, that do not satisfy 13.5.1 are to be proportioned and detailed according to this section. These provisions apply to the frame member for all load combinations if the axial load exceeds $0.1A_g f_c'$ in any load combination.

The geometric constraints in 13.6.1.1 and 13.6.1.2 follow from previous practice (SEAOC 1999).

R13.6.2 *Minimum flexural strength of columns*

The intent of 13.6.2.2 is to reduce the likelihood of yielding in columns that are considered as part of the seismic-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 13.6.2.2, the nominal strengths of the girders and columns are calculated at the joint faces, and those strengths are compared directly using Eq. (13-1). ACI 318-95 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 (French and Moehle 1991) on beam-column subassemblies under lateral loading indicates that using the effective flange widths defined in 8.12 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 13.6.2.2 cannot be satisfied at a joint, 13.6.2.3 requires that 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. Furthermore, the column must be provided with transverse reinforcement to increase its toughness to resist shear and axial forces.

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13.6.3 *Longitudinal reinforcement*

13.6.3.1 Area of longitudinal reinforcement, A_{st}, shall not be less than $\mathbf{0.01}A_g$ or more than $\mathbf{0.06}A_g$.

13.6.3.2 In columns with circular hoops, the minimum number of longitudinal bars shall be 6.

13.6.3.3 Mechanical splices shall conform to 13.1.6 and welded splices shall conform to 13.1.7. 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 13.6.4.2 and 13.6.4.3.

13.6.4 *Transverse reinforcement*

13.6.4.1 Transverse reinforcement required in 13.6.4.2 through 13.6.4.4 shall be provided over a length ℓ_o from each joint face and on both sides of any section where flexural yielding is likely to occur as a result of inelastic lateral displacements of the frame. Length ℓ_o shall not be less than the largest of (a), (b), and (c):

(a) The depth of the member at the joint face or at the section where flexural yielding is likely to occur

(b) One-sixth of the clear span of the member

(c) 18 in.

13.6.4.2 Transverse reinforcement shall be provided by either single or overlapping spirals satisfying 12.10.1.4, circular hoops, or rectilinear hoops with or without crossties. Crossties of the same or smaller bar size 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. Spacing of crossties or legs of rectilinear hoops, h_x, within a cross section of the member shall not exceed 14 in. on center.

COMMENTARY

R13.6.3 *Longitudinal reinforcement*

The lower limit of the area of longitudinal reinforcement 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 these 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. 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 (Sivakumar et al. 1983).

R13.6.4 *Transverse reinforcement*

Requirements of this section are concerned with confining the concrete and providing lateral support to the longitudinal reinforcement.

R13.6.4.1 Section 13.6.4.1 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 (Watson et al. 1994).

R13.6.4.2 Sections 13.6.4.2 and 13.6.4.3 provide requirements for configuration of transverse reinforcement for columns and joints of special moment frames. Figure R13.6.4.2 shows an example of transverse reinforcement provided by one hoop and three crossties. Crossties with a 90-degree hook are not as effective as either crossties with 135-degree hooks or hoops in providing confinement. Tests show that if crosstie ends with 90-degree hooks are alternated, confinement will be sufficient.

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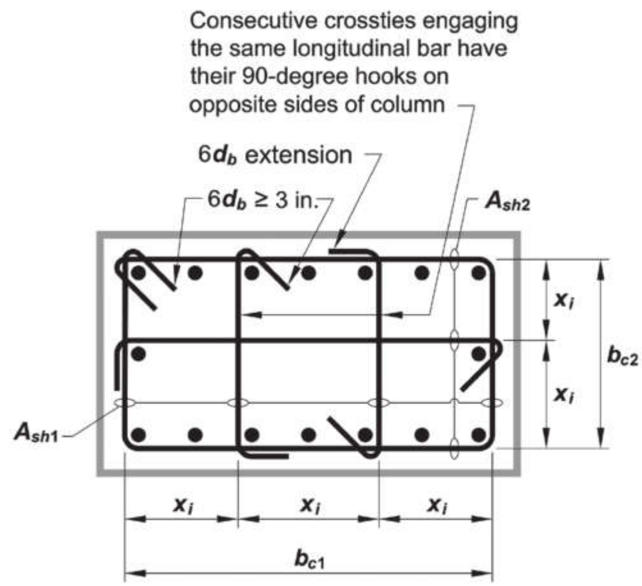

Fig. R13.6.4.2—Example of transverse reinforcement in columns

13.6.4.3 Spacing of transverse reinforcement along the length ℓ_o of the member shall not exceed the smallest of (a), (b), and (c):

(a) One-fourth of the minimum member dimension

(b) Six times the diameter of the smallest longitudinal bar

(c) s_o as defined by Eq. (13-2)

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right) \tag{13-2}$$

The value of s_o shall not exceed 6 in. and need not be taken less than 4 in.

R13.6.4.3 The requirement that spacing not exceed one-fourth 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; 13.6.4.3 permits this limit to be relaxed to a maximum of 6 in. if the spacing of crossties or legs of overlapping hoops is less than 8 in.

13.6.4.4 Amount of transverse reinforcement required in (a) or (b) shall be provided unless a larger amount is required by 13.6.5.

(a) The volumetric ratio of spiral or circular hoop reinforcement, ρ_s, shall not be less than required by Eq. (13-3)

$$\rho_s = 0.12 f_c'/f_{yt} \tag{13-3}$$

and shall not be less than required by Eq. (10-5).

(b) The total cross-sectional area of rectangular hoop reinforcement, A_{sh}, shall not be less than required by Eq. (13-4) and (13-5)

$$A_{sh} = 0.3(sb_c f_c'/f_{yt})[(A_g/A_{ch}) - 1] \tag{13-4}$$

$$A_{sh} = 0.09 sb_c f_c'/f_{yt} \tag{13-5}$$

R13.6.4.4 The effect of helical (spiral) reinforcement and adequately configured rectilinear hoop reinforcement on strength and ductility of columns is well established (Sakai and Sheikh 1989). While analytical procedures exist for calculation of strength and ductility capacity of columns under axial and moment reversals (Meinheit and Jirsa 1977), the axial load and deformation demands 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. (10-5) and (13-4) are required, with the intent that spalling of shell concrete will not result in a loss of axial load strength of the column. Equations (13-3) and (13-5) govern for large-diameter columns and are intended to ensure adequate flexural curvature capacity in yielding regions.

Equations (13-4) and (13-5) are to be satisfied in both cross-sectional directions of the rectangular core. For each

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direction, b_c is the core dimension perpendicular to the tie legs that constitute A_{sh}, as shown in Fig. R13.6.4.2.

Research results indicate that yield strengths higher than those specified in 11.4.2 can be used effectively as confinement reinforcement. A f_{yt} of 100,000 psi is permitted in Eq. (13-3), (13-4), and (13-5) where ASTM A1035 is used as confinement reinforcement.

13.6.4.5 Beyond the length ℓ_o specified in 13.6.4.1, the column shall contain spiral or hoop reinforcement satisfying 12.10 with center-to-center spacing s not exceeding the smaller of six times the diameter of the smallest longitudinal column bars and 6 in., unless a larger amount of transverse reinforcement is required by 13.6.3.2 or 13.6.5.

R13.6.4.5 The provisions of 13.6.4.5 are intended to provide reasonable protection and ductility to the midheight of columns outside the length ℓ_o. Observations after earthquakes have shown significant damage to columns in this region, and the minimum ties or spirals required should provide a more uniform toughness of the column along its length.

13.6.4.6 Columns supporting reactions from discontinued stiff members, such as walls, shall satisfy (a) and (b):

(a) Transverse reinforcement as required in 13.6.4.2 through 13.6.4.4 shall be provided over their full height at all levels beneath the discontinuity if the factored axial compressive force in these members, related to earthquake effect, exceeds $A_g f_c'/10$. Where design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $A_g f_c'/10$ shall be increased to $A_g f_c'/4$.

(b) The transverse reinforcement shall extend into the discontinued member at least a distance equal to ℓ_d, where ℓ_d is determined in accordance with 13.7.5 for the largest longitudinal column bar. Where the lower end of the column terminates on a wall, the required transverse reinforcement shall extend into the wall at least ℓ_d of the largest longitudinal column bar at the point of termination. Where the column terminates on a footing, mat, or pile cap, the required transverse reinforcement shall extend at least 12 in. into the footing, mat, or pile cap.

R13.6.4.6 Columns supporting discontinued stiff members, such as walls or trusses, may develop considerable inelastic response. Therefore, it is required that these columns have the specified 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. Refer to R13.11.7.5 for discussion of the overstrength factor Ω_o applied in some codes.

13.6.4.7 If the concrete cover outside the confining transverse reinforcement specified in 13.6.4.1, 13.6.4.5, and 13.6.4.6 exceeds 4 in., additional transverse reinforcement shall be provided. Concrete cover for additional transverse reinforcement shall not exceed 4 in. and spacing of additional transverse reinforcement shall not exceed 12 in.

R13.6.4.7 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.

13.6.5 *Shear strength requirements*

R13.6.5 *Shear strength requirements*

13.6.5.1 *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} at each end of the member associated with the range of factored axial loads P_u acting on the member. The member shears need not exceed those determined from joint strengths based on M_{pr} of the transverse members framing into the joint. In no case shall V_e be

R13.6.5.1 *Design forces*

The procedures of 13.5.4.1 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

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less than the factored shear determined by analysis of the structure.

13.6.5.2 *Transverse reinforcement*

Transverse reinforcement over the lengths ℓ_o, identified in 13.6.4.1, shall be proportioned to resist shear assuming V_c = **0** when both (a) and (b) occur:

(a) The earthquake-induced shear force, calculated in accordance with 13.6.5.1, represents one-half or more of the maximum required shear strength within ℓ_o

(b) The factored axial compressive force P_u including earthquake effects is less than $A_g f_c'/20$

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factor of 1.0 and reinforcing steel stress equal to at least **1.25**f_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. R13.5.4 may be computed from the flexural member strengths at the beam-column joints.

13.7—Joints of special moment frames

13.7.1 *Scope*

Requirements of 13.7 apply to beam-column joints of special moment frames forming part of the seismic-force-resisting system.

R13.7—Joints of special moment frames

13.7.2 *General requirements*

13.7.2.1 Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural tensile reinforcement is **1.25**f_y.

13.7.2.2 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 13.7.5 and in compression according to Chapter 12.

13.7.2.3 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 beam bar for normalweight concrete. For lightweight concrete, the dimension shall be not less than 26 times the bar diameter.

R13.7.2 *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 **1.25**f_y in the reinforcement (refer to 13.7.2.1). A detailed explanation of the reasons for the possible development of stresses in excess of the yield strength in beam tensile reinforcement is provided in ACI 352R.

R13.7.2.3 Research (Meinheit and Jirsa 1977; Briss et al. 1978; Ehsani 1982; Durrani and Wight 1982; Leon 1989) 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 reduce slip substantially 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 for beam bars through lightweight concrete joints, the limit was based on the amplification factor of 1.3 in Chapter 12 starting with ACI 318-89. The amplification factor was modified slightly in 2008 to 1/0.75 = 1.33, which did not affect this Code section. 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 Zhu and Jirsa (1983).

13.7.3 *Transverse reinforcement*

R13.7.3 *Transverse reinforcement*

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13.7.3.1 Joint transverse reinforcement shall satisfy either 13.6.4.4(a) or 13.6.4.4(b), and shall also satisfy 13.6.4.2, 13.6.4.3, and 13.6.4.7, except as permitted in 13.7.3.2.

13.7.3.2 Where members frame into all four sides of the joint and where each member width is at least three-fourths the column width, the amount of reinforcement specified in 13.6.4.4(a) or 13.6.4.4(b) shall be permitted to be reduced by half, and the spacing required in 13.6.4.3 shall be permitted to be increased to 6 in. within the overall depth $\boldsymbol{h}$ of the shallowest framing member.

13.7.3.3 Longitudinal beam reinforcement outside the column core shall be confined by transverse reinforcement passing through the column that satisfies spacing requirements of 13.5.3.2, and requirements of 13.5.3.3 and 13.5.3.6, if such confinement is not provided by a beam framing into the joint.

13.7.4 *Shear strength*

13.7.4.1 For normalweight concrete, V_n of the joint shall not be taken as greater than the values specified below.

For joints confined by beams on all four faces: $20\sqrt{f_c'}A_j$

For joints confined by beams on three faces or on two opposite faces: $15\sqrt{f_c'}A_j$

For other cases: $12\sqrt{f_c'}A_j$

A beam that frames into a face is considered to provide confinement to the joint if it covers at least three-fourths of the face of the joint. Extensions of beams at least one overall beam depth $\boldsymbol{h}$ beyond the joint face are permitted to be considered adequate for confining that joint face. Extensions of beams shall satisfy 13.5.1.3, 13.5.2.1, 13.5.3.2, 13.5.3.3 and 13.5.3.6. A_j is the effective cross-sectional area within a joint computed from joint depth times effective joint width. Joint depth shall be the overall depth of the column, $\boldsymbol{h}$. Effective joint width shall be the overall width of the column, except where a beam frames into a wider column, effective joint width shall not exceed the smaller of (a) and (b):

(a) Beam width plus joint depth

(b) Twice the smaller perpendicular distance from longitudinal axis of beam to column side

13.7.4.2 For lightweight concrete, the nominal shear strength of the joint shall not exceed three-fourths of the limits given in 13.7.4.1.

COMMENTARY

The Code requires transverse reinforcement in a joint regardless of the magnitude of the calculated shear force. In 13.7.3.2, the amount of confining reinforcement may be reduced and the spacing may be increased if horizontal members frame into all four sides of the joint.

Section 13.7.3.3 refers to a joint where the width of the beam exceeds the corresponding column dimension. In that case, beam reinforcement not confined by the column reinforcement should be provided lateral support either by a beam framing into the same joint or by transverse reinforcement.

An example of transverse reinforcement through the column provided to confine the beam reinforcement passing outside the column core is shown in Fig. R13.5.1. Additional detailing guidance and design recommendations for both interior and exterior wide-beam connections with beam reinforcement passing outside the column core may be found in ACI 352R.

R13.7.4 *Shear strength*

The requirements in Chapter 13 for proportioning joints are based on ACI 352R in that behavioral phenomena within the joint are interpreted in terms of a nominal shear strength of the joint. Because tests of joints (Meinheit and Jirsa 1977) and deep beams (Hirosawa 1977) indicated that shear strength was not as sensitive to joint (shear) reinforcement as implied by the expression developed by Joint ACI-ASCE Committee 326 (1962) for beams, ACI Committee 318 set the strength of the joint as a function of only the compressive strength of the concrete (refer to 13.7.4) and requires a minimum amount of transverse reinforcement in the joint (refer to 13.7.3). The effective area of joint, A_j, is illustrated in Fig. R13.7.4. In no case is A_j greater than the column

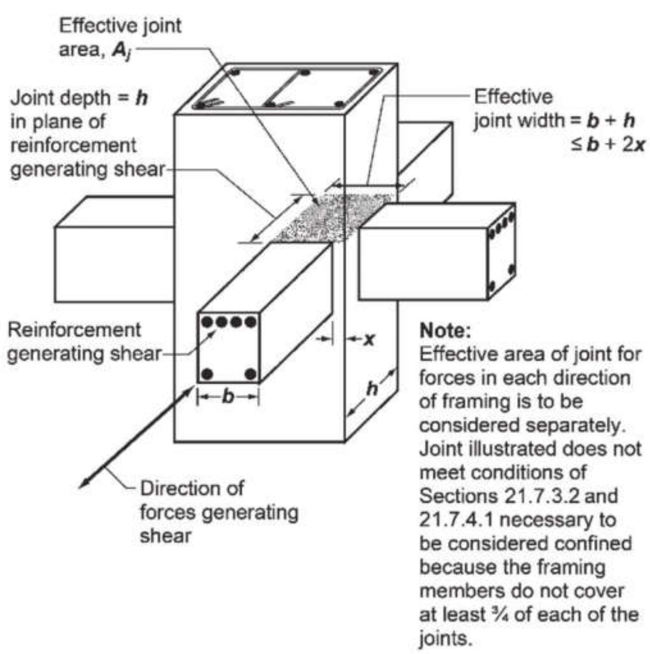

Fig. R13.7.4—Effective joint area.

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cross-sectional area. A circular column should be considered as having a square section of equivalent area.

The three levels of shear strength required by 13.7.4.1 are based on the recommendation in ACI 352R. Test data reviewed by the committee (Ehsani 1985) indicate that the lower value given in 13.7.4.1 of ACI 318-83 was unconservative when applied to corner joints.

Cyclic loading tests of joints with extensions of beams with lengths at least equal to their depths have indicated similar joint shear strengths to those of joints with continuous beams. These findings suggest that extensions of beams, when properly dimensioned and reinforced with longitudinal and transverse bars, provide effective confinement to the joint faces, thus delaying joint strength deterioration at large deformations (Meinheit and Jirsa 1981).

13.7.5 *Development length of bars in tension*

13.7.5.1 For bar sizes No. 3 through No. 11, the development length ℓ_{dh} for a bar with a standard 90-degree hook in normalweight concrete shall not be less than the largest of $8d_b$, 6 in., and the length required by Eq. (13-6)

$$\ell_{dh} = f_y d_b/(65\sqrt{f_c'}) \tag{13-6}$$

For lightweight concrete, ℓ_{dh} for a bar with a standard 90-degree hook shall not be less than the largest of $10d_b$, 7-1/2 in., and 1.25 times the length required by Eq. (13-6).

The 90-degree hook shall be located within the confined core of a column or of a boundary element.

13.7.5.2 For bar sizes No. 3 through No. 11, ℓ_{dh}, the development length in tension for a straight bar, shall not be less than the larger of (a) and (b):

(a) 2.5 times the length required by 13.7.5.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 12 in.

(b) 3.25 times the length required by 13.7.5.1 if the depth of the concrete cast in one lift beneath the bar exceeds 12 in.

13.7.5.3 Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary element. Any portion of ℓ_{dh} not within the confined core shall be increased by a factor of 1.6.

13.7.5.4 If epoxy-coated or zinc and epoxy dual-coated reinforcement is used, the development lengths in 13.7.5.1 through 13.7.5.3 shall be multiplied by applicable factors in 12.8.2.4 or 12.8.5.2.

R13.7.5 *Development length of bars in tension*

Minimum development length in tension for deformed bars with standard hooks embedded in normalweight concrete is determined using Eq. (13-6), which is based on the requirements of 12.8.5. Because Chapter 13 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. (13-6). The development length that would be derived directly from 12.8.5 is increased to reflect the effect of load reversals.

The development length in tension of a deformed 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 (refer to Fig. R12.8.5).

Factors such as the actual stress in the reinforcement being more than the yield stress 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. (13-6).

For lightweight concrete, the length required by Eq. (13-6) is to be increased by 25 percent to compensate for variability of bond characteristics of reinforcing bars in various types of lightweight concrete.

Section 13.7.5.2 specifies the minimum development length in tension for straight bars as a multiple of the length indicated by 13.7.5.1. Section 13.7.5.2(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 13.5.3, 13.6.4, or 13.7.3), the required development length is increased on the premise that the limiting bond stress outside the confined region is less than that inside.

$$\ell_{dm} = 1.6(\ell_d - \ell_{dc}) + \ell_{dc}$$

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$$\ell_{dm} = 1.6\ell_d - 0.6\ell_{dc}$$

where ℓ_{dm} is required development length if bar is not entirely embedded in confined concrete; ℓ_d is required development length in tension for straight bar embedded in confined concrete; and ℓ_{dc} is the length of bar embedded in confined concrete.

Lack of reference to No. 14 and No. 18 bars in 13.7.5 is due to the paucity of information on anchorage of such bars subjected to load reversals simulating earthquake effects.

13.8—Special moment frames constructed using precast concrete

13.8.1 *Scope*

Requirements of 13.8 apply to special moment frames constructed using precast concrete forming part of the seismic-force-resisting system.

13.8.2 Special moment frames with ductile connections constructed using precast concrete shall satisfy (a) and (b) and all requirements for special moment frames constructed with cast-in-place concrete:

(a) V_n for connections computed according to 11.6.4 shall not be less than $2V_e$, where V_e is calculated according to 13.5.4.1 or 13.6.5.1

(b) Mechanical splices of beam reinforcement shall be located not closer than $h/2$ from the joint face and shall meet the requirements of 13.1.6.

13.8.3 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 (a), (b), (c), and (d).

(a) Provisions of 13.5.1.2 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

(d) For column-to-column connections, ϕS_n shall not be less than $1.4S_e$. At column-to-column connections, ϕM_n shall be not less than $0.4M_{pr}$ for the column within the story height, and ϕV_n of the connection shall be not less than V_e determined by 13.6.5.1

R13.8—Special moment frames constructed using precast concrete

The detailing provisions in 13.8.2 and 13.8.3 are intended to produce frames that respond to design displacements essentially like monolithic special moment frames.

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 (Yoshioka and Sekine 1991; Kurose et al. 1991; Restrepo et al. 1995a,b). Requirements for mechanical splices are in addition to those in 13.1.6 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 13.8.2 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 13.5.4.1 or 13.6.5.1, 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. R13.8.3. Capacity-design techniques are used in 13.8.3(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 (Palmieri et al. 1996). Locations of strong connections should be selected carefully or other measures should be taken, such as debonding of reinforcing bars in highly stressed regions, to avoid strain concentrations that can result in premature fracture of reinforcement.

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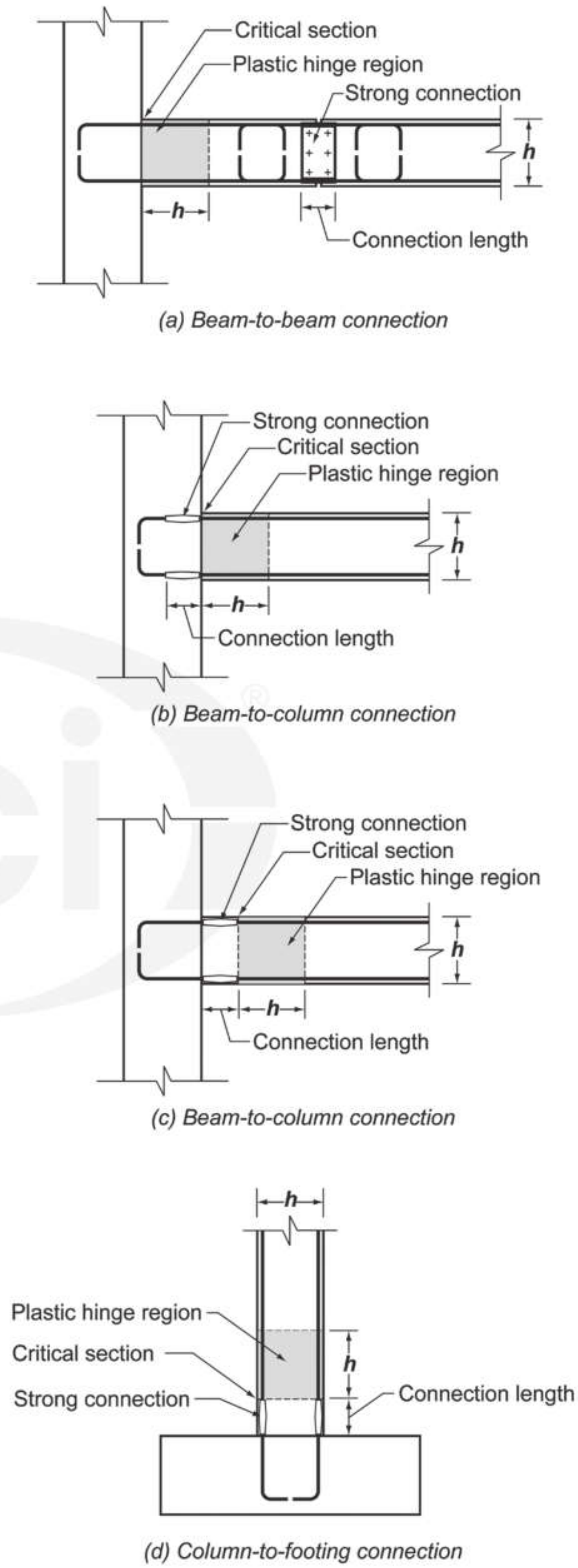

Fig. R13.8.3—Strong connection examples.

13.8.4 Special moment frames constructed using precast concrete and not satisfying the requirements of 13.8.2 or 13.8.3 shall satisfy the requirements of ACI 374.1 and the requirements of (a) and (b):

R13.8.4 Precast frame systems not satisfying the prescriptive requirements of Chapter 13 have been demonstrated in experimental studies to provide satisfactory seismic performance characteristics (Stone et al. 1995; Nakaki et

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(a) Details and materials used in the test specimens shall be representative of those used in the structure
(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

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al. 1999). ACI 374.1 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 licensed design professional can demonstrate that those deviations do not adversely affect the behavior of the framing system.

ACI ITG-1.2 defines design requirements for one type of special precast concrete moment frame for use in accordance with 13.8.4.

13.9—Special structural walls and coupling beams

13.9.1 *Scope*

Requirements of 13.9 apply to special structural walls and all components of special structural walls including coupling beams and wall piers forming part of the seismic-force-resisting system. Special structural walls constructed using precast concrete shall also comply with 13.10.

R13.9—Special structural walls and coupling beams

R13.9.1 *Scope*

This section contains requirements for the dimensions and details of special structural walls and all components including coupling beams and wall piers. Wall piers are defined in 2.2. Design provisions for vertical wall segments depend on the aspect ratio of the wall segment in the plane of the wall (h_w/ℓ_w), and the aspect ratio of the horizontal cross section (ℓ_w/b_w), and generally follow the descriptions in Table R13.9.1. The limiting aspect ratios for wall piers are based on engineering judgment. It is intended that flexural yielding of the vertical reinforcement in the pier should limit shear demand on the pier.

Table R13.9.1—Governing design provisions for vertical wall segment

Clear height of vertical wall segment/ length of vertical wall segment (h_w/ℓ_w)	Length of vertical wall segment/wall thickness (ℓ_w/b_w)		
	$\ell_w/b_w \le 2.5$	$2.5 < \ell_w/b_w \le 6.0$	$\ell_w/b_w > 6.0$
$h_w/\ell_w < 2.0$	Wall	Wall	Wall
$h_w/\ell_w \ge 2.0$	Wall pier required to satisfy specified column design requirements; refer to 21.9.8.1	Wall pier required to satisfy specified column design requirements or alternative requirements; refer to 21.9.8.1	Wall

Note: h_w is the clear height, ℓ_w is the horizontal length, and b_w is the width of the web of the wall segment.

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13.9.2 *Reinforcement*

13.9.2.1 The distributed web reinforcement ratios, ρ_ℓ and ρ_t, for structural walls shall not be less than 0.0025, except that if V_u does not exceed $A_{cv}\lambda\sqrt{f_c'}$, ρ_ℓ and ρ_t shall be permitted to be reduced to the values required in 15.4. Reinforcement spacing each way in structural walls shall not exceed 18 in. Reinforcement contributing to V_n shall be continuous and shall be distributed across the shear plane.

13.9.2.2 At least two curtains of reinforcement shall be used in a wall if V_u exceeds $2A_{cv}\lambda\sqrt{f_c'}$.

13.9.2.3 Reinforcement in structural walls shall be developed or spliced for f_y in tension in accordance with Chapter 12, except:

(a) The effective depth d of the member referenced in 12.8.10.3 shall be permitted to be taken as $0.8\ell_w$ for walls
(b) The requirements of 12.8.11, 12.8.12, and 12.8.13 need not be satisfied
(c) At locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements, development lengths of longitudinal reinforcement shall be 1.25 times the values calculated for f_y in tension
(d) Mechanical splices of reinforcement shall conform to 13.1.6 and welded splices of reinforcement shall conform to 13.1.7.

13.9.3 *Design forces*

V_u shall be obtained from the lateral load analysis in accordance with the factored load combinations.

13.9.4 *Shear strength*

13.9.4.1 V_n of structural walls shall not exceed

$$V_n = A_{cv}(\alpha_c\lambda\sqrt{f_c'} + \rho_t f_y) \tag{13-7}$$

where the coefficient α_c is 3.0 for $h_w/\ell_w \leq 1.5$, is 2.0 for $h_w/\ell_w \geq 2.0$, and varies linearly between 3.0 and 2.0 for h_w/ℓ_w between 1.5 and 2.0.

13.9.4.2 In 13.9.4.1, 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.

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R13.9.2 *Reinforcement*

Minimum reinforcement requirements in 13.9.2.1 follow from preceding codes. The requirement for distributed 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 in 13.9.2.2 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.

R13.9.2.3 Requirements are based on provisions in Chapter 12. Because actual forces in longitudinal reinforcement of structural walls may exceed calculated forces, reinforcement should be developed or spliced to reach the yield strength of the bar in tension. Requirements of 12.8.11, 12.8.12, and 12.8.13 address issues related to beams and do not apply to walls. At locations where yielding of longitudinal reinforcement is expected, a 1.25 multiplier is applied to account for the likelihood that the actual yield strength exceeds the specified yield strength of the bar, as well as the influence of strain hardening and cyclic load reversals. Where transverse reinforcement is used, development lengths for straight and hooked bars may be reduced as permitted in 12.8.2 and 12.8.5, respectively, because closely spaced transverse reinforcement improves the performance of splices and hooks subjected to repeated inelastic demands (Barda et al. 1977).

R13.9.3 *Design forces*

Design shears for structural walls are obtained from lateral load analysis with the appropriate load factors. However, the possibility of yielding in components of such structures should be considered, 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.

R13.9.4 *Shear strength*

Equation (13-7) recognizes the higher shear strength of walls with high shear-to-moment ratios (Hirosawa 1977; Joint ACI-ASCE Committee 326 1962; Barda et al. 1977). 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. (13-7) facilitates design calculations for walls with uniformly distributed reinforcement and walls with openings.

A vertical wall segment refers to a part of a wall bounded horizontally 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. When designing an

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13.9.4.3 Walls shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. If h_w/ℓ_w does not exceed 2.0, reinforcement ratio ρ_ℓ shall not be less than reinforcement ratio ρ_t.

13.9.4.4 For all vertical wall elements resisting a common lateral force, combined V_n shall not be taken larger than $8A_{cv}\sqrt{f_c'}$, where A_{cv} is the gross combined area of all vertical wall segments. For any one of the individual vertical wall segments, V_n shall not be taken larger than $10A_{cw}\sqrt{f_c'}$, where A_{cw} is the area of concrete section of the individual vertical wall segment considered.

13.9.4.5 For horizontal wall segments, including coupling beams, V_n shall not be taken larger than $10A_{cw}\sqrt{f_c'}$, where A_{cw} is the area of concrete section of a horizontal wall segment or coupling beam.

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isolated wall or a vertical wall segment, ρ_t refers to horizontal reinforcement and ρ_ℓ refers to vertical reinforcement.

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 13.9.4.2 is to make certain that any segment of a wall is not assigned a unit strength larger than that for the entire 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_t and ρ_ℓ should be appropriately distributed along the length and height of the wall (refer to 13.9.4.3). Chord reinforcement provided near wall edges in concentrated amounts for resisting bending moment is not to be included in determining ρ_t and ρ_ℓ. 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 vertical wall segments 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 vertical wall segment 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 13.9.4.5 refers to wall sections between two vertically aligned openings (refer to Fig. R13.9.4.5). It is, in effect, a vertical wall segment rotated through 90 degrees. A horizontal wall segment is also referred to as a coupling beam when the openings are aligned vertically over the building height. When designing a horizontal wall segment or coupling beam, ρ_t refers to vertical reinforcement and ρ_ℓ refers to horizontal reinforcement.

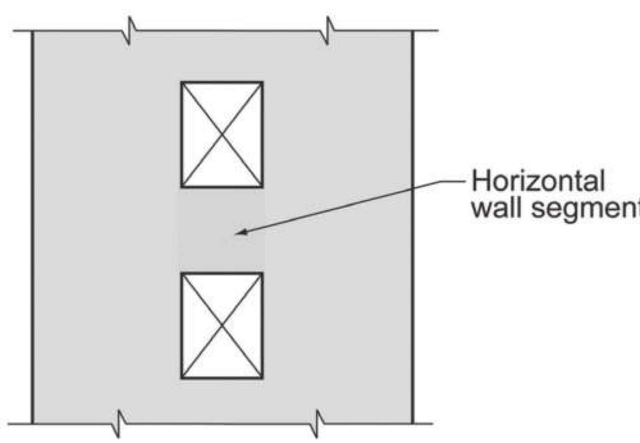

Fig. R13.9.4.5—Wall with openings.

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13.9.5 *Design for flexure and axial loads*

13.9.5.1 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.

13.9.5.2 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.

13.9.6 *Boundary elements of special structural walls*

13.9.6.1 The need for special boundary elements at the edges of structural walls shall be evaluated in accordance with 13.9.6.2 or 13.9.6.3. The requirements of 13.9.6.4 and 13.9.6.5 also shall be satisfied.

13.9.6.2 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 13.9.6.3.

(a) Compression zones shall be reinforced with special boundary elements where

$$c \geq \frac{\ell_w}{600(\delta_u/h_w)} \tag{13-8}$$

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R13.9.5 *Design for flexure and axial loads*

R13.9.5.1 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 (Taylor et al. 1998).

R13.9.5.2 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 (Wallace 1996) 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.

R13.9.6 *Boundary elements of special structural walls*

R13.9.6.1 Two design approaches for evaluating detailing requirements at wall boundaries are included in 13.9.6.1. Section 13.9.6.2 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 13.9.6.3 are similar to those of ACI 318-95 and have been retained because they are conservative for assessing required transverse reinforcement at wall boundaries for many walls. Requirements of 13.9.6.4 and 13.9.6.5 apply to structural walls designed by either 13.9.6.2 or 13.9.6.3.

R13.9.6.2 Section 13.9.6.2 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 (13-8) follows from a displacement-based approach (Moehle 1992; Wallace and Orakcal 2002). 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

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c in Eq. (13-8) corresponds to the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. Ratio δ_u/h_w in Eq. (13-8) shall not be taken less than 0.007.

(b) Where special boundary elements are required by 13.9.6.2(a), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of ℓ_w or $M_u/4V_u$.

13.9.6.3 Structural walls not designed to the provisions of 13.9.6.2 shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to load combinations including earthquake effects E, 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 13.9.5.2 shall be used.

13.9.6.4 Where special boundary elements are required by 13.9.6.2 or 13.9.6.3, (a) through (e) 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 $c/2$, where c is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with δ_u

(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) The boundary element transverse reinforcement shall satisfy the requirements of 13.6.4.2 through 13.6.4.4, except Eq. (13-4) need not be satisfied and the transverse reinforcement spacing limit of 13.6.4.3(a) shall be one-third of the least dimension of the boundary element

(d) The boundary element transverse reinforcement at the wall base shall extend into the support at least ℓ_d, according to 13.9.2.3, of the largest longitudinal reinforcement in the special boundary element unless the special boundary element terminates on a footing, mat, or pile cap, where special boundary element transverse reinforcement shall extend at least 12 in. into the footing, mat, or pile cap

(e) Horizontal reinforcement in the wall web shall extend to within 6 in. of the end wall. Reinforcement shall be anchored to develop f_y in tension within the confined core

COMMENTARY

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. (13-8) 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 design displacement δ_u.

R13.9.6.3 By this procedure, the wall is considered to be acted on by gravity loads 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 $0.2f_c'$. The stress is to be 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.

R13.9.6.4 The value of $c/2$ in 13.9.6.4(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 (13-4) does not apply to walls.

The horizontal reinforcement in a structural wall with low shear-to-moment ratio resists shear through truss action, with the horizontal bars acting like stirrups in a beam. Thus, the horizontal bars provided for shear reinforcement must be developed within the confined core of the boundary element and extended as close to the end of the wall as cover requirements and proximity of other reinforcement permit. The requirement that the horizontal web reinforcement be anchored within the confined core of the boundary element and extended to within 6 in. from the end of the wall applies to all horizontal bars whether straight, hooked, or headed, as illustrated in Fig. R13.9.6.4.

Tests (Thomsen and Wallace 2004) show that adequate performance can be achieved using spacing larger than permitted by 13.6.4.3(a).

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of the boundary element using standard hooks or heads. Where the confined boundary element has sufficient length to develop the horizontal web reinforcement, and $A_v f_y/s$ of the web reinforcement is not greater than $A_{sh} f_{yt}/s$ of the boundary element transverse reinforcement parallel to the web reinforcement, it shall be permitted to terminate the web reinforcement without a standard hook or head.

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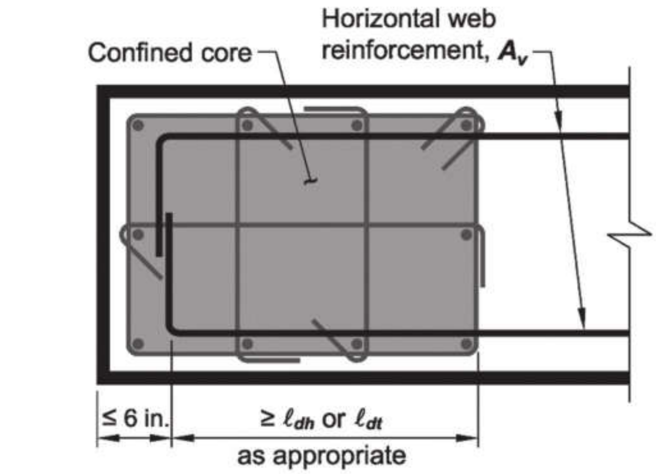
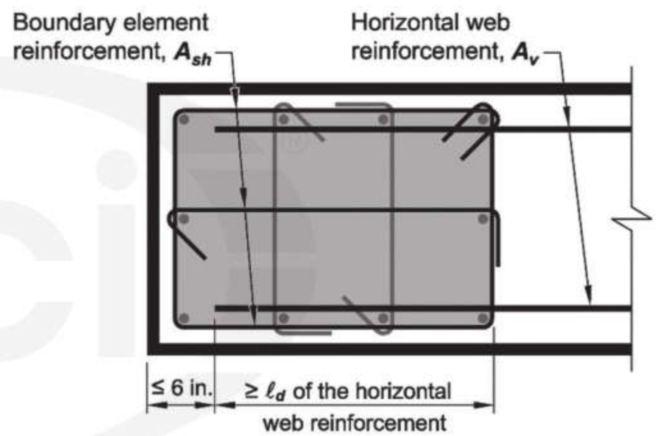


(a)
Option with standard hooks or headed reinforcement



(b)
Option with straight developed reinforcement

Fig. R13.9.6.4—Development of wall horizontal reinforcement in confined boundary element.

13.9.6.5 Where special boundary elements are not required by 13.9.6.2 or 13.9.6.3, (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 13.6.4.2 and 13.9.6.4(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 $A_{cv}\lambda\sqrt{f_c'}$, 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

R13.9.6.5 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. R13.9.6.5. A larger spacing of ties relative to 13.9.6.4(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.

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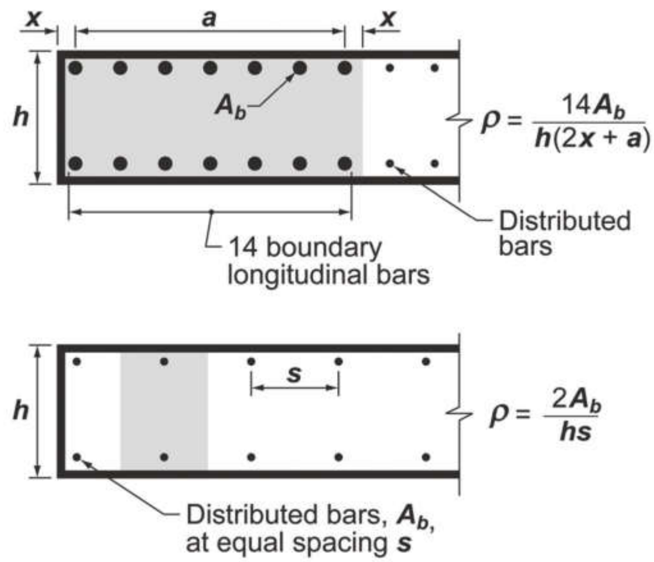

Fig. R13.9.6.5—Longitudinal reinforcement ratios for typical wall boundary conditions.

13.9.7 *Coupling beams*

13.9.7.1 Coupling beams with $(\ell_n/h) \geq 4$ shall satisfy the requirements of 13.5. The provisions of 13.5.1.3 and 13.5.1.4 need not be satisfied if it can be shown by analysis that the beam has adequate lateral stability.

13.9.7.2 Coupling beams with $(\ell_n/h) < 2$ and with V_u exceeding $4\lambda\sqrt{f_c'}A_{cw}$ 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 ability of the structure, the egress from the structure, or the integrity of nonstructural components and their connections to the structure.

13.9.7.3 Coupling beams not governed by 13.9.7.1 or 13.9.7.2 shall be permitted to be reinforced either with two intersecting groups of diagonally placed bars symmetrical about the midspan or according to 13.5.2 through 13.5.4.

13.9.7.4 Coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan shall satisfy (a), (b), and either (c) or (d). Requirements of 11.7 shall not apply.

(a) V_n shall be determined by

$$V_n = 2A_{vd}f_y\sin\alpha \leq 10\sqrt{f_c'}A_{cw} \qquad (13\text{-}9)$$

where α is the angle between the diagonal bars and the longitudinal axis of the coupling beam.

(b) Each group of diagonal bars shall consist of a minimum of four bars provided in two or more layers. The diagonal bars shall be embedded into the wall not less than 1.25 times the development length for f_y in tension.

(c) Each group of diagonal bars shall be enclosed by transverse reinforcement having out-to-out dimensions

R13.9.7 *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 (Paulay and Binney 1974; Barney et al. 1980) 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 aspect ratio $(\ell_n/h) \geq 4$. ACI 318-08 was changed to clarify that coupling beams of intermediate aspect ratio can be reinforced according to 13.5.2 through 13.5.4.

Diagonal bars should be placed approximately symmetrically in the beam cross section, in two or more layers. The diagonally placed bars are intended to provide the entire shear and corresponding moment strength of the beam; designs deriving their moment strength from combinations of diagonal and longitudinal bars are not covered by these provisions.

Two confinement options are described. According to 13.9.7.4(c), each diagonal element consists of a cage of longitudinal and transverse reinforcement, as shown in Fig. R13.9.7(a). Each cage contains at least four diagonal 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. Revisions were made in ACI 318-08 to relax spacing of transverse reinforcement confining the diagonal bars, to

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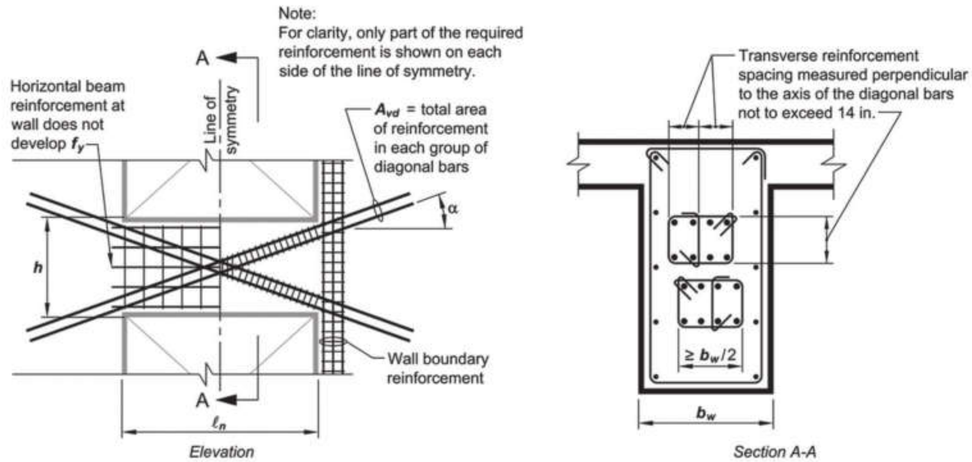
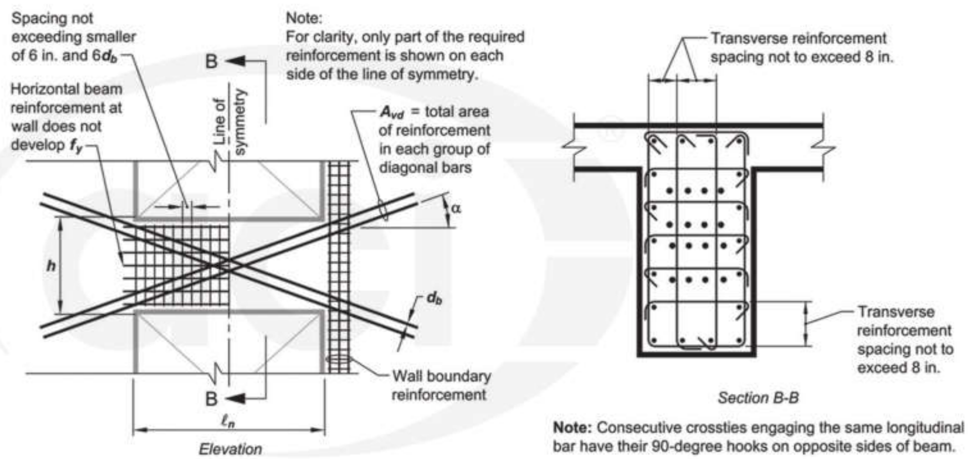


(a) Confinement of individual diagonals. Note: For clarity in the elevation view, only part of the total required reinforcement is shown on each side of the line of symmetry.



(b) Full confinement of diagonally reinforced concrete beam section.

Fig. R13.9.7—Coupling beams with diagonally oriented reinforcement. Wall boundary reinforcement shown on one side only for clarity.

not smaller than $b_w/2$ in the direction parallel to b_w and $b_w/5$ along the other sides, where b_w is the web width of the coupling beam. The transverse reinforcement shall satisfy 13.6.4.2 and 13.6.4.4, shall have spacing measured parallel to the diagonal bars satisfying 13.6.4.3(c) and not exceeding six times the diameter of the diagonal bars, and shall have spacing of crossties or legs of hoops measured perpendicular to the diagonal bars not exceeding 14 in. For the purpose of computing A_g for use in Eq. (10-5) and (13-4), the concrete cover as required in 12.7 shall be assumed on all four sides of each group of diagonal bars. The transverse reinforcement, or its alternatively configured transverse reinforcement satisfying the spacing and volume ratio requirements of the transverse reinforcement along the diagonals, shall continue through the intersection of the diagonal bars. Additional longitudinal and transverse reinforcement shall be distributed around the

clarify that confinement is required at the intersection of the diagonals, and to simplify design of the longitudinal and transverse reinforcement around the beam perimeter; beams with these new details are expected to perform acceptably.

Section 13.9.7.4(d) describes a second option for confinement of the diagonals introduced in ACI 318-08 (Fig. R21.9.7(b)). This second option is to confine the entire beam cross section instead of confining the individual diagonals. This option can considerably simplify field placement of hoops, which can otherwise be especially challenging where diagonal bars intersect each other or enter the wall boundary.

When coupling beams are not used as part of the lateral-force-resisting system, the requirements for diagonal reinforcement may be waived.

Test results (Barney et al. 1980) demonstrate that beams reinforced as described in Section 13.9.7 have adequate ductility at shear forces exceeding $10\sqrt{f_c'}\,b_w d$. Conse-

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beam perimeter with total area in each direction not less than $0.002b_w s$ and spacing not exceeding 12 in.

(d) Transverse reinforcement shall be provided for the entire beam cross section satisfying 13.6.4.2, 13.6.4.4, and 13.6.4.7, with longitudinal spacing not exceeding the smaller of 6 in. and six times the diameter of the diagonal bars, and with spacing of crossties or legs of hoops both vertically and horizontally in the plane of the beam cross section not exceeding 8 in. Each crosstie and each hoop leg shall engage a longitudinal bar of equal or larger diameter. It shall be permitted to configure hoops as specified in 13.5.3.6.

13.9.8 *Wall piers*

13.9.8.1 Wall piers shall satisfy the special moment frame requirements for columns of 13.6.3, 13.6.4, and 13.6.5, with joint faces taken as the top and bottom of the clear height of the wall pier. Alternatively, wall piers with $(\ell_w/b_w) > 2.5$ shall satisfy (a) through (f):

(a) Design shear force shall be determined in accordance with 13.6.5.1 with joint faces taken as the top and bottom of the clear height of the wall pier. Where the legally adopted general building code includes provisions to account for overstrength of the seismic-force-resisting system, the design shear force need not exceed Ω_o times the factored shear determined by analysis of the structure for earthquake effects.

(b) V_n and distributed shear reinforcement shall satisfy 13.9.4.

(c) Transverse reinforcement shall be in the form of hoops except it shall be permitted to use single-leg horizontal reinforcement parallel to ℓ_w where only one curtain of distributed shear reinforcement is provided. Single-leg horizontal reinforcement shall have 180-degree bends at each end that engage wall pier boundary longitudinal reinforcement.

(d) Vertical spacing of transverse reinforcement shall not exceed 6 in.

(e) Transverse reinforcement shall extend at least 12 in. above and below the clear height of the wall pier.

(f) Special boundary elements shall be provided if required by 13.9.6.3.

13.9.8.2 For wall piers at the edge of a wall, horizontal reinforcement shall be provided in adjacent wall segments above and below the wall pier and be proportioned to transfer the design shear force from the wall pier into the adjacent wall segments.

13.9.9 *Construction joints*

All construction joints in structural walls shall conform to 7.2 and contact surfaces shall be roughened as in 11.6.9.

13.9.10 *Discontinuous walls*

Columns supporting discontinuous structural walls shall be reinforced in accordance with 13.6.4.6.

COMMENTARY

quently, the use of a limit of $10\sqrt{f_c'}A_{cw}$ provides an acceptable upper limit.

R13.9.8 *Wall piers*

Door and window placements in structural walls sometimes lead to narrow vertical wall segments that are considered to be wall piers. The dimensions defining wall piers are given in 2.2. Shear failures of wall piers have been observed in previous earthquakes. The intent of 13.9.8 is to provide sufficient shear strength to wall piers so that a flexural yielding mechanism will develop. The provisions apply to wall piers designated as part of the seismic-force-resisting system. Provisions for wall piers not designated as part of the seismic-force-resisting system are given in 13.13. The effect of all vertical wall segments on the response of the structural system, whether designated as part of the seismic-force-resisting system or not, should be considered as required by 13.1.2.

Wall piers having $(\ell_w/b_w) \leq 2.5$ behave essentially as columns. Section 13.9.8.1 requires that such members satisfy reinforcement and shear strength requirements of 13.6.3 through 13.6.5. Alternative provisions are provided for wall piers having $(\ell_w/b_w) > 2.5$.

The design shear force determined according to 13.6.5.1 may be unrealistically large in some cases. As an alternative, 13.9.8.1(a) permits the design shear force to be determined using load combinations in which the earthquake effect has been amplified to account for system overstrength. Documents such as the NEHRP provisions, ASCE/SEI 7, and the International Building Code represent the amplified earthquake effect using the factor Ω_o.

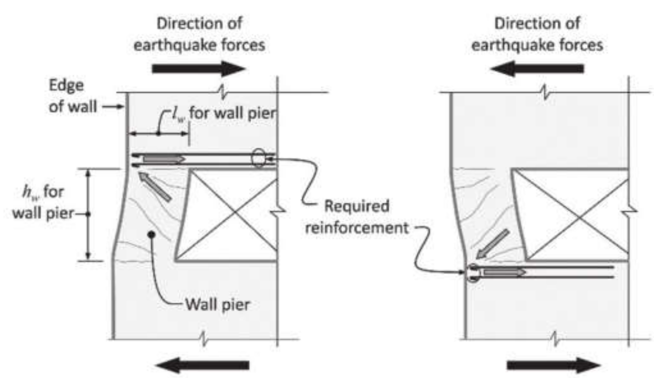

Fig. R13.9.8—Required horizontal reinforcement in wall segments above and below wall piers at the edge of a wall.

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Section 13.8.2 addresses wall piers at the edge of a wall. Under in-plane shear, inclined cracks can propagate into segments of the wall directly above and below the wall pier. Unless there is sufficient reinforcement in the adjacent wall segments, shear failure within the adjacent wall segments can occur. The length of embedment of the provided reinforcement into the adjacent wall segments should be determined considering both development length requirements and shear strength of the wall segments. Refer to Fig. R13.9.8.

13.10—Special structural walls constructed using precast concrete

13.10.1 *Scope*

Requirements of 13.10 apply to special structural walls constructed using precast concrete forming part of the seismic-force-resisting system.

13.10.2 Special structural walls constructed using precast concrete shall satisfy all requirements of 13.9 in addition to 13.4.2 and 13.4.3.

13.10.3 Special structural walls constructed using precast concrete and unbonded post-tensioning tendons and not satisfying the requirements of 13.10.2 are permitted provided they satisfy the requirements of ACI ITG-5.1.

R13.10—Special walls constructed using precast concrete

R13.10.3 Experimental and analytical studies (Priestley et al. 1999; Perez et al. 2003; Restrepo 2002) have demonstrated that some types of precast structural walls post-tensioned with unbonded tendons, and not satisfying the prescriptive requirements of Chapter 13, provide satisfactory seismic performance characteristics. ACI ITG-5.1 defines a protocol for establishing a design procedure, validated by analysis and laboratory tests, for such walls, with or without coupling beams.

ACI ITG-5.2 defines design requirements for one type of special structural wall constructed using precast concrete and unbonded post-tensioning tendons, and validated for use in accordance with 13.10.3.

13.11—Structural diaphragms and trusses

13.11.1 *Scope*

Floor and roof slabs acting as structural diaphragms to transmit forces induced by earthquake ground motions in structures assigned to SDC D, E, or F shall be designed in accordance with this section. This section also applies to collector elements and trusses forming part of the seismic-force-resisting system.

13.11.2 *Design forces*

The earthquake design forces for structural diaphragms shall be obtained from the legally adopted general building code using the applicable provisions and load combinations.

R13.11—Structural diaphragms and trusses

R13.11.1 *Scope*

Diaphragms as used in building construction are structural elements (such as a floor or roof) that provide some or all the following functions:

(a) Support for building elements (such as walls, partitions, and cladding) resisting horizontal forces but not acting as part of the seismic-force-resisting system

(b) Transfer of lateral forces from the point of application to the vertical elements of the seismic-force-resisting system

(c) Connection of various components of the vertical seismic-force-resisting system with appropriate strength, stiffness, and toughness so the building responds as intended in the design (Wyllie 1987).

R13.11.2 *Design forces*

In the general building codes, earthquake design forces for floor and roof diaphragms typically are not computed directly during the lateral-force analysis that provides story forces and story shears. Instead, diaphragm design forces

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at each level are computed by a formula that amplifies the story forces recognizing dynamic effects and includes minimum and maximum limits. These forces are used with the governing load combinations to design diaphragms for shear and moment.

For collector elements, general building codes in use in the United States specify load combinations that amplify earthquake forces by a factor $\boldsymbol{\Omega_o}$. The forces amplified by $\boldsymbol{\Omega_o}$ are also used for local diaphragm shear force resulting from the transfer of collector forces, and for local diaphragm flexural moments resulting from any eccentricity of collector forces. The specific requirements for earthquake design forces for diaphragms and collectors depend on which general building code is used. The requirements may also vary according to the SDC.

For most concrete buildings subjected to inelastic earthquake demands, it is desirable to limit inelastic behavior of floor and roof diaphragms under the imposed earthquake forces and deformations. It is preferable for inelastic behavior to occur only in the intended locations of the vertical earthquake-force-resisting system that are detailed for ductile response, such as in the beam plastic hinges of special moment frames, or in flexural plastic hinging at the base of structural walls or in coupling beams. For buildings without long diaphragm spans between lateral-force-resisting elements, elastic diaphragm behavior is typically not difficult to achieve. For buildings where diaphragms could reach their flexural or shear strength before yielding occurs in the vertical seismic systems, designers should consider providing increased diaphragm strength.

13.11.3 *Seismic load path*

13.11.3.1 All diaphragms and their connections shall be proportioned and detailed to provide for a complete transfer of forces to collector elements and to the vertical elements of the seismic-force-resisting system.

13.11.3.2 Elements of a structural diaphragm system that are subjected primarily to axial forces and used to transfer diaphragm shear or flexural forces around openings or other discontinuities, shall comply with the requirements for collectors in 13.11.7.5 and 13.11.7.6.

R13.11.3.2 Section 13.11.3.2 applies to strut-like elements that often are present around openings, diaphragm edges, or other discontinuities in diaphragms. Figure R13.11.3.2 shows an example. Such elements can be subjected to earthquake axial forces in combination with bending and shear from earthquake or gravity loads.

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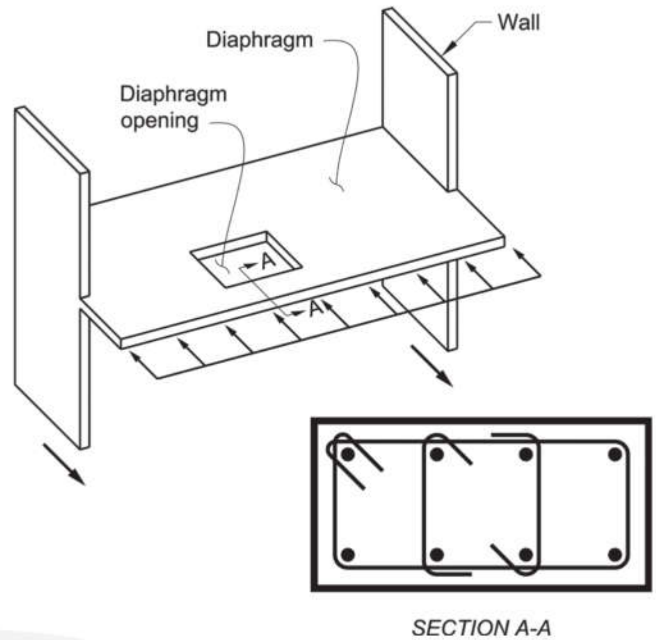

Fig. R13.11.3.2—Example of diaphragm subject to the requirements of 13.11.3.2 and showing an element having confinement as required by 13.11.7.5.

13.11.4 *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 the surface of the previously hardened concrete on which the topping slab is placed is clean, free of laitance, and intentionally roughened.

R13.11.4 *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.

13.11.5 *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 earthquake forces.

R13.11.5 *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.

13.11.6 *Minimum thickness of diaphragms*

Concrete slabs 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 earthquake forces, shall have thickness not less than 2-1/2 in.

R13.11.6 *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.

13.11.7 *Reinforcement*

R13.11.7 *Reinforcement*

13.11.7.1 The minimum reinforcement ratio for structural diaphragms shall be in conformance with 12.13. Except for post-tensioned slabs, reinforcement spacing each way in floor or roof systems shall not exceed 12 in. Where welded wire reinforcement 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

R13.11.7.1 Minimum reinforcement ratios for diaphragms correspond to the required amount of temperature and shrinkage reinforcement (12.13). The maximum spacing for web reinforcement is intended to control the width of inclined cracks. Minimum average prestress requirements (12.13.3) are considered adequate to limit the crack widths

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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.

13.11.7.2 Bonded tendons used as reinforcement to resist collector forces or diaphragm shear or flexural tension shall be proportioned such that the stress due to design earthquake forces does not exceed 60,000 psi. Precompression from unbonded tendons shall be permitted to resist diaphragm design forces if a seismic load path is provided.

13.11.7.3 All reinforcement used to resist collector forces, diaphragm shear, or flexural tension shall be developed or spliced for f_y in tension.

13.11.7.4 Type 2 splices are required where mechanical splices are used to transfer forces between the diaphragm and the vertical elements of the seismic-force-resisting system.

13.11.7.5 Collector elements with compressive stresses exceeding $0.2f_c'$ at any section shall have transverse reinforcement satisfying 13.9.6.4(c) over the length of the element. The specified transverse reinforcement is permitted to be discontinued at a section where the calculated compressive stress is less than $0.15f_c'$.

Where design forces have been amplified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $0.2f_c'$ shall be increased to $0.5f_c'$, and the limit of $0.15f_c'$ shall be increased to $0.4f_c'$.

COMMENTARY

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 (refer to 13.11.7.1) 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.[13.59] 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 13.11.7.1 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.

R13.11.7.3 Bar development and lap splices are designed according to requirements of Chapter 12 for reinforcement in tension. Reductions in development or splice length for calculated stresses less than f_y are not permitted, as indicated in 12.8.2.5.

R13.11.7.5 In documents such as the NEHRP provisions, ASCE/SEI 7, the International Building Code, and the Uniform Building Code, collector elements of diaphragms are designed for forces amplified by a factor Ω_o to account for the overstrength in the vertical elements of the seismic-force-resisting systems. The amplification factor Ω_o ranges between 2 and 3 for most concrete structures, depending on the document selected and on the type of seismic system. In some documents, the factor can be calculated based on the maximum forces that can be developed by the elements of the vertical seismic-force-resisting system.

Compressive stress calculated for the factored forces on a linearly elastic model based on gross section of the structural diaphragm is used as an index value to determine whether confining reinforcement is required. A calculated compressive stress of $0.2f_c'$ in a member, or $0.5f_c'$ for forces amplified by Ω_o, is assumed to indicate that integrity of the entire structure depends on the ability of that member to resist substantial compressive force under severe cyclic loading. Therefore, transverse reinforcement is required in such members to provide confinement for the concrete and the reinforcement.

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13.11.7.6 Longitudinal reinforcement for collector elements 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 11.4.6.3, except as required in 13.11.7.5.

COMMENTARY

R13.11.7.6 Section 13.11.7.6 is intended to reduce the possibility of bar buckling and provide adequate bar development conditions in the vicinity of splices and anchorage zones.

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13.11.8 *Flexural strength*

Diaphragms and portions of diaphragms shall be designed for flexure in accordance with 10.2 and 10.3 except that the nonlinear distribution of strain requirements of 10.2.2 for deep beams need not apply. The effects of openings shall be considered.

COMMENTARY

R13.11.8 *Flexural strength*

Flexural strength for diaphragms is calculated using the same assumptions as for walls, columns, or beams. The design of diaphragms for flexure and other actions uses the applicable load combinations of 9.2 to consider earthquake forces acting concurrently with gravity or other loads.

The influence of slab openings on flexural and shear strength is to be considered, including evaluating the potential critical sections created by the openings. Strut-and-tie models are potentially useful for designing diaphragms with openings.

Earlier design practice assumed design moments for structural diaphragms were resisted entirely by chord forces acting at opposite edges of the diaphragm. This idealization was implicit in earlier versions of the code but has been replaced by an approach in which all longitudinal reinforcement, within the limits of 13.11.7, is assumed to contribute to the flexural strength of the diaphragm. This change reduces the required area of longitudinal reinforcement concentrated near the edge of the diaphragm but should not be interpreted as a requirement to eliminate all boundary reinforcement.

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13.11.9 *Shear strength*

13.11.9.1 V_n of structural diaphragms shall not exceed

$$V_n = A_{cv}(2\lambda\sqrt{f_c'} + \rho_t f_y) \tag{13-10}$$

For cast-in-place topping slab diaphragms on precast floor or roof members, A_{cv} shall be computed using the thickness of topping slab only for noncomposite topping slab diaphragms and the combined thickness of cast-in-place and precast elements for composite topping slab diaphragms. For composite topping slab diaphragms, the value of f_c' used to determine V_n shall not exceed the smaller of f_c' for the precast members and f_c' for the topping slab.

13.11.9.2 V_n of structural diaphragms shall not exceed $8A_{cv}\sqrt{f_c'}$.

13.11.9.3 Above joints between precast elements in noncomposite and composite cast-in-place topping slab diaphragms, V_n shall not exceed

$$V_n = A_{vf} f_y \mu \tag{13-11}$$

COMMENTARY

R13.11.9 *Shear strength*

The shear strength requirements for diaphragms are similar to those for slender structural walls and are based on the shear provisions for beams. The term A_{cv} refers to the gross area of the diaphragm but may not exceed the thickness times the width of the diaphragm. This corresponds to the gross area of the effective deep beam that forms the diaphragm. Distributed slab reinforcement ρ_t used to calculate shear strength of a diaphragm in Eq. (13-10) is positioned perpendicular to the diaphragm flexural reinforcement. Section 13.11.9.2 limits the maximum shear strength of the diaphragm.

In addition to satisfying the provisions in 13.11.9.1 and 13.11.9.2, cast-in-place topping slab diaphragms must also satisfy 13.11.9.3 and 13.11.9.4. Cast-in-place topping slabs on a precast floor or roof system tend to have shrinkage cracks that are aligned with the joints between adjacent precast members. Therefore, the additional shear strength requirements for topping slab diaphragms in 13.11.9.3 are based on a shear friction model,[13.59] and the assumed crack plane corresponds to joints in the precast system along the direction of the applied shear, as shown in Fig. R11.6.4. The coefficient of friction, μ, in the shear friction model is taken equal to 1.0 for normalweight concrete due to the presence of these shrinkage cracks.

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where A_{vf} is total area of shear friction reinforcement within topping slab, including both distributed and boundary reinforcement, that is oriented perpendicular to joints in the precast system and coefficient of friction μ is 1.0λ, where λ is given in 11.6.4.3. At least one-half of A_{vf} shall be uniformly distributed along the length of the potential shear plane. Area of distributed reinforcement in topping slab shall satisfy 12.13.2.1 in each direction.

13.11.9.4 Above joints between precast elements in noncomposite and composite cast-in-place topping slab diaphragms, V_n shall not exceed the limits in 11.6.5 where A_c is computed using the thickness of the topping slab only.

13.11.10 *Construction joints*

All construction joints in diaphragms shall conform to 7.2 and contact surfaces shall be roughened as in 11.6.9.

13.11.11 *Structural trusses*

13.11.11.1 Structural truss elements with compressive stresses exceeding $0.2f_c'$ at any section shall have transverse reinforcement, as given in 13.6.4.2 through 13.6.4.4 and 13.6.4.7, over the length of the element.

13.11.11.2 All continuous reinforcement in structural truss elements shall be developed or spliced for f_y in tension.

COMMENTARY

Both distributed and boundary reinforcement in the topping slab may be considered as shear friction reinforcement A_{vf}. Boundary reinforcement within the diaphragm was called chord reinforcement in ACI 318 before 2008. Although the boundary reinforcement also resists flexural forces in the diaphragm, the reduction in the shear-friction resistance in the tension zone is offset by the increase in shear-friction resistance in the compression zone. Therefore, the area of boundary reinforcement used to resist shear friction need not be added to the area of boundary reinforcement used to resist flexural forces. The distributed topping slab reinforcement must contribute at least half of the nominal shear strength. It is assumed that connections between the precast elements do not contribute to the shear strength of the topping slab diaphragm.

Section 13.11.9.4 limits the maximum shear that may be transmitted by shear friction within a topping slab diaphragm.

13.12—Foundations

13.12.1 *Scope*

13.12.1.1 Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground in structures assigned to SDC D, E, or F shall comply with 13.12 and other applicable Code provisions.

13.12.1.2 The provisions in this section for piles, drilled piers, caissons, and slabs-on-ground shall supplement other applicable code design and construction criteria. Refer to 1.1.6 and 1.1.7.

13.12.2 *Footings, foundation mats, and pile caps*

13.12.2.1 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.

13.12.2.2 Columns designed assuming fixed-end conditions at the foundation shall comply with 13.12.2.1 and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90-degree hooks near the bottom of the foundation with the free end of the bars oriented toward the center of the column.

13.12.2.3 Columns or boundary elements of special structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse

R13.12—Foundations

R13.12.1 *Scope*

Requirements for foundations supporting buildings assigned to SDC D, E, or F were added to ACI 318-99. 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.

R13.12.2 *Footings, foundation mats, and pile caps*

R13.12.2.2 Tests (Nilsson and Loseberg 1976) have demonstrated that flexural members terminating in a footing, slab or beam (a T-joint) should have their hooks turned inward 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.

R13.12.2.3 Columns or boundary members supported close to the edge of the foundation, as often occurs near

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reinforcement in accordance with 13.6.4.2 through 13.6.4.4 provided below the top of the footing. This reinforcement shall extend into the footing, mat, or pile cap and be developed for f_y in tension.

13.12.2.4 Where earthquake effects create uplift forces in boundary elements of special structural walls or columns, flexural reinforcement shall be provided in the top of the footing, mat, or pile cap to resist actions resulting from the design load combinations, and shall not be less than required by 10.5.

13.12.3 *Grade beams and slabs-on-ground*

13.12.3.1 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.

13.12.3.2 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 and 12 in.

13.12.3.3 Grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the seismic-force-resisting system shall conform to 13.5.

13.12.3.4 Slabs-on-ground that resist seismic forces from walls or columns that are part of the seismic-force-resisting system shall be designed as structural diaphragms in accordance with 13.11. The contract documents shall clearly state that the slab-on-ground is a structural diaphragm and part of the seismic-force-resisting system.

13.12.4 *Piles, piers, and caissons*

13.12.4.1 Provisions of 13.12.4 shall apply to concrete piles, piers, and caissons supporting structures designed for earthquake resistance.

13.12.4.2 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.

13.12.4.3 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

COMMENTARY

property lines, should be detailed to prevent an edge failure of the footing, pile cap, or mat.

R13.12.2.4 The purpose of 13.12.2.4 is to emphasize that top reinforcement should be provided as well as other required reinforcement.

R13.12.3 *Grade beams and slabs-on-ground*

For seismic conditions, slabs-on-ground (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. Refer to 1.1.7.

R13.12.3.2 Grade beams between pile caps or footings can be separate beams beneath the slab-on-ground or can be a thickened portion of the slab-on-ground. The cross-sectional limitation and minimum tie requirements provide reasonable proportions.

R13.12.3.3 Grade beams resisting seismic flexural stresses from column moments should have reinforcement details similar to the beams of the frame above the foundation.

R13.12.3.4 Slabs-on-ground often act as a diaphragm to hold the building together at the ground level and minimize the effects of out-of-phase ground motion that may occur over the footprint of the building. In these cases, the slab-on-ground should be adequately reinforced and detailed. The contract documents should clearly state that these slabs-on-ground are structural members to prohibit saw cutting of the slab.

R13.12.4 *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. Refer to R1.1.6.

R13.12.4.2 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.

R13.12.4.3 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 strength. Alternatively, reinforcing bars can be cast in the upper portion of the pile,

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in the top of the pile, the grouting system shall have been demonstrated by test to develop at least $\mathbf{1.25}f_y$ of the bar.

exposed by chipping of concrete and mechanically spliced or welded to an extension.

13.12.4.4 Piles, piers, or caissons shall have transverse reinforcement in accordance with 13.6.4.2 through 13.6.4.4 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 13.12.4.4(a)

R13.12.4.4 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 Code requirement for confinement reinforcement at the top of the pile is based on numerous failures observed at this location in earthquakes. Transverse reinforcement is required in this region to provide ductile performance. Possible inelastic action in the pile at abrupt changes in soil deposits should also be considered, 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 contract documents 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 13.12.4.4 may not be available after the excess pile length is cut off.

13.12.4.5 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.

13.12.4.6 Concrete piles, piers, or caissons in foundations supporting one- and two-story stud bearing wall construction are exempt from the transverse reinforcement requirements of 13.12.4.4 and 13.12.4.5.

13.12.4.7 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.

R13.12.4.7 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.

13.13—Members not designated as part of the seismic-force-resisting system

13.13.1 *Scope*

Requirements of 13.13 apply to frame members not designated as part of the seismic-force-resisting system in structures assigned to SDC D, E, and F.

13.13.2 Members assumed not to contribute to lateral resistance, except two-way slabs without beams and wall piers, shall be detailed according to 13.13.3 or 13.13.4 depending on the magnitude of moments induced in those members when subjected to the design displacement $\boldsymbol{\delta_u}$. If effects of $\boldsymbol{\delta_u}$ are not explicitly checked, it shall be permitted to apply the requirements of 13.13.4. Slab-column connections of two-way slabs without beams shall satisfy the requirements of 13.13.6. Wall piers shall satisfy the requirements of 13.13.7.

13.13.3 Where the induced moments and shears under design displacements $\boldsymbol{\delta_u}$ combined with the factored gravity moments and shears do not exceed the design moment and shear strength of the frame member, the conditions of 13.13.3.1, 13.13.3.2, and 13.13.3.3 shall be satisfied. The gravity load combinations of $(\mathbf{1.2}\boldsymbol{D} + \mathbf{1.0}\boldsymbol{L} + \mathbf{0.2}\boldsymbol{S})$ or $\mathbf{0.9}\boldsymbol{D}$, whichever is critical, shall be used. The load factor on the live load $\boldsymbol{L}$ shall be permitted to be reduced to 0.5 except for

R13.13—Members not designated as part of the seismic-force-resisting system

This section applies only to structures assigned to SDC D, E, or F. Model building codes, such as the 2006 IBC, require all structural members not designated as a part of the seismic-force-resisting system to be designed to support gravity loads while subjected to the design displacement. For concrete structures, the provisions of 13.13 satisfy this requirement for columns, beams, slabs, and wall piers of the gravity system. The design displacement is defined in 2.2.

The provisions of 13.13 are based on the principle enabling flexural yielding of columns, beams, slabs, and wall piers under the design displacement, by providing sufficient confinement and shear strength in elements that yield. By the provisions of 13.13.2 through 13.13.4 and 13.13.7, columns, beams, and wall piers, respectively, are assumed to yield if the combined effects of factored gravity loads and design displacements exceed the strengths specified in those provisions, or if the effects of design displacements are not calculated. Requirements for transverse reinforcement and shear strength vary with the axial load on the member and whether or not the member yields under the design displacement.

Models used to determine design displacement of buildings should be chosen to produce results that conservatively bound the values expected during the design earthquake and should include, as appropriate, effects of concrete cracking,

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garages, areas occupied as places of public assembly, and all areas where L is greater than 100 lb/ft^2.

13.13.3.1 Members with factored gravity axial forces not exceeding $A_g f_c'/10$ shall satisfy 13.5.2.1. Stirrups shall be spaced not more than $d/2$ throughout the length of the member.

13.13.3.2 Members with factored gravity axial forces exceeding $A_g f_c'/10$ shall satisfy 13.6.3.1, 13.6.4.2, and 13.6.5. The maximum longitudinal spacing of ties shall be s_o for the full member length. Spacing s_o shall not exceed the smaller of six diameters of the smallest longitudinal bar enclosed and 6 in.

13.13.3.3 Members with factored gravity axial forces exceeding $0.35P_o$ shall satisfy 13.13.3.2 and 13.6.4.7. The amount of transverse reinforcement provided shall be one-half of that required by 13.6.4.4 but shall not be spaced greater than s_o for the full member length.

13.13.4 If the induced moment or shear under design displacements, δ_u, exceeds ϕM_n or ϕV_n of the frame member, or if induced moments are not calculated, the conditions of 13.13.4.1, 13.13.4.2, and 13.13.4.3 shall be satisfied.

13.13.4.1 Materials shall satisfy 13.1.4.2, 13.1.4.3, 13.1.5.2, 13.1.5.4, and 13.1.5.5. Mechanical splices shall satisfy 13.1.6 and welded splices shall satisfy 13.1.7.1.

13.13.4.2 Members with factored gravity axial forces not exceeding $A_g f_c'/10$ shall satisfy 13.5.2.1 and 13.5.4. Stirrups shall be spaced at not more than $d/2$ throughout the length of the member.

13.13.4.3 Members with factored gravity axial forces exceeding $A_g f_c'/10$ shall satisfy 13.6.3, 13.6.4, 13.6.5, and 13.7.3.1.

13.13.5 Precast concrete frame members assumed not to contribute to lateral resistance, including their connections, shall satisfy (a), (b), and (c), in addition to 13.13.2 through 13.13.4:

(a) Ties specified in 13.13.3.2 shall be provided over the entire column height, including the depth of the beams

(b) Structural integrity reinforcement, as specified in 17.5, shall be provided

(c) Bearing length at support of a beam shall be at least 2 in. longer than determined from calculations using bearing strength values from 10.14

13.13.6 For slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the requirements of 11.11.3 and 11.11.5 and providing V_s not

COMMENTARY

foundation flexibility, and deformation of floor and roof diaphragms.

R13.13.5 Damage to some buildings with precast concrete gravity systems during the 1994 Northridge earthquake was attributed to several factors addressed in 13.13.5. 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 conservative for the ground motions expected for structures assigned to SDC D, E, or F. In addition to the provisions of 13.13.5, precast frame members assumed not to contribute to lateral resistance should also satisfy 13.13.2 through 13.13.4, as applicable.

R13.13.6 Provisions for shear reinforcement at slab-column connections were added in ACI 318-05 to reduce the likelihood of slab punching shear failure. The shear

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less than $3.5\sqrt{f_c'}\,b_o d$ shall extend at least four times the slab thickness from the face of the support, unless either (a) or (b) is satisfied:

(a) The requirements of 11.11.7 using the design shear V_{ug} and the induced moment transferred between the slab and column under the design displacement

(b) The design story drift ratio does not exceed the larger of 0.005 and $[0.035 - 0.05(V_{ug}/\phi V_c)]$

Design story drift ratio shall be taken as the larger of the design story drift ratios of the adjacent stories above and below the slab-column connection. V_c is defined in 11.11.2. V_{ug} is the factored shear force on the slab critical section for two-way action, calculated for the load combination $1.2D + 1.0L + 0.2S$.

The load factor on the live load L shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 100 lb/ft^2.

COMMENTARY

reinforcement is required unless either 13.13.6 (a) or (b) is satisfied.

Section 13.13.6(a) requires calculation of shear stress due to the factored shear force and induced moment according to 11.11.7. The induced moment is the moment that is calculated to occur at the slab-column connection when subjected to the design displacement. Section 13.5.1.2 and the accompanying Commentary provide guidance on selection of the stiffness of the slab-column connection for the purpose of this calculation.

Section 13.13.6(b) does not require the calculation of induced moments, and is based on research[13.61,13.62] that identifies the likelihood of punching shear failure considering the story drift ratio and shear due to gravity loads. Figure R13.13.6 illustrates the requirement. The requirement can be satisfied by adding slab shear reinforcement, increasing slab thickness, changing the design to reduce the design story drift ratio, or a combination of these.

If column capitals, drop panels, shear caps, or other changes in slab thickness are used, the requirements of 13.13.6 are evaluated at all potential critical sections, as required by 11.11.1.2.

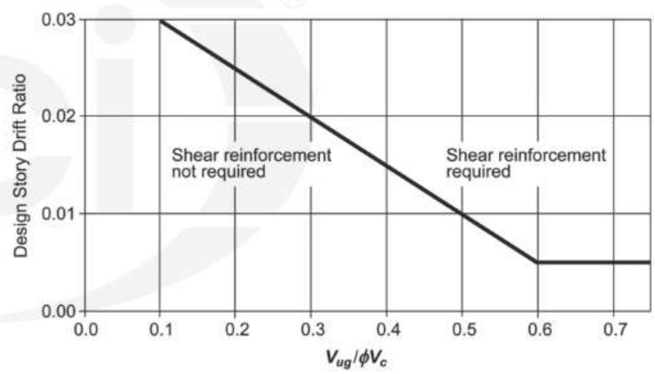

Fig. R13.13.6—Illustration of the criterion of 13.13.6(b).

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13.13.7 Wall piers not designated as part of the seismic-force-resisting system shall satisfy the requirements of 13.9.8. Where the legally adopted general building code includes provisions to account for overstrength of the seismic-force-resisting system, it shall be permitted to determine the design shear force as Ω_o times the shear induced under design displacements, δ_u.

COMMENTARY

R13.13.7 Section 13.9.8 requires that the design shear force be determined according to 13.6.5.1, which in some cases may result in unrealistically large forces. As an alternative, the design shear force can be determined as the product of an overstrength factor and the shear induced when the wall pier is displaced by δ_u. The overstrength factor Ω_o included in documents such as the NEHRP provisions, ASCE/SEI 7, and the International Building Code can be used for this purpose.

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CHAPTER 14—TWO-WAY SLAB SYSTEMS

14.1—Scope

14.1.1 Provisions of Chapter 14 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.

14.1.2 For a slab system supported by columns or walls, dimensions c_1, c_2, and 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 or shear cap if present, 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 degrees to the axis of the column.

14.1.3 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 14.

14.1.4 Minimum thickness of slabs designed in accordance with Chapter 14 shall be as required by 9.5.3.

COMMENTARY

CHAPTER R14—TWO-WAY SLAB SYSTEMS

R14.1—Scope

The design methods given in Chapter 14 are based on analysis of the results of an extensive series of tests (Hatcher et al. 1965, 1969; Guralnick and LaFraugh 1963; Jirsa et al. 1966; Gamble et al. 1969; Vanderbilt et al. 1969; Xanthakis and Sozen 1963) and the well-established performance record of various slab systems. Much of Chapter 14 is concerned with the selection and distribution of flexural reinforcement. Safety of a slab system requires consideration of 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.

The requirements of ACI 318-11 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.

The fundamental design principles contained in Chapter 14 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 14 applies. General characteristics of slab systems that may be designed according to Chapter 14 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 slabs-on-ground that do not transmit vertical loads from other parts of the structure to the soil.

For slabs with beams, the explicit design procedures of Chapter 14 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 14. 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 (refer to 14.2.3). Wall-like columns less than a full panel length can be treated as columns.

Design aids for use in the engineering analysis and design of two-way slab systems are given in the *ACI Design Handbook* (ACI SP-17). Design aids are provided to simplify application of the direct design and equivalent frame methods of Chapter 14.

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14.2—General

14.2.1 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.

14.2.2 Middle strip is a design strip bounded by two column strips.

14.2.3 A panel is bounded by column, beam, or wall centerlines on all sides.

14.2.4 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.

14.2.5 When used to reduce the amount of negative moment reinforcement over a column or minimum required slab thickness, a drop panel shall:

14.2.5.1 Project below the slab at least one-fourth of the adjacent slab thickness; and

14.2.5.2 Extend in each direction from the centerline of support a distance not less than one-sixth the span length measured from center-to-center of supports in that direction.

14.2.6 When used to increase the critical concrete section for shear at a slab-column joint, a shear cap shall project below the slab and extend a minimum horizontal distance from the face of the column that is equal to the thickness of the projection below the slab soffit.

14.3—Slab reinforcement

14.3.1 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 12.13.1.

14.3.2 Spacing of reinforcement at critical sections 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 12.13.

COMMENTARY

R14.2—General

R14.2.3 A panel includes all flexural elements between column centerlines. Thus, the column strip includes the beam, if any.

R14.2.4 For monolithic or fully composite construction, the beams include portions of the slab as flanges. Two examples of the rule are provided in Fig. R14.2.4.

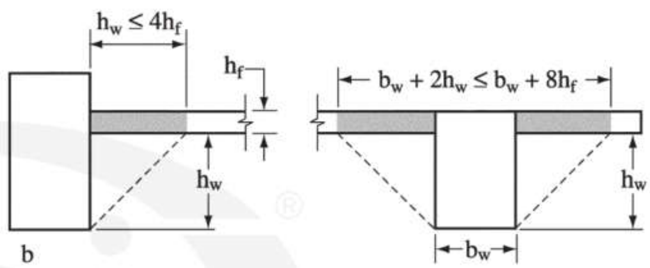

Fig. R14.2.4—Examples of the portion of slab to be included with the beam under 14.2.4

R14.2.5 and R14.2.6 Drop panel dimensions specified in 14.2.5 are necessary when reducing the amount of negative moment reinforcement following 14.3.7 or to satisfy some minimum slab thicknesses permitted in 9.5.3. If the dimensions are less than specified in 14.2.5, the projection may be used as a shear cap to increase the shear strength of the slab. For slabs with changes in thickness it is necessary to check the shear strength at several sections. Refer to 11.11.1.2.

R14.3—Slab reinforcement

R14.3.2 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 in joists or waffle slabs. This limitation is to ensure slab action, cracking, and provide for the

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14.3.3 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.

14.3.4 Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored in spandrel beams, columns, or walls, and shall be developed at face of support according to provisions of Chapter 12.

14.3.5 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.

14.3.6 At exterior corners of slabs supported by edge walls or where one or more edge beams have a value of $\boldsymbol{\alpha_f}$ greater than 1.0, top and bottom slab reinforcement shall be provided at exterior corners in accordance with 14.3.6.1 through 14.3.6.4.

14.3.6.1 Corner reinforcement in both top and bottom of slab shall be sufficient to resist a moment per unit width equal to the maximum positive moment per unit width in the slab panel.

14.3.6.2 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.

COMMENTARY

possibility of loads concentrated on small areas of the slab. Refer also to R10.6.

A 12 in. maximum spacing of reinforcement is required in liquid-containing structures to ensure adequate crack control.

R14.3.3, R14.3.4, and R14.3.5 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.

R14.3.6 Unrestrained corners of two-way slabs tend to lift when loaded. If this lifting tendency is restrained by edge walls or beams, bending moments result in the slab. This

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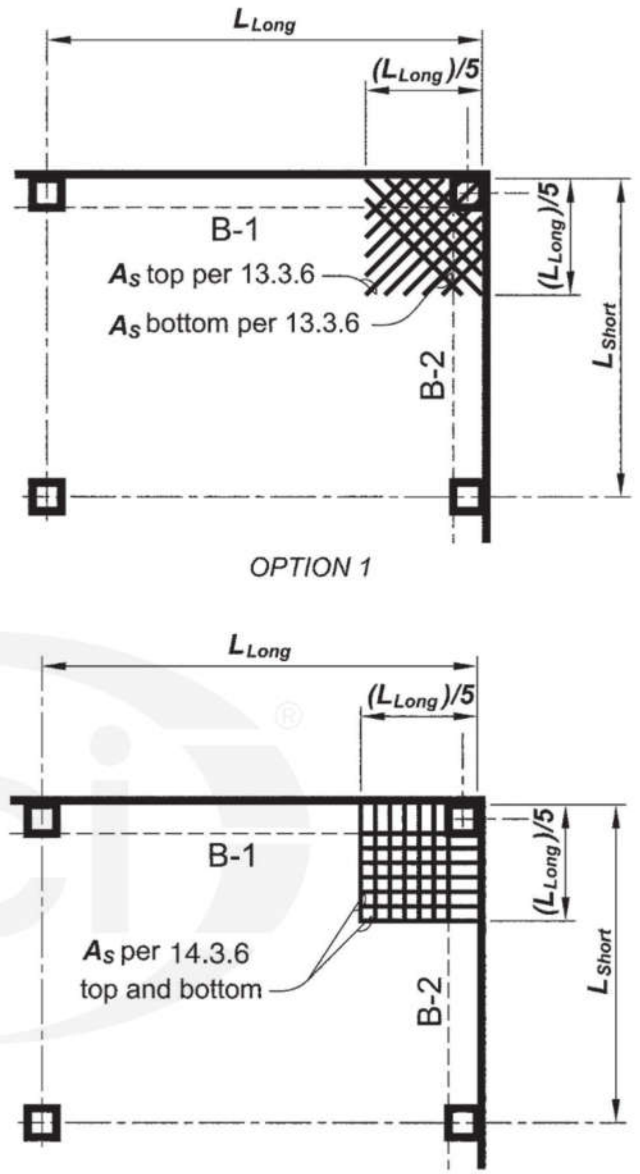

Notes:
1. Applies where B-1 or B-2 has α_f >1.0
2. Max. bar spacing 2h, where h = slab thickness.

Fig. R14.3.6—Slab corner reinforcement.

14.3.6.3 Corner reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth the longer span.

14.3.6.4 Corner reinforcement shall be placed parallel to the diagonal in the top of the slab and perpendicular to the diagonal in the bottom of the slab. Alternatively, reinforcement shall be placed in two layers parallel to the sides of the slab in both the top and bottom of the slab.

14.3.7 When a drop panel is used to reduce the amount of negative moment reinforcement over the column of a flat

section requires steel to resist these moments and control cracking. Reinforcement provided for flexure in the primary directions may be used to satisfy this requirement. Refer to Fig. R14.3.6.

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slab, the dimensions of the drop panel shall be in accordance with 14.2.5. In calculating required slab reinforcement, the thickness of the drop panel below the slab shall not be assumed to be greater than one-fourth the distance from the edge of drop panel to the face of column or column capital.

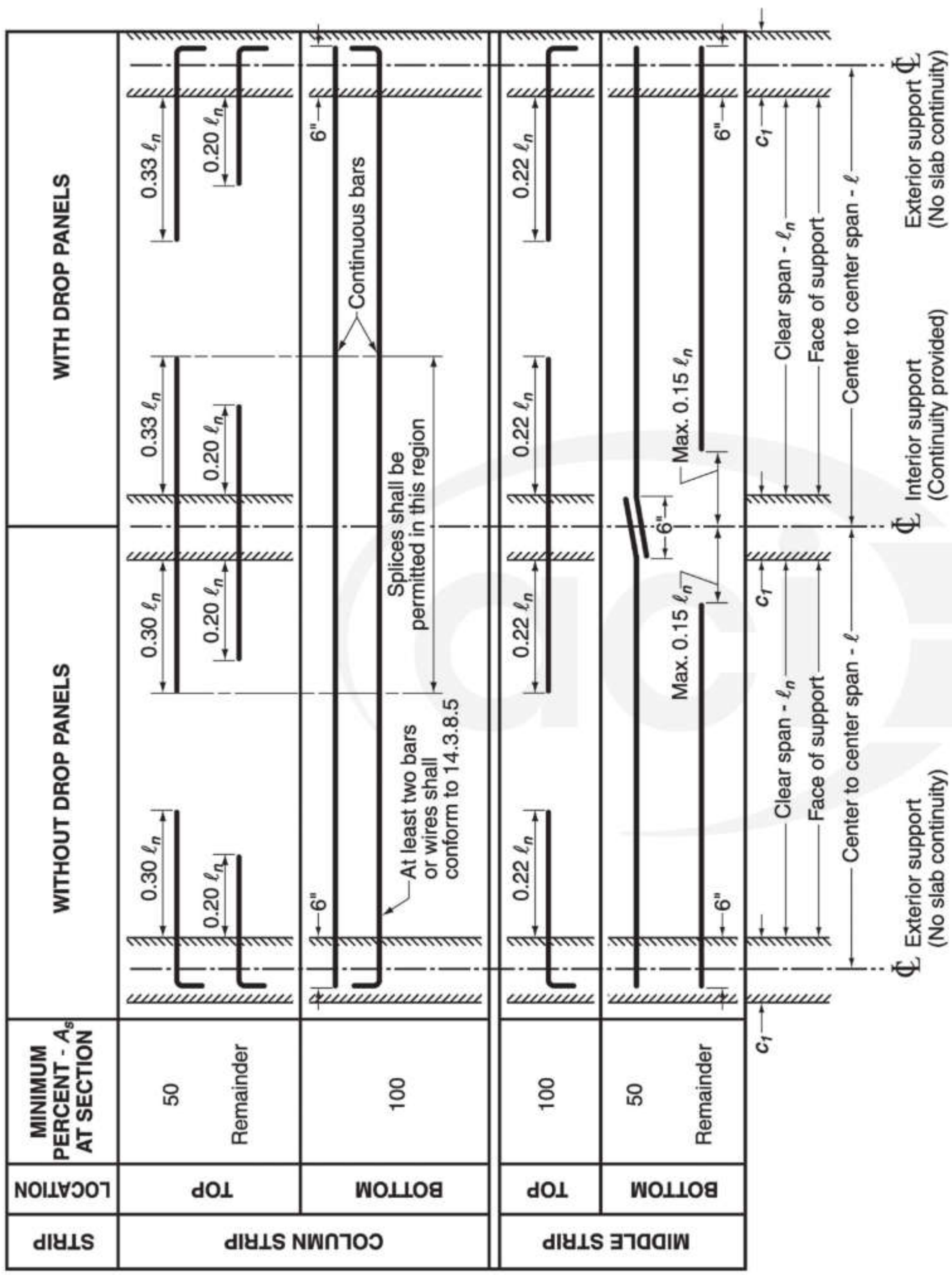

Fig. 14.3.8—Minimum extensions for reinforcement in slabs without beams (refer to 12.11.1 for reinforcement extension into supports).

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14.3.8 *Details of reinforcement in slabs without beams*

14.3.8.1 In addition to the other requirements of 14.3, reinforcement in slabs without beams shall have minimum extensions as prescribed in Fig. 14.3.8.

14.3.8.2 Where adjacent spans are unequal, extensions of negative moment reinforcement beyond the face of support as prescribed in Fig. 14.3.8 shall be based on requirements of the longer span.

14.3.8.3 Bent bars shall be permitted only when span-depth ratio permits use of bends of 45 degrees or less.

14.3.8.4 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. 14.3.8.

14.3.8.5 All bottom bars or wires within the column strip, in each direction, shall be continuous or spliced with Class B tension lap splices or with mechanical or welded splices in accordance with 12.9.1.3. Splices shall be located as shown in Fig. 14.3.8. At least two of the column strip bottom bars or wires in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at exterior supports.

14.3.8.6 In slabs with shearheads and in lift-slab construction where it is not practical to pass the bottom bars required by 14.3.8.5 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 B tension lap splice or with mechanical or welded splices satisfying 12.9.1.3. At exterior columns, the reinforcement shall be anchored at the shearhead or lifting collar.

14.4—Openings in slab systems

14.4.1 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 set forth in 9.2 and 9.3, and that all serviceability conditions, including the limits on deflections, are met.

14.4.2 As an alternate to special analysis as required by 14.4.1, openings shall be permitted in slab systems without beams only in accordance with 14.4.2.1 through 14.4.2.4.

14.4.2.1 Openings of any size shall be permitted in the area common to intersecting middle strips, provided total

COMMENTARY

R14.3.8 *Details of reinforcement in slabs without beams*

In ACI 318-89, bent bars were removed from Fig. 14.3.8. This was done because bent bars are seldom used and are difficult to place properly. Bent bars are permitted, however, if they comply with 14.3.8.3. Refer to 14.4.8 of ACI 318-83.

R14.3.8.4 For moments resulting from combined lateral and gravity loadings, the minimum lengths and extensions of bars in Fig. 14.3.8 may not be sufficient.

R14.3.8.5 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 strength following a single punching shear failure at a single support (Mitchell and Cook 1984). In ACI 318-02, mechanical and welded splices were explicitly recognized as alternative methods of splicing reinforcement.

R14.3.8.6 In ACI 318-02, 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 should 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.

R14.4—Openings in slab systems

Refer to R11.11.6.

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amount of reinforcement required for the panel without the opening is maintained.

14.4.2.2 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.

14.4.2.3 In the area common to one column strip and one middle strip, not more than one-fourth 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.

14.4.2.4 Shear requirements shall be in accordance with 11.11.6.

14.5—Design procedures

14.5.1 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 limits on deflections, are met.

14.5.1.1 Design of a slab system for gravity loads, and uniform fluid pressures, including the slab and beams, if any, between supports and supporting columns or walls forming orthogonal frames, by either the direct design method of 14.6 or the equivalent frame method of 14.7 shall be permitted.

COMMENTARY

R14.5—Design procedures

R14.5.1 This section permits a design to be based directly on fundamental principles of structural mechanics, provided it can be demonstrated explicitly that all strength 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 design of a slab system involves more than its analysis, and any deviations in physical dimensions of the slab from common practice should be justified 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. PCA (1969) discusses the analysis and design of rectangular tanks, including multi-cell tanks. Moody (1960) provides additional tables for a greater variety of loading cases and boundary conditions.

R14.5.1.1 For gravity load analysis of two-way slab systems, two analysis methods are given in 14.6 and 14.7. 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

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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.

14.5.1.2 For lateral loads, analysis of frames shall take into account effects of cracking and reinforcement on stiffness of frame members.

R14.5.1.2 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 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 structure may be modeled 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 (Carpenter et al 1973; Morrison and Sozen 1981). The selected approach should recognize effects of cracking as well as parameters such as ℓ_2/ℓ_1, c_2/ℓ_1, and c_2/c_1. Some of the available approaches are summarized in Vanderbilt and Corley (1983), which includes a 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 nonprestressed slabs, it is normally appropriate to reduce slab bending stiffness to between one-half and one-fourth of the uncracked stiffness. For prestressed slabs, stiffnesses greater than those of cracked, nonprestressed 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.

14.5.1.3 Combining the results of the gravity load analysis with the results of the lateral load analysis shall be permitted.

14.5.2 The slab and beams, if any, between supports shall be proportioned for factored moments prevailing at every section.

14.5.3 When gravity load, wind, earthquake, or other lateral loads cause transfer of moment between slab and column, a fraction of the unbalanced moment shall be transferred by flexure in accordance with 14.5.3.2 and 14.5.3.3.

R14.5.3 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, $1.5h$, on each side of the column. The calculated shear stresses in the slab around the column are required to conform to the requirements of 11.11.2. Refer to R11.11.1.2 and R11.11.2.1 for more details on application of this section.

14.5.3.1 The fraction of unbalanced moment not transferred by flexure shall be transferred by eccentricity of shear in accordance with 11.11.7.

14.5.3.2 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 thickness **(1.5*h*)** outside opposite faces of the column or capital, where M_u is the factored moment to be transferred and

$$\gamma_f = \frac{1}{1+(2/3)\sqrt{b_1/b_2}} \tag{14-1}$$

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14.5.3.3 *Transfer of unbalanced moments to columns*

For nonprestressed slabs with unbalanced moments transferred between the slab and columns, it shall be permitted to increase the value of γ_f given by Eq. (14-1) in accordance with the following:

14.5.3.3.1 For edge columns with unbalanced moments about an axis parallel to the edge, $\gamma_f = \mathbf{1.0}$ provided that V_u at an edge support does not exceed $\mathbf{0.75}\phi V_c$, or at a corner support does not exceed $\mathbf{0.5}\phi V_c$.

14.5.3.3.2 For unbalanced moments at interior supports, and for edge columns with unbalanced moments about an axis perpendicular to the edge, increase γ_f to as much as 1.25 times the value from Eq. (14-1), but not more than $\gamma_f = \mathbf{1.0}$, provided that V_u at the support does not exceed $\mathbf{0.4}\phi V_c$. The net tensile strain ε_t calculated for the effective slab width defined in 14.5.3.2 shall not be less than 0.010.

The value of V_c in items (a) and (b) shall be calculated in accordance with 11.11.2.1.

COMMENTARY

R14.5.3.3 ACI 318-89 procedures remain unchanged, except that under certain conditions it 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 ACI 318-95 recognized, to some extent, design practices prior to the ACI 318-71 Code (Grossman 1989).

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 11.11.2.1 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 (Moehle 1988; ACI 352.1R). Note that as $\gamma_v M_u$ is decreased, $\gamma_f M_u$ is increased.

Evaluation of tests of interior supports indicate that some flexibility in distributing unbalanced moments transferred by shear and flexure is 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 11.11.2.1.

When the factored shear for a slab-column connection is large, the column-slab joint cannot always develop all the reinforcement provided in the effective width. The modifications for interior slab-column connections in 14.5.3.3 are permitted only when the reinforcement (within the effective width) required to develop the unbalanced moment $\gamma_f M_u$ has a net tensile strain E_t not less than 0.010. The use of Eq. (14-1) without the modification permitted in 14.5.3.3 will generally indicate overstress conditions on the joint. The provisions of 14.5.3.3 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 approximately 2 has been observed to be appropriate.

For ACI 318-08, two changes were introduced to 14.5.3.3: 1) the limitation for the amount of reinforcement in the effective slab width to 37.5 percent of the balanced steel ratio was updated to refer to a minimum net tensile strain of 0.010 to be consistent with the unified design approach adopted in ACI 318-02; and 2) the requirement for the minimum net tensile strain was eliminated for moment transfer about the slab edge for edge and corner connections based on the original recommendations from ACI 352.1R.

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14.5.3.4 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 14.5.3.2.

14.5.4 Design for transfer of load from slabs to supporting columns or walls through shear and torsion shall be in accordance with Chapter 11.

14.6—Direct design method

14.6.1 *Limitations*

Design of slab systems within the limitations of 14.6.1.1 through 14.6.1.8 by the direct design method shall be permitted.

14.6.1.1 There shall be a minimum of three continuous spans in each direction.

14.6.1.2 Panels shall be rectangular, with a ratio of longer to shorter span center-to-center of supports within a panel not greater than 2.

14.6.1.3 Successive span lengths center-to-center of supports in each direction shall not differ by more than one-third the longer span.

14.6.1.4 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.

COMMENTARY

R14.6—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 (refer to 14.6.2)
2) Distribution of the total factored static moment to negative and positive sections (refer to 14.6.3)
3) Distribution of the negative and positive factored moments to the column and middle strips and to the beams, if any (refer to 14.6.4 through 14.6.6). The distribution of moments to column and middle strips is also used in the equivalent frame method (refer to 14.7)

R14.6.1 *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 slab systems to be designed using the direct design method should conform to the limitations in this section.

R14.6.1.1 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.

R14.6.1.2 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.

R14.6.1.3 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. 14.3.8.

R14.6.1.4 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.

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14.6.1.5 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 14.6.1.5 and when using Eq. (14-7) 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 load factor applicable to fluid loads, per Chapter 9. If fluid pressures do not act simultaneously on all panels, live load, including that resulting from fluid pressures, shall not exceed two times dead load.

14.6.1.6 For a panel with beams between supports on all sides, Eq. (14-2) shall be satisfied for beams in the two perpendicular directions

$$0.2 \le \frac{\alpha_{f1}\ell_2^2}{\alpha_{f2}\ell_1^2} \le 5.0 \qquad (14\text{-}2)$$

where $\boldsymbol{\alpha_{f1}}$ and $\boldsymbol{\alpha_{f2}}$ are calculated in accordance with Eq. (14-3).

$$\alpha_f = \frac{E_{cb}I_b}{E_{cs}I_s} \qquad (14\text{-}3)$$

14.6.1.7 Moment redistribution as permitted by 8.4 shall not be applied for slab systems designed by the Direct Design Method. Refer to 14.6.7.

14.6.1.8 Variations from the limitations of 14.6.1 shall be permitted if demonstrated by analysis in accordance with the requirements of 14.5.1.

14.6.2 *Total factored static moment for a span*

COMMENTARY

R14.6.1.5 The direct design method is based on tests (Jirsa et al. 1969) 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 (refer to 16.10) involve application of known column loads. Therefore, even where the soil reaction is assumed to be uniform, a frame analysis should be performed.

In ACI 318-95, the limit of applicability of the direct design method for ratios of live load to dead load was reduced from 3 to 2. Where the live-to-dead load ratio will be less than 2, it will not be necessary to check the effects of pattern loading.

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.

R14.6.1.6 The elastic distribution of moments will deviate significantly from those assumed in the direct design method unless the requirements for stiffness are satisfied.

R14.6.1.7 Moment redistribution as permitted by 8.4 is not intended for use where approximate values for bending moments are used. For the direct design method, 10 percent modification is allowed by 14.6.7.

R14.6.1.8 It 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 nonmovable load (such as a water reservoir in which the load on all panels is expected to be the same), the live load limitation of 14.6.1.5 need not be satisfied.

R14.6.2 *Total factored static moment for a span*

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14.6.2.1 Total factored static moment M_o for a span shall be determined in a strip bounded laterally by centerline of panel on each side of centerline of supports.

14.6.2.2 Absolute sum of positive and average negative factored moments in each direction shall not be less than

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} \tag{14-4}$$

where ℓ_n is length of clear span in direction that moments are being determined.

14.6.2.3 Where the transverse span of panels on either side of the centerline of supports varies, ℓ_2 in Eq. (14-4) shall be taken as the average of adjacent transverse spans.

14.6.2.4 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. (14-4).

14.6.2.5 Clear span ℓ_n shall extend from face to face of columns, capitals, brackets, or walls. Value of ℓ_n used in Eq. (14-4) 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.

14.6.3 *Negative and positive factored moments*

14.6.3.1 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.

14.6.3.2 In an interior span, total static moment M_o shall be distributed as follows:

Negative factored moment: 0.65
Positive factored moment: 0.35

14.6.3.3 In an end span, total factored static moment M_o shall be distributed as shown in the table above.

COMMENTARY

R14.6.2.2 Equation (14-4) follows directly from Nichol's (1914) derivation with the simplifying assumption that the reactions are concentrated along the faces of the support perpendicular to the span considered. In general, it will be expedient to calculate static moments for two adjacent half panels that include a column strip with a half middle strip along each side.

R14.6.2.5 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. R14.6.2.5

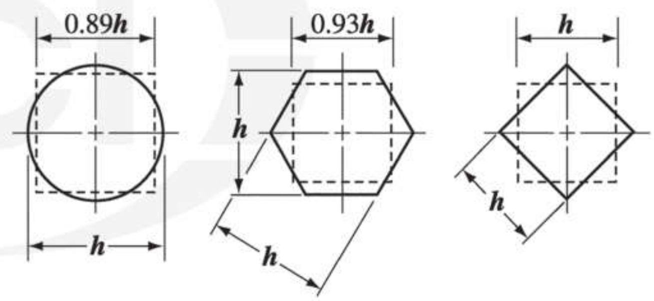

Fig. R14.6.2.5—Examples of equivalent square section for supporting members.

R14.6.3 *Negative and positive factored moments*

R14.6.3.3 The moment coefficients for an end span are based on the equivalent column stiffness expressions from Corley et al. (1961), Jirsa et al. (1963), and Corley and Jirsa (1970). The coefficients for an unrestrained edge would be used, for example, if the slab were simply supported on

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	(1)	(2)	(3)	(4)	(5)
			Slab without beams between interior supports		
	Exterior edge unrestrained	Slab with beams between all supports	Without edge beam	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

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 to 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 strength 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) of the table 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 ACI 318-77, 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 14.6.3.3.

14.6.3.4 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.

R14.6.3.4 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.

14.6.3.5 Edge beams or edges of slab shall be proportioned to resist in torsion their share of exterior negative factored moments.

R14.6.3.5 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.

14.6.3.6 The gravity load moment to be transferred between slab and edge column in accordance with 14.5.3.1 shall be $0.3M_o$.

14.6.4 *Factored moments in column strips*

R14.6.4, R14.6.5, and R14.6.6 *Factored moments in column strips, beams, and middle strips*

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14.6.4.1 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
$(\alpha_{f1}\ell_2/\ell_1) = 0$	75	75	75
$(\alpha_{f1}\ell_2/\ell_1) \geq 1.0$	90	75	45

Linear interpolations shall be made between values shown.

14.6.4.2 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
$(\alpha_{f1}\ell_2/\ell_1) = 0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	075	075	075
$(\alpha_{f1}\ell_2/\ell_1) \geq 1.0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	090	075	045

Linear interpolations shall be made between values shown, where β_t is calculated in Eq. (14-5) and C is calculated in Eq. (14-6).

$$\beta_t = \frac{E_{cb}C}{2E_{cs}I_s} \tag{14-5}$$

$$C = \Sigma\left(1 - 0.63\frac{x}{y}\right)\frac{x^3y}{3} \tag{14-6}$$

The constant C for T- or L-sections shall be permitted to be evaluated by dividing the section into separate rectangular parts, as defined in 14.2.4, and summing the values of C for each part.

14.6.4.3 Where supports consist of columns or walls extending for a distance equal to or greater than three-fourths the span length ℓ_2 used to compute M_o, negative moments shall be considered uniformly distributed across ℓ_2.

14.6.4.4 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
$(\alpha_{f1}\ell_2/\ell_1) = 0$	60	60	60
$(\alpha_{f1}\ell_2/\ell_1) \geq 1.0$	90	75	45

Linear interpolations shall be made between values shown.

14.6.4.5 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.

COMMENTARY

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 (Gamble 1972) 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. (14-4) 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.

R14.6.4.2 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 $\alpha_{f1}\ell_2/\ell_1$ value greater than 1. 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 that is monolithic with the slab.

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14.6.5 *Factored moments in beams*

14.6.5.1 Beams between supports shall be proportioned to resist 85 percent of column strip moments if $(\alpha_{f1}\ell_2/\ell_1)$ is equal to or greater than 1.0.

14.6.5.2 For values of $\alpha_{f1}\ell_2/\ell_1$ between 1.0 and zero, proportion of column strip moments resisted by beams shall be obtained by linear interpolation between 85 and zero percent.

14.6.5.3 In addition to moments calculated for uniform loads according to 14.6.2.2, 14.6.5.1, and 14.6.5.2, 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.

14.6.6 *Factored moments in middle strips*

14.6.6.1 That portion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to corresponding half middle strips.

14.6.6.2 Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.

14.6.6.3 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.

14.6.7 *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, M_o, in the direction considered is not less than that required by Eq. (14-4).

COMMENTARY

R14.6.5 *Factored moments in beams*

Loads assigned directly to beams are in addition to the uniform dead load of the slab; uniform superimposed dead loads, such as the ceiling, floor finish, or assumed equivalent partition loads; and uniform live loads. All these loads are normally included with w_u in Eq. (14-4). 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 14.2.4 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.

R14.6.7 *Modification of factored moments*

This section permits a reduction of 10 percent in negative or positive factored moments, calculated in accordance with 14.6.3, provided that the total static moment for a panel in the direction considered is not less than M_o required by Eq. (14-4). This is intended to recognize a limited amount of inelastic behavior and moment redistribution can occur in slabs that were analyzed with the Direct Design Method.

R14.6.8 *Factored shear in slab systems with beams*

The tributary area for computing shear on an interior beam is shown shaded in Fig. R14.6.8. If the stiffness for the beam $\alpha_{f1}\ell_2/\ell_1$ 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

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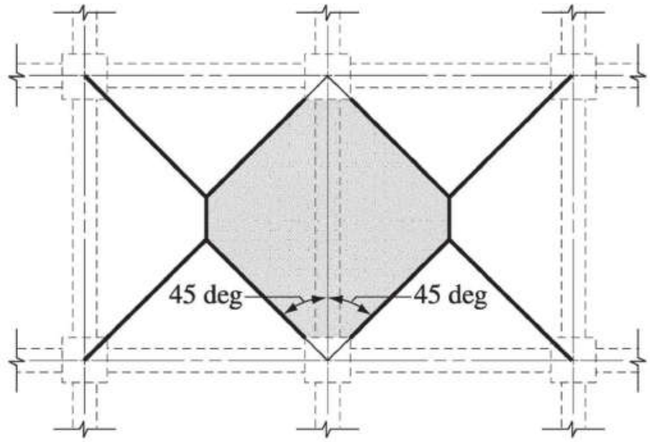

Fig. R14.6.8—Tributary area for shear on an interior beam.

14.6.8 *Factored shear in slab systems with beams*

14.6.8.1 Beams with $\alpha_{f1}\ell_2/\ell_1$ equal to or greater than 1.0 shall be proportioned to resist shear caused by factored loads on tributary areas which are bounded by 45-degree lines drawn from the corners of the panels and the centerlines of the adjacent panels parallel to the long sides.

14.6.8.2 In proportioning beams with $\alpha_{f1}\ell_2/\ell_1$ less than 1.0 to resist shear, linear interpolation, assuming beams carry no load at $\alpha_{f1} = 0$, shall be permitted.

14.6.8.3 In addition to shears calculated according to 14.6.8.1 and 14.6.8.2, beams shall be proportioned to resist shears caused by factored loads applied directly on beams.

14.6.8.4 Calculation of slab shear strength on the assumption that load is distributed to supporting beams in accordance with 14.6.8.1 or 14.6.8.2 shall be permitted. Resistance to total shear occurring on a panel shall be provided.

14.6.8.5 Shear strength shall satisfy the requirements of Chapter 11.

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 14.6.8.4. Sections 14.6.8.1 through 14.6.8.3 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.

14.6.9 *Factored moments in columns and walls*

14.6.9.1 Columns and walls built integrally with a slab system shall resist moments caused by factored loads on the slab system.

14.6.9.2 At an interior support, supporting elements above and below the slab shall resist the factored moment specified by Eq. (14-7) in direct proportion to their stiffnesses unless a general analysis is made.

$$M_u = 0.07[(w_{Du} + 0.5w_{Lu})\ell_2\ell_n^2 - w_{Du}'\ell_2'(\ell_n')^2] \quad (14\text{-}7)$$

where w_{Du}', ℓ_2', and ℓ_n' refer to shorter span.

R14.6.9 *Factored moments in columns and walls*

Equation (14-7) refers to two adjoining spans, with one span longer than the other, and 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_{Lu} in Eq. (14-7) 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 $0.5w_{Lu}$ in Eq. (14-7).

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 in the contract documents, such as concentration of reinforcement over the column by closer spacing or additional reinforcement.

14.7—Equivalent frame method

14.7.1 Design of slab systems by the equivalent frame method shall be based on assumptions given in 14.7.2

R14.7—Equivalent frame method

The equivalent frame method involves the representation of the three-dimensional slab system by a series of two-

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through 14.7.6, and all sections of slabs and supporting members shall be proportioned for moments and shears thus obtained.

COMMENTARY

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 14.6.4 (column strips), 14.6.5 (beams), and 14.6.6 (middle strips). The equivalent frame method is based on studies reported in Corley et al. (1961), Jirsa et al. (1963), and Corley and Jirsa (1970). Many of the details of the equivalent frame method given in the Commentary in ACI 318-89 were removed in ACI 318-95.

14.7.1.1 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.

R14.7.1.1 Metal column capitals (that is, shear heads) are seldom used in liquid-containing structures.

14.7.1.2 It shall be permitted to neglect the change in length of columns and slabs due to direct stress, and deflections due to shear.

14.7.2 *Equivalent frame*

14.7.2.1 The structure shall be considered to be made up of equivalent frames on column lines taken longitudinally and transversely through the building.

14.7.2.2 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.

14.7.2.3 Columns or supports shall be assumed to be attached to slab-beam strips by torsional members (refer to 14.7.5) 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.

14.7.2.4 Frames adjacent and parallel to an edge shall be bounded by that edge and the centerline of adjacent panel.

14.7.2.5 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.

14.7.2.6 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.

R14.7.2 *Equivalent frame*

Application of the equivalent frame to a regular structure is illustrated in Fig. R14.7.2. 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

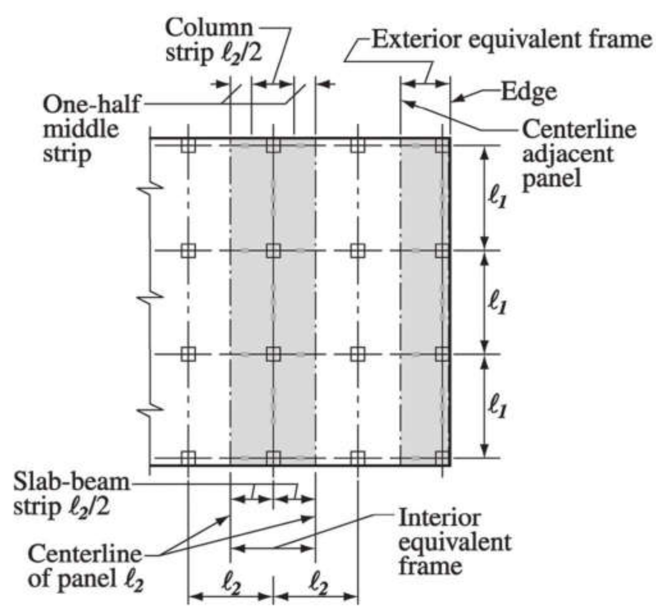

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Fig. R14.7.2—Definitions of equivalent frame.

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14.7.3 *Slab-beams*

14.7.3.1 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.

14.7.3.2 Variation in moment of inertia along axis of slab-beams shall be taken into account.

14.7.3.3 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.

14.7.4 *Columns*

14.7.4.1 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.

14.7.4.2 Variation in moment of inertia along axis of columns shall be taken into account.

14.7.4.3 Moment of inertia of columns from top to bottom of the slab-beam at a joint shall be assumed infinite.

COMMENTARY

elements of the structure that provide moment transfer between the horizontal and vertical members.

R14.7.3 *Slab-beams*

R14.7.3.3 A support is defined as a column, capital, bracket, or wall. A beam is not considered to be a support member for the equivalent frame.

R14.7.4 *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

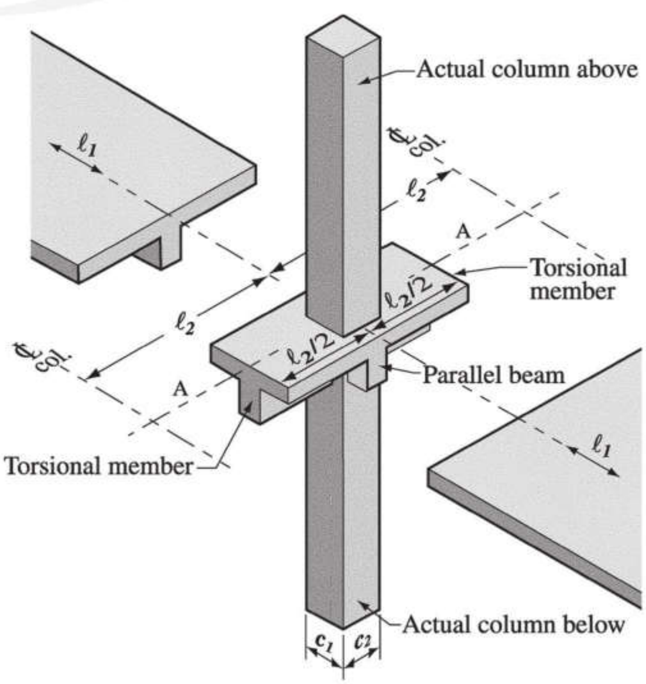

Fig. R14.7.4—Equivalent column (column plus torsional members).

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14.7.5 *Torsional members*

14.7.5.1 Torsional members (refer to 14.7.2.3) 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 14.2.4

14.7.5.2 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.

COMMENTARY

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. R14.7.4.

R14.7.5 *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 14.7.5.1. Up to ACI 318-89 Eq. (14-6) specified the stiffness coefficient K_t of the torsional members. In ACI 318-95, the approximate expression for f was moved to the commentary.

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. R14.7.5.

An approximate expression for the stiffness of the torsional member, based on the results of three-dimensional analyses of various slab configurations (ACI 352.1R; Jirsa et al. 1969; Nichols 1914) is

$$K_t = \sum \frac{9E_{cs}C}{\ell_2\left(1-\frac{c_2}{\ell_2}\right)^3}$$

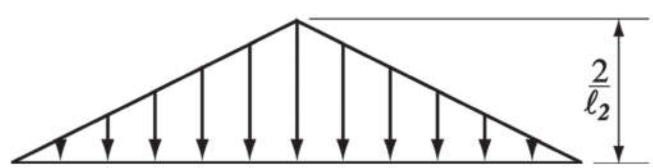

Fig. R14.7.5—Distribution of unit twisting moment along column centerline AA shown in Fig. R14.7.4.

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14.7.6 *Arrangement of live load*

14.7.6.1 When the loading pattern is known, the equivalent frame shall be analyzed for that load.

14.7.6.2 When the unfactored live load is variable but does not exceed three-fourths of the unfactored 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.

14.7.6.3 For loading conditions other than those defined in 14.7.6.2, it shall be permitted to assume that maximum positive factored moment near midspan of a panel occurs with three-fourths of the full factored live load on the panel and on alternate panels; it shall be permitted to assume that maximum negative factored moment in the slab at a support occurs with three-fourths of the full factored live load on adjacent panels only.

14.7.6.4 Factored moments shall be taken not less than those occurring with full factored live load on all panels.

14.7.7 *Factored moments*

14.7.7.1 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 the center of a column.

14.7.7.2 At exterior supports 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.

14.7.7.3 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.

14.7.7.4 Where slab systems within limitations of 14.6.1 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. (14-4).

14.7.7.5 Distribution of moments at critical sections across the slab-beam strip of each frame to column strips, beams, and middle strips as provided in 14.6.4, 14.6.5, and 14.6.6 shall be permitted if the requirement of 14.6.1.6 is satisfied.

COMMENTARY

R14.7.6 *Arrangement of live load*

The use of only three-fourths 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 system after redistribution of moment is not less than that required to carry the full factored dead and live loads on all panels.

R14.7.7 *Factored moments*

R14.7.7.1, R14.7.7.72, and R14.7.7.3 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 R14.6.2.5 illustrates several equivalent rectangular supports for use in establishing faces of supports for design with nonrectangular supports.

R14.7.7.4 Previous ACI 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. (14-4), it is considered that these values are satisfactory for design when applicable limitations are met.

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CHAPTER 15—WALLS

15.1—Scope

15.1.1 Provisions of Chapter 15 shall apply for design of planar walls subjected to axial load, with or without flexure; and circular walls with axial load and hoop tension, with or without flexure.

15.1.2 Cantilever retaining walls are designed according to flexural design provisions of Chapter 10 with minimum horizontal reinforcement according to 15.4.3.

15.1.3 Walls prestressed circumferentially by wrapping with high-strength steel wire or strand shall be designed as described in this chapter and the provisions of Chapter 19.

15.1.4 Walls prestressed horizontally or circumferentially with internal tendons shall be designed according to provisions of Chapter 19.

15.2—General

15.2.1 Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.

15.2.2 Walls subject to axial loads shall be designed in accordance with 15.2, 15.4, and either 15.5, 15.6, or 15.8.

15.2.3 Design for shear shall be in accordance with 11.10.

15.2.4 Unless otherwise demonstrated by an analysis, the horizontal length of wall considered as effective for each concentrated load shall not exceed the smaller of the center-to-center distance between loads, and the bearing width plus four times the wall thickness.

15.2.5 Compression members built integrally with walls shall conform to 10.8.2.

15.2.6 Transfer of force to footing at base of wall shall be in accordance with 16.8.

15.3—Walls prestressed circumferentially by wrapping with high-strength steel wire or strand

15.3.1 *Core walls*

15.3.1.1 Core walls to be circumferentially prestressed by wrapping with high-strength steel wire or strand shall consist of shotcrete, precast or cast-in-place concrete, or combinations thereof.

15.3.1.2 Shotcrete core walls shall be built up of individual layers of shotcrete, 2 in. or less in thickness.

15.3.1.3 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.

COMMENTARY

CHAPTER R15—WALLS

R15.1—Scope

Cantilever retaining walls are designed according to the flexural design provisions of Chapter 10. Walls designed to resist shear forces, such as shear walls, should be designed in accordance with Chapter 15 and 11.9 as applicable.

In ACI 318-77, walls could be designed according to Chapter 14 or 10.15. In ACI 318-83, these two were combined in Chapter 14.

R15.2—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 15.5 unless the wall meets the requirements of 15.6.1.

In either case, walls may be designed using either the strength design method of the code or the alternate design method of Appendix A in accordance with A.6.3.

15.3—Walls prestressed circumferentially by wrapping with high-strength steel wire or strand

R15.3.1 The vast majority of wire- and strand-wrapped tanks use one of these four basic wall types. Details of each type include:

(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. In addition, cast-in-place concrete wall tanks usually have vertical nonprestressed reinforcement near each face for strength and crack control.

(b) Shotcrete with full height, vertically fluted steel diaphragm and prestressed circumferentially with either high-strength wire or strand, wound on the external

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15.3.1.4 Vertical construction joints in walls without a metal diaphragm shall contain waterstops.

15.3.1.5 Core walls shall have a bondable exterior surface prior to application of wire- or strand-wrapped prestressed reinforcement. Dust, efflorescence, oil, and other foreign material shall be removed. Defects in the core wall shall be filled flush with mortar or shotcrete that is bonded to the core wall. Concrete core wall surfaces cast against forms or other smooth surfaces shall be cleaned and roughened by abrasive blasting prior to application of prestressed reinforcement and shotcrete.

15.3.1.6 The core wall, including the vertical construction joints and wall base joint, shall be liquid-tight in accordance with ACI 350.1.

15.3.2 Circumferential wire or strand prestressed reinforcement shall have a minimum clear spacing between of at least 1.5 times the wire or strand diameter, or 1/4 in. for wires and 3/8 in. for strands, whichever is greater.

COMMENTARY

surface of the shotcrete core wall. Shotcrete walls use a steel diaphragm near one face of the wall as a surface to shoot shotcrete against. Usually, 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. The diaphragm is coated first with shotcrete on both sides, after which the composite wall is 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 reinforcement near the interior face. In precast walls with diaphragms, adjacent sections of diaphragm, both within the panels and between panels, are joined and sealed. The joints between the panels are filled with high-strength mortar or shotcrete. The diaphragm is coated first with shotcrete, after which the composite wall is prestressed circumferentially by winding with high-strength steel wire or strand.

(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 reinforcement near the interior face. In cast-in-place concrete walls using a steel diaphragm, adjacent sections of the diaphragm are joined and sealed. 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.

R15.3.1.6 Considerations with respect to leakage through the walls used in the vast majority of wire-and strand-wrapped tanks are:

(a) The provision of a full-height, vertically fluted steel diaphragm having sealed edge joints that form an impervious membrane and which extends 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 helping to provide liquid-tightness.

(c) A dense, well-consolidated concrete, free of honeycombing and cold joints, is essential for providing a durable, liquid-tight concrete wall.

R15.3.2 The circumferential prestressing is covered with shotcrete to provide bond, protection from mechanical damage, and corrosion protection.

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15.3.3 *Shotcrete cover coats*

Wire or strand wrapping shall be covered by a shotcrete cover coat.

15.3.4 *Design of nonprestressed vertical reinforcement*

15.3.4.1 Walls in liquid-containing tanks having a steel diaphragm shall be vertically reinforced with nonprestressed reinforcement. The area of nonprestressed 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, internal loads, backfill, external loads, in addition to the effects of shrinkage, elastic shortening, concrete creep, relaxation of prestressed reinforcement, and temperature and moisture gradients. 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 in the reinforcement under service loads 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.

15.3.4.2 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 in accordance with Chapter 19. 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.

15.3.5 *Restraint cables*

15.3.5.1 *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 designed to provide a compressible area around the restraint cables to accommodate radial movement of the wall.

COMMENTARY

R15.3.5 *Restraint cables*

Restraint 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 restraint cables, including adequately sized separation sleeves and attention to corrosion resistance, are essential to providing long-term seismic protection. The separation sleeves are designed to provide a compressible area around the restraint cables that accommodates radial movement of the wall.

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15.3.5.2 *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.

15.3.6 *Elastomeric bearing pads*

15.3.6.1 *Positioning*
Bearing pads under the wall or between the wall and roof 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.

15.3.6.2 *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 ensure freedom from all obstructions to provide for free movement of the wall base.

15.3.7 *Sponge rubber fillers*

15.3.7.1 *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.

15.3.7.2 *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.

15.4—Minimum reinforcement

15.4.1 In concretes made with ASTM C150 or C595 cements, the minimum vertical and horizontal reinforcement for temperature and shrinkage shall be in accordance with 15.4.2 and 15.4.3.

15.4.2 Minimum vertical reinforcement shall be based on 12.13.2 for nonprestressed walls, 15.3.4 for a circumferentially prestressed wall, and 12.13.3 for a circumferentially and vertically prestressed wall.

15.4.3 Minimum horizontal reinforcement shall be based on 12.13.2 except for walls that are prestressed in accordance with Chapter 19 where 12.13.2 does not apply.

COMMENTARY

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.

15.3.6 *Elastomeric bearing pads*

R15.3.6.1 *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.

R15.4—Minimum reinforcement

The requirements of 15.4 are similar to those in previous ACI 318 codes. These apply to walls designed according to 15.5 or 15.6. 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 15.4.

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15.4.4 Nonprestressed 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 12.7 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 12.7 nor more than one-third of the thickness of wall from the interior surface

15.4.5 Vertical and horizontal reinforcement shall not be spaced farther apart than 12 in.

15.4.6 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.

15.4.7 In addition to the minimum reinforcement, 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.

15.5—Walls designed as compression members

Except as provided in 15.6, walls subject 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, 10.14, 15.2, and 15.4.

15.6—Empirical Design Method

15.6.1 Walls of solid rectangular cross section shall be permitted to be designed by the empirical provisions of 15.6 if the resultant of all factored loads is located within the middle third of the overall thickness of the wall and all limits of 15.2, 15.4, and 15.6 are satisfied.

15.6.2 Design axial strength ϕP_n of a wall satisfying limitations of 15.6.1 shall be computed by Eq. (15-1) unless designed in accordance with 15.5

$$\phi P_n = 0.55\phi f_c' A_g \left[1 - \left(\frac{k\ell_c}{32h}\right)^2\right] \quad (15\text{-}1)$$

where ϕ shall correspond to compression-controlled sections in accordance with 9.3.2.2 and effective length factor k shall be:

For walls braced top and bottom against lateral translation and

COMMENTARY

R15.4.7 Where there are openings in walls subjected to lateral loads, the reinforcement described in 15.4.7 is considered minimum reinforcement. The wall should be designed to replace the strength lost at the opening and transfer the load around the opening.

R15.6—Empirical Design Method

The Empirical Design Method applies only to solid rectangular cross sections. All other shapes should be designed according to 15.5.

Eccentric loads and lateral forces are used to determine the total eccentricity of the factored axial force 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 force P_u should be less than or equal to the design axial strength ϕP_n computed by Eq. (15-1), $P_u \leq \phi P_n$.

With the ACI 318-77 supplement from 1980, Eq. (15-1) was revised to reflect the general range of end conditions encountered in wall designs. The wall strength equation in ACI 318-77 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

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(a) restrained against rotation at one or both ends (top, bottom, or both): 0.8

(b) unrestrained against rotation at both ends: 1.0

For walls not braced against lateral translation: 2.0

COMMENTARY

determined from the original equation were unconservative when compared to test results (Oberlander and Everard 1977) for walls with pinned conditions at both ends, as occurs with some precast and tilt-up applications, or when the top of the wall is not effectively braced against translation, as occurs with free-standing walls or in large structures where significant roof diaphragm deflections occur due to wind and earthquake-induced loads. Equation (15-1) gives the same results as ACI 318-77 for walls braced against translation and with reasonable base restraint against rotation (Kripanarayanan 1977). Values of effective 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. (15-1) results in relatively comparable strengths by either 15.4 or 15.5 for members loaded at the middle third of the thickness with different braced and restrained end conditions. Refer to Fig. R15.6.

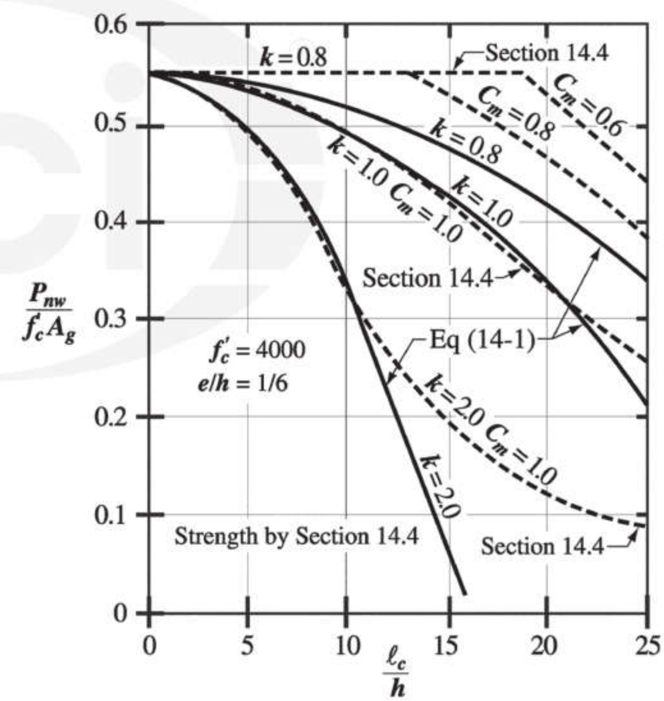

Fig. R15.6—Empirical design of walls, Eq. (15-1), versus 15.5.

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15.6.3 *Minimum thickness of walls designed by empirical design method*

15.6.3.1 Thickness of bearing walls that do not receive their vertical stability from curvature shall not be less than 1/25 the unsupported height or length, whichever is less, nor less than 8 in.

15.7—Minimum wall thickness

15.7.1 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 provide lateral support.

15.7.2 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.

15.7.3 Minimum core wall thickness for walls prestressed circumferentially with either high-strength steel wire or strand, wound on the external surface of the core wall shall be:

(a) 3-1/2 in. for shotcrete walls with a steel diaphragm
(b) 4 in. for precast concrete walls with a steel diaphragm
(c) 8 in. for cast-in-place concrete walls

15.8—Walls as grade beams

15.8.1 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.

15.8.2 Portions of grade beam walls exposed above grade shall also meet requirements of 15.4.

COMMENTARY

R15.6.3 *Minimum thickness of walls designed by empirical design method*

The minimum thickness requirements need not be applied to walls designed according to 15.5.

R15.7.3 Experience in wrapped prestressed tank design and construction has demonstrated that these are practical production limits.

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CHAPTER 16—FOOTINGS

16.1—Scope

16.1.1 Provisions of Chapter 16 shall apply for design of isolated footings and, where applicable, to combined footings and mats.

16.1.2 Additional requirements for design of combined footings and mats are given in 16.10.

16.2—Loads and reactions

16.2.1 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 16.

16.2.2 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.

16.2.3 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

CHAPTER R16—FOOTINGS

R16.1—Scope

While the provisions of Chapter 16 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 (ACI 336.2R; Kramrisch and Rogers 1961).

R16.2—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 accordance 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 10.10.6.5 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 (refer to 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 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 16.2.2 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 A 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

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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 A.2.2.

16.3—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.

16.4—Moment in footings

16.4.1 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.

16.4.2 Maximum factored moment M_u for an isolated footing shall be computed as prescribed in 16.4.1 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

16.4.3 In one-way footings, and two-way square footings, reinforcement shall be distributed uniformly across entire width of footing.

16.4.4 In two-way rectangular footings, reinforcement shall be distributed in accordance with 16.4.4.1 and 16.4.4.2.

16.4.4.1 Reinforcement in long direction shall be distributed uniformly across entire width of footing.

16.4.4.2 For reinforcement in short direction, a portion of the total reinforcement given by Eq. (16-1) 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, $(1 - \gamma_s)A_s$, shall be distributed uniformly outside center band width of footing.

$$\gamma_s = \frac{2}{(\beta + 1)} \tag{16-1}$$

R16.4—Moment in footings

R16.4.4 In previous codes, the reinforcement in the short direction of rectangular footings should be distributed so that an area of steel given by Eq. (16-1) 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.

16.5—Shear in footings

16.5.1 Shear strength of footings shall be in accordance with 11.11.

R16.5—Shear in footings

R16.5.1 and R16.5.2 The shear strength of footings is determined for the more severe condition of 11.11.1.1 or 11.11.1.2. The critical section for shear is measured from

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16.5.2 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 16.4.2(c).

COMMENTARY

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 $\boldsymbol{q_s}$ 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 11.11.1.2. If shear perimeters overlap, the modified critical perimeter $\boldsymbol{b_o}$ should be taken as that portion of the smallest envelope of individual shear perimeter that will resist the critical shear for the group under consideration. One such situation is illustrated in Fig. R16.5.

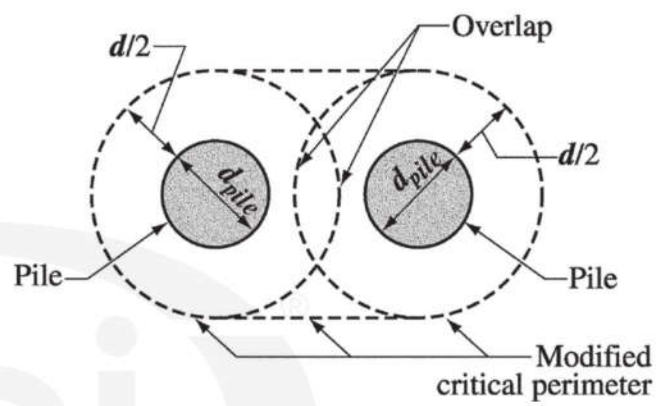

Fig. R16.5—Modified critical perimeter for shear with overlapping critical perimeters.

16.5.3 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 11.11 and 16.5.4. Other pile caps shall satisfy 11.11 or 16.5.4.

R16.5.3 When piles are located inside the critical sections $\boldsymbol{d}$ or $\boldsymbol{d/2}$ 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* (CRSI 2008) offers guidance for this situation.

16.5.4 Computation of shear on any section through a footing supported on piles shall be in accordance with 16.5.4.1, 16.5.4.2, and 16.5.4.3.

16.5.4.1 Entire reaction from any pile whose center is located $\boldsymbol{d_p/2}$ or more outside the section shall be considered as producing shear on that section.

16.5.4.2 Reaction from any pile whose center is located $\boldsymbol{d_p/2}$ or more inside the section shall be considered as producing no shear on that section.

16.5.4.3 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 $\boldsymbol{d_p/2}$ outside the section and zero value at $\boldsymbol{d_p/2}$ inside the section.

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16.6—Development of reinforcement in footings

16.6.1 Development of reinforcement in footings shall be in accordance with Chapter 12.

16.6.2 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.

16.6.3 Critical sections for development of reinforcement shall be assumed at the same locations as defined in 16.4.2 for maximum factored moment, and at all other vertical planes where changes of section or reinforcement occur. Refer also to 12.8.10.6.

16.7—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.

16.8—Transfer of force at base of column, wall, or reinforced pedestal

16.8.1 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.

16.8.1.1 Bearing on concrete at contact surface between supported and supporting member shall not exceed concrete bearing strength for either surface as given by 10.14.

COMMENTARY

R16.8—Transfer of force at base of column, wall, or reinforced pedestal

Section 16.8 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.

The requirements of 16.8.1 apply to both cast-in-place construction and precast construction. Additional requirements for cast-in-place construction are given in 16.8.2. Section 16.8.3 gives additional requirements for precast construction.

R16.8.1.1 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 $\mathbf{0.85\phi} f_c'$ 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 $\mathbf{0.85\phi} f_c'$. The permissible bearing strength on the footing may be increased in accordance with 10.14 and will usually be two times $\mathbf{0.85\phi} f_c'$. 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.

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16.8.1.2 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 16.8.2 or 16.8.3.

16.8.1.3 If calculated moments are transferred to supporting pedestal or footing, then reinforcement, dowels, or mechanical connectors shall be adequate to satisfy 12.9.4.

16.8.1.4 Lateral forces shall be transferred to supporting pedestal or footing in accordance with shear-friction provisions of 11.6, or by other appropriate means.

16.8.2 In cast-in-place construction, reinforcement required to satisfy 16.8.1 shall be provided either by extending longitudinal bars into supporting pedestal or footing, or by dowels.

16.8.2.1 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.

16.8.2.2 For cast-in-place walls, area of reinforcement across interface shall be not less than minimum vertical reinforcement given in 15.4.

16.8.2.3 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 16.8.1. Dowels shall not be larger than No. 11 bar and shall extend into supported member a distance not less than the larger of ℓ_{dc} of No. 14 or No. 18 bars and compression lap-splice length of the dowels, whichever is greater, and into the footing a distance not less than ℓ_{dc} of the dowels.

COMMENTARY

For the alternate design method of Appendix A, permissible bearing stresses are limited to 50 percent of the values in 10.14.

R16.8.1.2 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.

R16.8.1.3 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 $\mathbf{0.85\phi f_c'}$ under factored load conditions and, as a result, all the reinforcement will generally have to be doweled into the footing.

R16.8.1.4 The shear-friction method given in 11.6 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 16.8.2.1, 16.8.3.1, and the shear-friction requirements of 11.6. In precast construction, resistance to lateral forces may be provided by shear-friction, shear keys, or mechanical devices.

R16.8.2.1 and R16.8.2.2 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.

R16.8.2.3 Lap splices of No. 14 and No. 18 longitudinal bars in compression only to dowels from a footing are specifically permitted in 16.8.2.3. The dowel bars should be No. 11 or smaller in size. The dowel lap splice length should meet the larger of the two criteria: 1) be able to transfer the stress in the No. 14 and No. 18 bars; and 2) fully develop the stress in the dowels as a splice.

This provision is an exception to 12.9.1.2.1, 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

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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 12.9.3.2.

16.8.2.4 If a pinned or rocker connection is provided in cast-in-place construction, connection shall conform to 16.8.1 and 16.8.3.

16.8.3 In precast construction, anchor bolts or suitable mechanical connectors shall be permitted for satisfying 16.8.1. Anchor bolts shall be designed in accordance with Appendix E.

16.8.3.1 Connection between precast columns or pedestals and supporting members shall meet the requirements of 17.5.1.3(a).

16.8.3.2 Connection between precast walls and supporting members shall meet the requirements of 17.5.1.3(b) and (c).

R16.8.3.1 and R16.8.3.2 For cast-in-place columns, 16.8.2.1 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, $A_s f_y = \mathbf{200}A_g$ (refer to 17.5.1.3(a)). The minimum tensile strength required for precast wall-to-footing connection (refer to 17.5.1.3(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 17.5.1.3 have been arbitrarily chosen, it is not necessary to include a strength reduction factor ϕ for these calculations.

16.8.3.3 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 E.

16.9—Sloped or stepped footings

16.9.1 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. (Refer also to 12.8.10.6.)

16.9.2 Sloped or stepped footings designed as a unit shall be constructed to assure action as a unit.

16.10—Combined footings and mats

16.10.1 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.

R16.10—Combined footings and mats

R16.10.1 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 (refer to 16.1). Similarly, as prescribed in 16.2.2 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.

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16.10.2 The Direct Design Method of Chapter 14 shall not be used for design of combined footings and mats.

16.10.3 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.

16.10.4 Minimum reinforcing steel in nonprestressed mat foundations shall meet the requirements of 12.13.1 in each principal direction. Maximum spacing shall not exceed 12 in.

COMMENTARY

Detailed recommendations for design of combined footings and mats are reported in 336.2R. Refer also to Kramrisch and Rogers (1961).

R16.10.2 Minimum reinforcing steel may be distributed near the top or bottom of the section, or may be allocated between the two faces of the section as deemed appropriate for specific conditions, such that the total area of continuous reinforcing steel satisfies 12.13.1.

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CHAPTER 17—PRECAST CONCRETE

17.1—Scope

17.1.1 All provisions of this Code, not specifically excluded and not in conflict with the provisions of Chapter 17, shall apply to structures incorporating precast concrete structural members.

17.2—General

17.2.1 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.

17.2.2 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.

17.2.3 Tolerances for both precast members and interfacing members shall meet the requirements of ACI ITG-7

COMMENTARY

CHAPTER R17—PRECAST CONCRETE

R17.1—Scope

R17.1.1 Refer to 2.2 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 to precast concrete, they have not been repeated. Similarly, items related to composite concrete in Chapter 18 and to prestressed concrete in Chapter 19 that apply to precast concrete are not restated.

More detailed recommendations concerning precast concrete are given in ACI SP-48 (*Industrialization in Concrete Building Construction* 1975), Waddell (1974), PCI (1988, 1992), ATC (1981), PCI Committee on Building Code and PCI Technical Activities Committee (1986), and ACI 550R. Tilt-up concrete construction is a form of precast concrete. It is recommended that ACI 551R 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's *Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products, Fourth Edition, MNL-116-99* (PCI 1999).

R17.2—General

R17.2.1 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. Care should be given to the methods of storing, transporting, and erecting precast members so that performance at service loads and strength under factored loads meet Code requirements.

R17.2.2 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. Where precast members are part of a liquid-containing structure, connections should be designed and detailed to maintain liquid tightness and should be protected from the corrosive effects of the liquid contents. In building structures that house environmental processes, the atmosphere may be wet, humid, and corrosive and the connections should be detailed and protected from the corrosive environment.

R17.2.3 Design of precast members and connections is particularly sensitive to tolerances on the dimensions of individual members and on their location in the structure.

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and Chapter 15. Design of precast members and connections shall include the effects of these tolerances.

17.2.4 In addition to the requirements for contract documents in 1.2, the following 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.

17.2.5 Embedded items, such as inserts or lifting devices, shall be permanently protected against corrosion.

17.3—Distribution of forces among members

17.3.1 Distribution of forces that are perpendicular to the plane of members shall be established by analysis or by test.

17.3.2 Where the system behavior requires in-plane forces to be transferred between the members of a precast floor or wall system, 17.3.2.1 and 17.3.2.2 shall apply.

17.3.2.1 In-plane force paths shall be continuous through both connections and members.

17.3.2.2 Where tension forces occur, a continuous path of steel or steel reinforcement shall be provided.

COMMENTARY

To prevent misunderstanding, the tolerances used in design should be specified in the contract documents. Instead of specifying individual tolerances, the tolerance standard assumed in design may be specified. It is important to specify any deviations from accepted standards.

The tolerances required by 12.5 are considered a minimum acceptable standard for reinforcement in precast concrete. The tolerances required by ACI ITG-7 are considered a minimum acceptable standard for product and erection tolerances in precast concrete.

R17.2.4 The additional requirements may be included in either contract documents or shop drawings, depending on the assignment of responsibility for design.

R17.2.5 Embedded items, including inserts and lifting devices, are commonly used in precast construction, and should not be left exposed to the liquid contents unless they are made from a corrosion-resistant metal suitable for the intended exposure. They are typically recessed a minimum of 1/2 in. from the face of the precast element (due to commercially available hardware), the surface is prepared, and a non-shrink, cementitious grout is used to fill the void around the embedded item. Where reduced cover exists compared to that required for reinforcement, additional corrosion-protection measures such as corrosion inhibitors, barrier coatings, or mounding of the grout should be used.

R17.3—Distribution of forces among members

R17.3.1 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 LaGue (1971), Johnson and Ghadiali (1972), Pfeifer and Nelson (1983), Stanton (1987, 1992), PCI (1985b), and Aswad and Jacques (1992). Large openings can cause significant changes in distribution of forces.

R17.3.2 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 includes bolts, weld plates, headed studs, or

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CODE | COMMENTARY

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.

17.4—Member design

17.4.1 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 12.13 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.

R17.4—Member design

R17.4.1 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 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.

17.4.2 For precast, nonprestressed walls, the reinforcement shall be designed in accordance with the provisions of Chapters 10 or 15, except that the area of horizontal and vertical reinforcement each shall not be less than $0.001A_g$, where A_g is the gross cross-sectional area of the wall panel, instead of the minimum reinforcement areas for walls stated in 15.3. Spacing of reinforcement shall not exceed 12 in.

R17.4.2 This minimum area of wall reinforcement, instead of the minimum values in 15.3, has been used for many years and is recommended by the PCI (1992), CSA A23.3, and CSA A23.4. 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 develop less shrinkage stress than comparable cast-in-place walls.

17.5—Structural integrity

17.5.1 Except where the provisions of 17.5.2 govern, the minimum provisions of 17.5.1.1 through 17.5.1.4 for structural integrity shall apply to all precast concrete structures.

R17.5—Structural integrity

R17.5.1 The provisions of 12.14.3 apply to all precast concrete structures. Sections 17.5.1 and 17.5.2 give minimum requirements to satisfy 12.14.3. 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.

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17.5.1.1 Longitudinal and transverse ties required by 12.14.3 shall connect members to a lateral-load-resisting system.

17.5.1.2 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.

17.5.1.3 Vertical tension tie requirements of 12.14.3 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 $\mathbf{200}A_g$, 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 17.5.1.3(b) shall be permitted to be anchored into an appropriately reinforced concrete floor slab-on-ground

17.5.1.4 Connection details that rely solely on friction caused by gravity loads shall not be permitted.

17.5.2 For precast concrete bearing wall structures three or more stories in height, the minimum provisions of 17.5.2.1 through 17.5.2.5 shall apply.

COMMENTARY

R17.5.1.1 Individual members may be connected into a 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 is connected to the diaphragm.

R17.5.1.2 Diaphragms are typically provided as part of the lateral-load-resisting system. The ties prescribed in 17.5.1.2 are the minimum required to attach members to the floor or roof diaphragms.

R17.5.1.3 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 17.5.1.3 are not additive to these design requirements. Common practice for buildings 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.

R17.5.1.4 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 ability. This situation 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.

R17.5.2 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 (PCI 1985b). 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 17.5.1 apply to bearing wall structures less than three stories in height.

Minimum ties in structures three or more stories in height, in accordance with 17.5.2.1, 17.5.2.2, 17.5.2.3, 17.5.2.4, and 17.5.2.5, are required for structural integrity (Fig. R17.5.2).

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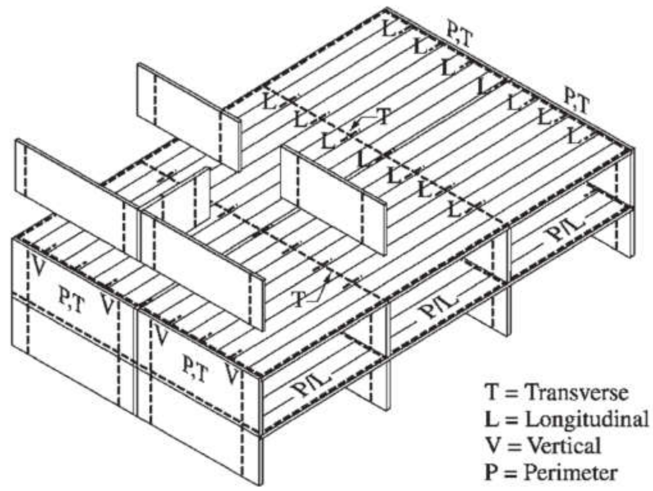

Fig. R17.5.2—Typical arrangement of tensile ties in large panel structures.

These provisions are based on PCI's recommendations for design of precast concrete bearing wall buildings (Stanton 1992). Tie capacity is based on yield strength.

17.5.2.1 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.

R17.5.2.1 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 the yield strength (Salmons and McCrate 1977). It is not uncommon to have ties positioned in the walls reasonably close to the plane of the floor or roof system.

17.5.2.2 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.

17.5.2.3 Transverse ties perpendicular to floor or roof slab spans shall be spaced not greater than the bearing wall spacing.

R17.5.2.3 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.

17.5.2.4 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.

R17.5.2.4 The perimeter tie requirements need not be additive with the longitudinal and transverse tie requirements.

17.5.2.5 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.

17.6—Connection and bearing design

17.6.1 Forces shall be permitted to be transferred between members by grouted joints, shear keys, mechanical connectors, bearing pads, reinforcing steel, strand connections, reinforced topping, or a combination of these means.

R17.6—Connection and bearing design

R17.6.1 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.

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17.6.1.1 The adequacy of connections to transfer forces between members shall be determined by analysis or by test. Where shear is the primary result of imposed loading, it shall be permitted to use the provisions of 11.6 as applicable.

17.6.1.2 When designing a connection using materials with different structural properties, their relative stiffnesses, strengths, and ductilities shall be considered.

17.6.2 Bearing for precast floor and roof members on simple supports shall satisfy 17.6.2.1 and 17.6.2.2.

17.6.2.1 The allowable bearing stress at the contact surface between supported and supporting members and between any intermediate bearing elements shall not exceed the

COMMENTARY

R17.6.1.1 Various components in a connection (such as bolts, welds, plates, and inserts) have different properties that can affect the overall behavior of the connection.

R17.6.1.2 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 PCI Committee on Precast Concrete Bearing Wall Buildings (1976).

R17.6.1.3 This section differentiates between bearing length and length of the end of a precast member over the support (Fig. R17.6.1.3). 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.

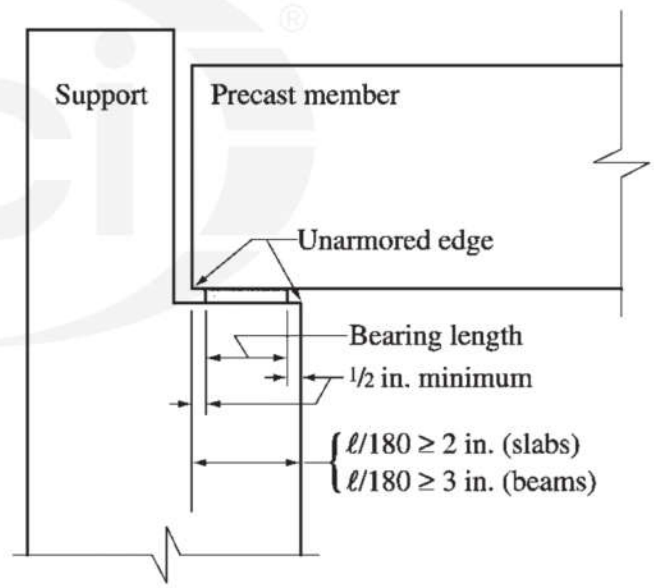

Fig. R17.6.1.3—Bearing length on support.

R17.6.1.4 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 structural plain concrete where reinforcement has been discontinued.

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bearing strength for either surface or the bearing element, or both. Concrete bearing strength shall be as given in 10.14.

17.6.2.2 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 ℓ_n/**180**, 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

17.6.2.3 The requirements of 12.8.11.1 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 12.5.2.3 and 17.2.3.

17.7—Items embedded after concrete placement

17.7.1 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 that 17.7.1.1, 17.7.1.2, and 17.7.1.3 are met.

17.7.1.1 Embedded items are not required to be hooked or tied to reinforcement within the concrete.

17.7.1.2 Embedded items are maintained in the correct position while the concrete remains plastic.

17.7.1.3 The concrete is properly consolidated around the embedded item.

17.8—Marking and identification

17.8.1 Each precast member shall be marked to indicate its location and orientation in the structure and date of manufacture.

17.8.2 Identification marks shall correspond to placing drawings.

17.9—Handling

17.9.1 Member design shall consider forces and distortions during curing, stripping, storage, transportation, and erection so that precast members are not overstressed or otherwise damaged.

COMMENTARY

R17.7—Items embedded after concrete placement

R17.7.1 Section 17.7.1 is an exception to the provisions of 12.5.1. 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. 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 will be hooked or tied to embedded reinforcement.

R17.9—Handling

R17.9.1 The Code requires acceptable performance at service loads and adequate strength under factored loads. However, handling loads 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 is

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given in PCI reports (PCI Committee on Quality Control and Performance Criteria 1983, 1985) on fabrication and shipment cracks.

17.9.2 During erection, precast members and structures shall be adequately supported and braced to ensure proper alignment and structural integrity until permanent connections are completed.

R17.9.2 All temporary erection connections, bracing, shoring as well as the sequencing of removal of these items should be shown on the erection drawings.

17.10—Strength evaluation of precast construction

17.10.1 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 17.10.1.1 and 17.10.1.2.

17.10.1.1 Test loads shall be applied only when calculations indicate the isolated precast element will not be critical in compression or buckling.

17.10.1.2 The test load shall be that load that, 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 22.7.

17.10.2 The provisions of 22.7 shall be the basis for acceptance or rejection of the precast element.

R17.10—Strength evaluation of precast construction

The strength evaluation procedures of Chapter 22 are applicable to precast members.

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CHAPTER 18—COMPOSITE CONCRETE FLEXURAL MEMBERS

18.1—Scope

18.1.1 Provisions of Chapter 18 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.

18.1.2 All provisions of the Code shall apply to composite concrete flexural members, except as specifically modified in Chapter 18.

18.2—General

18.2.1 The use of an entire composite member or portions thereof for resisting shear and moment shall be permitted.

18.2.2 Individual elements shall be investigated for all critical stages of loading.

18.2.3 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.

18.2.4 In strength computations of composite members, no distinction shall be made between shored and unshored members.

18.2.5 All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.

18.2.6 Reinforcement shall be provided in accordance with 12.13 and 12.14, and as required to minimize cracking and prevent separation of individual elements of composite members.

18.2.7 Composite members shall meet requirements for control of deflections in accordance with 9.5.5.

COMMENTARY

CHAPTER R18—COMPOSITE CONCRETE FLEXURAL MEMBERS

R18.1—Scope

R18.1.1 The scope of Chapter 18 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. Design provisions for such composite members are covered in AISC's "Specification for Structural Steel Buildings: Allowable Stress Design and Plastic Design."

R18.2—General

R18.2.4 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.

R18.2.6 The evaluation and significance of cracking is dependent on such factors as placement environment, aesthetics, required durability and serviceability of the structure. The calculated stress in the reinforcement and spacing of the reinforcement should satisfy the requirements of 9.2.6 and 10.6.4.

R18.2.7 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 does not operate until slippage occurs.

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18.3—Shoring

When used, shoring shall not be removed until supported elements have developed design properties required to support all loads and limit deflections and maintain serviceability requirements at time of shoring removal.

COMMENTARY

R18.3—Shoring

The provisions of 9.5.5 cover the requirements pertaining to deflections of shored and unshored members.

18.4—Vertical shear strength

18.4.1 Where 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.

18.4.2 Shear reinforcement shall be fully anchored into interconnected elements in accordance with 12.8.13.

18.4.3 Extended and anchored shear reinforcement shall be permitted to be included as ties for horizontal shear.

18.5—Horizontal shear strength

18.5.1 In a composite member, full transfer of horizontal shear forces shall be ensured at contact surfaces of interconnected elements.

R18.5—Horizontal shear strength

R18.5.1 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.

18.5.2 For the provisions of 18.5, d shall be taken as the distance from extreme compression fiber for entire composite section to centroid of prestressed and nonprestressed longitudinal tension reinforcement, if any, but need not be taken less than $0.80h$ for prestressed concrete members.

R18.5.2 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 A is used for design of composite members, V_u is the shear due to service loads, and 55 percent of the values given in 18.5.2 are applicable. Refer to A.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 A.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. (18-1). The handling load horizontal shear should be compared with a nominal horizontal shear strength value of $0.55V_{nh}$ (as provided in Appendix A 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.

18.5.3 Unless calculated in accordance with 18.5.4, design of cross sections subject to horizontal shear shall be based on

$$V_u \leq \phi V_{nh} \tag{18-1}$$

where V_{nh} is nominal horizontal shear strength in accordance with 18.5.3.1 through 18.5.3.4.

R18.5.3 The nominal horizontal shear strengths V_{nh} apply when the design is based on the load factors and ϕ-factors of Chapter 9.

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18.5.3.1 Where contact surfaces are clean, free of laitance, and intentionally roughened, V_{nh} shall not be taken greater than $80b_v d$.

18.5.3.2 Where minimum ties are provided in accordance with 18.6, and contact surfaces are clean and free of laitance, but not intentionally roughened, V_{nh} shall not be taken greater than $80b_v d$.

18.5.3.3 Where ties are provided in accordance with 18.6, and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in., V_{nh} shall be taken equal to $(260 + 0.6\rho_v f_y)\lambda b_v d$, but not greater than $500b_v d$. Values for λ in 11.6.4.3 shall apply and ρ_v is $A_v/(b_v s)$.

18.5.3.4 Where V_u at section considered exceeds $\phi(500b_v d)$, design for horizontal shear shall be in accordance with 11.6.4.

18.5.4 As an alternative to 18.5.3, 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 V_u shall not exceed horizontal shear strength ϕV_{nh} as given in 18.5.3.1 through 18.5.3.4, where area of contact surface A_c shall be substituted for $b_v d$.

18.5.4.1 Where ties provided to resist horizontal shear are designed to satisfy 18.5.4, the tie area to tie spacing ratio along the member shall approximately reflect the distribution of shear forces in the member.

18.5.5 Where 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 18.6.

18.6—Ties for horizontal shear

18.6.1 Where ties are provided to transfer horizontal shear, tie area shall not be less than that required by 11.4.6.3 and tie spacing shall not exceed four times the least dimension of supported element, nor exceed 24 in.

COMMENTARY

R18.5.3.3 The permitted horizontal shear strengths and the requirement of 1/4 in. amplitude for intentional roughness are based on tests discussed in Kaar et al. (1960), Saemann and Washa (1964), and Hanson (1960).

R18.5.4.1 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.

R18.5.5 Proper anchorage of ties extending across interfaces is required to maintain contact of the interfaces.

R18.6—Ties for horizontal shear

The minimum areas and maximum spacings are based on test data given in Kaar et al. (1960), Saemann and Washa (1964), Hanson (1960), Grossfield and Birnstiel (1962), and Mast (1968).

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18.6.2 Ties for horizontal shear shall consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire reinforcement.

18.6.3 All ties shall be fully anchored into interconnected elements in accordance with 12.8.13.

COMMENTARY

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CHAPTER 19—PRESTRESSED CONCRETE

19.1—Scope

19.1.1 Provisions of Chapter 19 shall apply to members prestressed with wire, strands, or bars conforming to provisions for prestressing steel in 3.5.5.

19.1.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of Chapter 19, shall apply to prestressed concrete.

19.1.3 The following provisions of this code shall not apply to prestressed concrete, except as specifically noted: Sections 12.6.5, 8.4, 8.12.2, 8.12.3, 8.12.4, 8.13, 10.3.2, 10.3.3, 10.5, 10.6, 10.9.1, and 10.9.2; Chapter 14; and Sections 15.4, 15.6, and 15.7.

COMMENTARY

CHAPTER 19—PRESTRESSED CONCRETE

R19.1—Scope

R19.1.1 The provisions of Chapter 19 are for internal tendon stressing of structural members and slab systems such as two-way flat plate mat foundations; horizontally or vertically prestressed walls of water, wastewater, and other liquid-containing tanks (including cylindrical, rectangular, and egg-shaped); and two-way prestressed roofs over water, wastewater, and other liquid-containing 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 designer 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.

R19.1.3 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 12.6.5—Section 12.6.5 of the Code is excluded from application to prestressed concrete because the requirements for bonded reinforcement and unbonded tendons for cast-in-place members are provided in 19.9 and 19.12, respectively.

Section 8.4—Section 8.4 of the Code is excluded because moment redistribution for prestressed concrete is provided in 19.10.4.

Sections 8.12.2, 8.12.3, and 8.12.4—The empirical provisions of 8.12.2, 8.12.3, and 8.12.4 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 8.12.2, 8.12.3, and 8.12.4, 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 designer. Where possible, the flange widths in 8.12.2, 8.12.3, and 8.12.4 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 8.12.2.

Sections 8.12.1 and 8.12.5 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.

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COMMENTARY

Section 8.13—The empirical limits established for conventionally reinforced concrete joist floors are based on successful past performance of joist construction using standard joist forming systems; refer to R8.13. For prestressed joist construction, experience and judgment should be used. The provisions of 8.13 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 19.8, 19.9, and 19.11.2.

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 14—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 14. Because of these unique properties associated with prestressing, many of the design procedures of Chapter 14 are not appropriate for prestressed concrete structures and are replaced by the provisions of 19.12.

Sections 15.4, 15.6, and 15.7—The requirements for wall design in 15.4, 15.6, and 15.7 are largely empirical, utilizing considerations not intended to apply to prestressed concrete.

19.1.4 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.

R19.1.4 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 19.3.3 and 19.4.2 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 forces only because the shear is not resisted by the prestressed reinforcement.

19.2—General

19.2.1 Prestressed members shall meet the strength requirements specified in this Code.

19.2.2 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.

R19.2—General

R19.2.1 and R19.2.2 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) 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.

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COMMENTARY

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 19.3.2 provides assumptions that may be used for investigation at service loads and after transfer of the prestressing force.

19.2.3 Stress concentrations due to prestressing shall be considered in design.

19.2.4 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.

19.2.5 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.

R19.2.5 Section 19.2.5 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, it is not possible to buckle the member under the prestressing force being introduced.

19.2.6 In computing section properties before bonding of prestressing steel, effect of loss of area due to open ducts shall be considered.

R19.2.6 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.

19.2.7 When circumferential prestressing tendons are used in cylindrical tanks, they shall be bonded tendons.

R19.2.7 When unbonded tendons are used in other wall applications, consideration should be given to increasing the reinforcement requirement of 19.9. This additional reinforcement is recommended to control cracking at service levels.

19.3—Design assumptions

19.3.1 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-containing structures shall be based on elastic analysis methods for external loads and prestressing.

R19.3—Design assumptions

R19.3.1 Strength design methods are of doubtful relevance to the design of prestressed concrete liquid-containing structures. Serviceability of the structure under the design loads is of paramount importance. A working stress approach is therefore required. Provided that the allowable stresses spec-

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ified 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 prescribed in 19.4.2 and 19.5.1. 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.

19.3.2 For investigation of stresses at transfer of prestress, at service loads, and at cracking loads, elastic theory shall be used with the assumptions of 19.3.2.1 and 19.3.2.2.

19.3.2.1 Strains vary linearly with depth through entire load range.

19.3.2.2 At cracked sections, concrete resists no tension.

19.3.3 Prestressed flexural members shall be classified as Class U or Class T based on f_t, the computed extreme fiber stress in tension in the precompression tensile zone calculated at service loads, 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 systems shall be designed as Class U with $f_t \leq 6.0\sqrt{f'_c}$.

R19.3.3 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 R19.3.3. For comparison, Table R19.3.3 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 318-11 are not used in environmental engineering 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.

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Table R19.3.3—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 19.3.4	Gross Section 19.3.4
Allowable stress at transfer	19.4.1	19.4.1
Allowable compressive stress based on uncracked section properties	19.4.2	19.4.2
Tensile stress at service loads 19.3.3	$\leq 7.5\sqrt{f_c'}$	$\leq 7.5\sqrt{f_c'} < f_t < 12\sqrt{f_c'}$
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

19.3.4 For prestressed flexural members, stresses at service loads shall be permitted to be calculated using the uncracked section.

19.3.5 Deflections of prestressed flexural members shall be calculated in accordance with 9.5.4.

19.4—Serviceability requirements—flexural members

R19.4—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.

19.4.1 Stresses in concrete immediately after prestress transfer (before time-dependent prestress losses):

(a) Extreme fiber stress in compression except as permitted in (b) and (d) shall not exceed $\mathbf{0.60}f_{ci}'$

R19.4.1 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. The compressive transfer stress at ends of simply supported members was raised from $\mathbf{0.60}f_{ci}'$ to $\mathbf{0.70}f_{ci}'$ in the ACI 318-08 code to reflect research and precast, prestressed concrete industry practice (Castro et al. 2004; Dolan and Krohn 2007; Hale and Russell 2006).

(b) Extreme fiber stress in compression due to initial prestress in cylindrical prestressed walls: $\mathbf{0.55}f_{ci}'$

(c) Extreme fiber stress in compression at ends of simply supported members shall not exceed $\mathbf{0.70}f_{ci}'$

R19.4.1(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 $\mathbf{0.6}f_y$, 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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(d) Where computed concrete tensile stress f_t exceeds $\mathbf{6}\sqrt{f_{ci}'}$ at ends of simply supported members, or $\mathbf{3}\sqrt{f_{ci}'}$ at other locations, additional bonded reinforcement shall be provided in the tensile zone to resist the total tensile force in concrete computed with the assumption of an uncracked section

(e) Concrete compressive stress in cylindrical prestressed walls shall not exceed $\mathbf{0.55}f_{ci}'$

19.4.2 Net tension under service loads shall not be permitted in unreinforced sections in prestressed environmental engineering concrete structures. Nonprestressed reinforcement at the stress levels permitted in Appendix A shall be provided to resist 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: $\mathbf{0.45}f_c'$

(b) Extreme fiber stress in compression due to prestress plus total load: $\mathbf{0.60}f_c'$

(c) Extreme fiber stress in tension in precompressed tensile zone and in fillet regions of domes containing prestressed dome rings: $\mathbf{6}\sqrt{f_c'}$

COMMENTARY

R19.4.2(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 ACI 318-95 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 19.4.2(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 19.4.2(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.

R19.4.2(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 12.7.3.1. 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 environ-

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(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 12.7.3.2: $12\sqrt{f_c'}$

19.4.3 Permissible stresses in concrete of 19.4.1 and 19.4.2 shall be permitted to be exceeded if shown by test or analysis that performance will not be impaired.

COMMENTARY

ments are encountered, greater cover than that required by 12.7.3.1 should be used, in accordance with 12.7.3.2, and tension stresses reduced to eliminate possible cracking at service loads. The licensed design professional needs to use judgment to determine the amount of increased cover and whether reduced tension stresses are required.

R19.4.2(c) and (d) The permissible concrete tensile stress depends on whether 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.

The fillet region of a dome is the area of transition between the prestressed dome ring and the spherical shell. Bending in the radial direction of the fillet results from unbalanced inward load due to prestressing the dome ring for a portion of the live load combined with the effects of temperature, and internal pressure (when present). Although the bottom of the fillet is normally in compression, service load stresses should be checked on both top and bottom faces, and reinforcement should be provided on both faces.

Because of the bonded reinforcement requirements of 19.9, it is considered that the behavior of segmental members generally will be comparable to that of similarly constructed monolithic concrete members. Therefore, the permissible tensile stress limits of 19.4.2(c) and 19.4.2(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.

R19.4.2(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 licensed design professional is required to increase the concrete protection on the reinforcement, as stipulated in 12.7.3.2, 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 Joint ACI-ASCE Committee 423 (1974), 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 19.4.3.

PCI (1992) provides information on the use of bilinear moment-deflection relationships.

R19.4.3 This section provides a mechanism whereby development of new products, materials, and techniques in prestressed concrete construction need not be inhibited by

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Code limits on stress. Approvals for the design should be in accordance with 1.4 of the Code.

19.5—Permissible stresses in prestressing steel

19.5.1 Tensile stress in prestressing steel shall not exceed the following:

(a) Due to prestressing steel jacking force: $\mathbf{0.94}f_{py}$ 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

(b) Post-tensioning tendons, at anchorages and couplers, immediately after tendon anchorage: $\mathbf{0.70}f_{pu}$

R19.5—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 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.

R19.5.1 Because of the high yield strength of low-relaxation wire and strand meeting the requirements of ASTM A421 and A416, it is appropriate to specify permissible stresses in terms of specified ASTM yield strength along with the specified minimum ASTM tensile strength.

Because of the higher allowable initial prestressing steel stresses permitted since the ACI 318-83 code, final stresses can be greater. Structures subject to corrosive conditions or repeated loadings should be of concern when setting a limit on final stress.

19.6—Loss of prestress

19.6.1 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
(g) Subgrade friction

R19.6—Loss of prestress

R19.6.1 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, refer to Joint ACI-ASCE Committee 423 (1958), ACI Committee 435 (1963), PCI Committee on Prestress Losses (1975), and Zia et al. (1979). Lump-sum values of prestress losses for both pretensioned and post-tensioned members that were indicated in pre-1983 editions of the ACI 318 commentary are considered obsolete. Reasonably accurate estimates of prestress losses can be easily calculated in accordance with the recommendations in Zia et al. (1979) 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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19.6.1.1 For cylindrical externally prestressed concrete tanks with a continuous helix of wire or strand, the long-term loss of prestress shall be taken as not less than 25,000 psi.

19.6.2 *Friction loss in post-tensioning tendons*

19.6.2.1 The required effective prestress shall be indicated in the contract documents.

19.6.2.2 Computed friction loss shall be based on experimentally determined wobble and curvature friction coefficients.

COMMENTARY

R19.6.1.1 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 extended 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 Joint ACI-ASCE Committee 423 (1958), Zia et al. (1979), Ghali and Favre (1986), Heger et al. (1982), Hoffman et al. (1983), Magura et al. (1964), PCI Committee on Prestress Losses (1975), Priestley (1976), and ACI 209R for guidance in calculating prestress losses.

R19.6.2 *Friction loss in post-tensioning tendons*

The coefficients tabulated in Table R19.6.2 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 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 estimate 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.

Table R19.6.2—Friction coefficients for post-tensioned tendons for use in Eq. (19-1) or (19-2)

		Wobble coefficient K	Curvature coefficient μ
	Wire tendons	0.0010 to 0.0015	0.15 to 0.25
	High-strength bars	0.0001 to 0.0006	0.08 to 0.30
	Seven-wire strand	0.0005 to 0.0020	0.15 to 0.25
Unbonded tendons Mastic coated Pregreased	Wire tendons	0.0010 to 0.0020	0.05 to 0.15
	Seven-wire strand	0.0010 to 0.0020	0.05 to 0.15
	Wire tendons	0.0003 to 0.0020	0.05 to 0.15
	Seven-wire strand	0.0003 to 0.0020	0.05 to 0.15

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19.6.2.3 The prestress force and friction losses shall be verified during tendon stressing operations as specified in 19.20.

19.6.3 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.

19.7—Flexural strength

19.7.1 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.

19.7.2 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

$$f_{ps} = f_{pu}\left\{1-\frac{\gamma_p}{\beta_1}\left[\rho_p\frac{f_{pu}}{f_c'}+\frac{d}{d_p}(\omega-\omega')\right]\right\} \quad (19\text{-}1)$$

where ω is $\rho f_y/f_c'$; ω' is $\rho' f_y/f_c'$; and γ_p is 0.55 for f_{py}/f_{pu} not less than 0.80, 0.40 for f_{py}/f_{pu} not less than 0.85, and 0.28 for f_{py}/f_{pu} not less than 0.90.

If any compression reinforcement is taken into account when calculating f_{ps} by Eq. (19-1), 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

$$f_{ps} = f_{se}+10{,}000+\frac{f_c'}{100\rho_p} \quad (19\text{-}2)$$

but f_{ps} in Eq. (19-2) shall not be taken greater than the lesser of f_{py}, and $(f_{se} + 60{,}000)$.

COMMENTARY

R19.6.2.3 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 licensed design professional in conformance with the permissible stresses of 19.4 and 19.5.

R19.7—Flexural strength

R19.7.1 Design moment strength of prestressed flexural members may be computed using strength equations similar to those for conventionally reinforced concrete members. Textbooks and ACI 318-83 commentary (ACI 318R-83) 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.

R19.7.2 Equation (19-1) 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. (19-1) is appropriate when all 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.

The γ_p term in Eq. (19-1) reflects the influence of different types of prestressing reinforcement on the value of f_{ps}. For high-strength prestressing bars confirming to ASTM A722 (Type I), f_{py}/f_{pu} is equal to or greater than 0.85; for high-strength prestressing bars confirming to ASTM A722 (Type II), f_{py}/f_{pu} is equal to or greater than 0.80; for stress-relieved stand (strand) and wire conforming to ASTM A416 and A421, f_{py}/f_{pu} is equal to or greater than 0.85; and for low-relaxation strand and wire confirming to ASTM A416 and A421, f_{py}/f_{pu} is equal to or greater than 0.90.

By inclusion of the ω' term, Eq. (19-1) 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. (19-1) is small, the neutral axis depth is small; hence, the compression reinforcement does not develop its yield strength, and Eq. (19-1) becomes unconservative. This is the reason why the term $[\rho_p f_{pu}/f_c' + (d/d_p)(\omega - \omega')]$ in Eq. (19-1) 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. (19-1)—that

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(c) For members with unbonded tendons and with a span-to-depth ratio greater than 35

$$f_{ps} = f_{se} + 10{,}000 + \frac{f_c'}{300\rho_p} \quad (19\text{-}3)$$

but f_{ps} in Eq. (19-3) shall not be taken greater than the lesser of f_{py}, and $(f_{se} + 30{,}000)$.

19.7.3 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.

19.8—Limits for reinforcement of flexural members

19.8.1 Ratio of prestressed and nonprestressed reinforcement used for computation of moment strength of a member, except as provided in 19.8.2, 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 $0.36\beta_1$.

19.8.2 When a reinforcement ratio in excess of that specified in 19.8.1 is provided, design moment strength shall not exceed the moment strength based on the compression portion of the moment couple.

19.8.3 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.

COMMENTARY

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. (19-1). It is for this reason that the applicability of Eq. (19-1) 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. (19-1) 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 (19-3) reflects results of tests on members with unbonded tendons and span-to-depth ratios greater than 35 (one-way slabs, flat plates, and flat slabs) (Mojtahedi and Gamble 1978). These tests also indicate that Eq. (19-2), formerly used for all span-to-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. (19-2) 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.

R19.8—Limits for reinforcement of flexural members

R19.8.1 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 19.8.1.

R19.8.2 Design moment strength of over-reinforced members may be computed using strength equations similar to those for conventionally reinforced concrete members. Textbooks and ACI 318R-83 provide strength equations for rectangular and flanged sections.

R19.8.3 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

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flexural strength should be reached shortly after cracking, the warning deflection would not occur.

19.9—Minimum bonded reinforcement

19.9.1 A minimum area of bonded reinforcement shall be provided in all flexural members with unbonded tendons as required by 19.9.2 and 19.9.3.

R19.9—Minimum bonded reinforcement

R19.9.1 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 19.9, helps to ensure adequate performance.

Research has shown that unbonded post-tensioned members do not inherently provide large capacity for energy dissipation 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 13.1.1. The minimum bonded reinforcement areas required by Eq. (19-4) and (19-6) are absolute minimum areas independent of grade of steel or design yield strength.

19.9.2 Except as provided in 19.9.3, minimum area of bonded reinforcement shall be computed by

$$A_s = 0.004A_{ct} \tag{19-4}$$

19.9.2.1 Bonded reinforcement required by Eq. (19-4) shall be uniformly distributed over precompressed tensile zone as close as practicable to extreme tension fiber.

19.9.2.2 Bonded reinforcement shall be required regardless of service load stress conditions.

R19.9.2 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 (Mattock et al. 1971). Based on this research, it is advisable to apply the provisions of 19.9.2 also to one-way slab systems.

19.9.3 For two-way flat plates, defined as solid slabs of uniform thickness, minimum area and distribution of bonded reinforcement shall be as required in 19.9.3.1, 19.9.3.2, and 19.9.3.3.

R19.9.3 The minimum amount of bonded reinforcement in two-way flat slab systems is based on reports (Joint ACI-ASCE Committee 423 1958; ACI 423.3R). Limited research available for two-way flat slabs with drop panels (Odello and Meaty 1967) indicates that behavior of these particular systems is similar to the behavior of flat plates. ACI 423.3R was revised by ACI Committee 423 in 1983 to clarify that Section 19.9.3 applies to two-way flat slab systems.

19.9.3.1 Bonded reinforcement shall not be required in positive moment areas where f_t, the extreme fiber stress in tension in the precompressed tensile zone at service load (after allowance for all prestress losses), does not exceed $2\sqrt{f_c'}$.

R19.9.3.1 For usual loads and span lengths, flat plate tests summarized in the Joint ACI-ASCE Committee 423 (1958) report and experience since the ACI 318-63 code was adopted indicate satisfactory performance without bonded reinforcement in the areas described in 19.9.3.1.

19.9.3.2 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

R19.9.3.2 In positive moment areas, where the maximum computed concrete tensile stress exceeds $2\sqrt{f_c'}$ but is less than $6\sqrt{f_c'}$, in accordance with 19.3.3, a minimum bonded reinforcement area proportioned according to Eq. (19-5) is

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$$A_s = \frac{N_c}{0.5 f_y} \qquad (19\text{-}5)$$

where the value of f_y used in Eq. (19-5) shall not exceed 60,000 psi. Bonded reinforcement shall be uniformly distributed over precompressed tensile zone as close as practicable to extreme tension fiber.

19.9.3.3 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

$$A_s = 0.00075 A_{cf} \qquad (19\text{-}6)$$

where A_{cf} is the larger gross cross-sectional area of the slab-beam strips in two orthogonal equivalent frames intersecting at a column in a two-way slab.

Bonded reinforcement required by Eq. (19-6) 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.

19.9.4 The minimum ratio of nonprestressed reinforcement area to gross concrete area in the dome ring for cast-in-place dome rings shall be in accordance with 20.2.12.4(a). The dome ring reinforcement shall have sufficient strength to meet the requirements for dead- and live-load factors and for strength reduction factors.

19.9.5 Minimum length of bonded reinforcement required by 19.9.2 and 19.9.3 shall be as required in 19.9.5.1, 19.9.5.2, and 19.9.5.3.

19.9.5.1 In positive moment areas, minimum length of bonded reinforcement shall be one-third the clear span length ℓ_n and centered in positive moment area.

COMMENTARY

required. The tensile force N_c is computed at service load on the basis of an uncracked, homogeneous section.

R19.9.3.3 Research on unbonded post-tensioned two-way flat slab systems evaluated by Joint ACI-ASCE Committee 423 (1958), ACI 423.3R, Odello and Meaty (1967), and Mast (1998) 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 tributary spans, Eq. (19-6) is given on the basis of the equivalent frame as defined in 14.7.2 and pictured in Fig. R14.7.2. For rectangular slab panels, Eq. (19-8) 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 Smith and Burns (1974), Burns and Hemakom (1977), Hawkins (1981), PTI (1990), and Foutch et al. (1990).

19.9.4 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 19.7.3, or for tensile stress conditions at service load in accordance with 19.9.3.2, will be adequately anchored to develop tension or compression forces. For bonded reinforcement required by 19.9.2 or 19.9.3.3, but not required for flexural strength in accordance with 19.7.3, the minimum lengths apply. Research (Joint ACI-ASCE Committee 423 1974) on continuous spans shows that these minimum lengths provide adequate behavior under service load and factored load conditions.

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19.9.5.2 In negative moment areas, bonded reinforcement shall extend one-sixth the clear span ℓ_n on each side of support.

19.9.5.3 Where bonded reinforcement is provided for ϕM_n in accordance with 19.7.3, or for tensile stress conditions in accordance with 19.9.3.2, minimum length also shall conform to provisions of Chapter 12.

19.10—Statically indeterminate structures

19.10.1 Frames and continuous construction of prestressed concrete shall be designed for satisfactory performance at service load conditions and for adequate strength.

19.10.2 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.

19.10.3 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 19.10.4.

19.10.4 *Redistribution of negative moments in continuous prestressed flexural members*

19.10.4.1 Where bonded reinforcement is provided at supports in accordance with 19.9, it shall be permitted to decrease negative or positive moments calculated by elastic theory for any assumed loading, in accordance with 8.4.

19.10.4.2 The reduced moment shall be used for calculating redistributed moments at all other sections within the

R19.10—Statically indeterminate structures

R19.10.3 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 (refer to Bondy [2003], Lin and Thornton [1972], and Collins and Mitchell [1997]). The elastic deformations caused by a non-concordant 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: 1) determine moments due to dead and live load; 2) modify by algebraic addition of secondary moments; and 3) 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.

R19.10.4 *Redistribution of negative moments in continuous prestressed flexural members*

The provisions for redistribution of moments given in 8.4 apply equally to prestressed members. Refer to Breen et al. (1994) for a comparison of research results and to Section 18.10.4 of the ACI 318-99 code.

For the moment redistribution principles of 19.10.4 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

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spans. Static equilibrium shall be maintained after redistribution of moments for each loading arrangement.

19.11—Compression members—combined flexure and axial loads

19.11.1 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. Horizontal prestressing in walls of liquid-containing tanks with or without combined flexural stresses shall be proportioned for the service load stresses of this section.

19.11.2 *Limits for reinforcement of prestressed compression members*

19.11.2.1 Members with average compressive stress in concrete less than 225 psi shall have minimum reinforcement in accordance with 12.10.1, 10.9.1, and 10.9.2 for columns, or 15.4 for walls.

19.11.2.2 Except for walls, members with average compressive stress in concrete equal to or greater than 225 psi shall have all tendons enclosed by spirals or transverse ties in accordance with (a) through (d):

(a) Spirals shall conform to 12.10.1.4

(b) Transverse ties shall be at least No. 3 in size or welded wire reinforcement 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

19.11.2.3 For walls with average compressive stress in concrete equal to or greater than 225 psi, minimum reinforcement required by 15.4 shall not apply where structural analysis shows adequate strength and stability.

19.11.3 *Provisions for externally prestressed circumferential concrete walls*

19.11.3.1 Spacing of vertical prestressing elements shall not exceed 50 in.

19.11.3.2 The average vertical compressive force at the centroid of the tank wall element under maximum service loads after all losses shall not be less than 200 psi for liquid-containing elements. This requirement shall not apply to

COMMENTARY

series of tied arches. The minimum bonded reinforcement requirements of 19.9 serves this purpose.

R19.11—Compression members—combined flexure and axial loads

R19.11.2 *Limits for reinforcement of prestressed compression members*

R19.11.2.3 The minimum amounts of reinforcement, specified in 15.4 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.

R19.11.3.2 *Provisions for externally prestressed concrete and shotcrete walls*

When designing circumferentially prestressed walls, the following items should be considered:

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tank walls containing a steel diaphragm conforming to the requirements of Chapter 15.

19.11.3.3 For wall penetrations, the horizontal circumferential prestressed wires or strands normally required over the height of the penetration shall be relocated into circumferential bands immediately above and below the penetration in accordance with the following:

(a) Each band shall provide one-half of the relocated prestressing force

(b) The wires or strands shall not be located closer than 2 in. to the wall penetrations

(c) The wall thickness shall be adequate to resist 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

19.11.4 *Horizontal wall prestressing*

19.11.4.1 The horizontal prestressing shall be consistent with an elastic analysis using the elastic theory appropriate for the shape of the tank.

(a) The minimum calculated effective horizontal prestressing (after all losses) shall resist all forces caused by internal loads, plus stresses caused by through-the-wall-thickness thermal and moisture gradients (but not less than 100 psi).

(b) In lieu of calculations, a minimum residual compressive stress of 200 psi for the above-ground portion of the tank wall shall be provided, tapering linearly to 50 psi at 6 ft below ground, with the tank filled to the overflow level.

(c) For cylindrical tanks without roofs (open at the top of the wall), the residual compression shall include provision for the calculated stresses caused by vertical thermal and moisture gradients, or allowance for a minimum residual compressive stress of 400 psi for the exposed top portion of the tank wall, tapering to at least that required for the through-the-wall-thickness thermal and moisture gradients required above at not less than 6 ft below the normal operating level of the water surface.

19.11.4.2 Horizontal tendons in circular tanks or curved portions of noncircular tank wall shall be located entirely exterior to the centerline of the wall.

COMMENTARY

(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 well-consolidated concrete, free of honeycombing and cold joints, is essential for providing a durable concrete with low permeability

R19.11.3.3 Relocated prestressing is typically placed in circumferential bands within 1 to 2 ft above and below the wall penetration.

R19.11.4 The provisions of 19.11.4.2 and 19.11.4.3 are intended to prevent splitting of the inside portion of curved walls from the outside portion.

R19.11.4.1 Residual compression of 400 psi at the top of the wall of open-top cylindrical prestressed concrete tanks has been shown to prevent potential cracks at the top of the wall due to moisture and temperature differentials in the portions of the wall above and below the water line.

R19.11.4.2 Prestressing of the dome ring is used to eliminate or control the circumferential tension in the dome ring and the dome edge region. Additional prestress may be

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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. The dome edge ring is often proportioned such that the initial nominal compressive stresses are limited within a range of 400 to 1000 psi based on the net cross section of the ring.

19.11.4.3 In curved tank walls, the minimum vertical spacing of horizontal tendons shall not be less than 3 in. and shall limit the tensile stress in the concrete between adjacent tendons caused by tendon curvature to $\mathbf{1.2}\sqrt{f_c'}$ ($\mathbf{0.1}\sqrt{f_c'}$). Alternatively, hairpins or other nonprestressed reinforcement may be provided in the wall to counteract the radial tensile splitting forces in the wall created by curved post-tensioning tendons.

19.11.4.4 For prestressed dome rings, an effective prestressing force, after all losses, shall be provided to counteract no less than 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 reinforcement shall be maintained at reduced stress, or additional nonprestressed reinforcement shall be added, to obtain the required strength.

19.12—Slab systems

19.12.1 Factored moments and shears in prestressed slab systems reinforced for flexure in more than one direction shall be determined in accordance with provisions of 14.7 (excluding 14.7.7.4 and 14.7.7.5), or by more detailed design procedures.

R19.12—Slab systems

R19.12.1 Use of the equivalent frame method of analysis (refer to 14.7) 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 (refer to Smith and Burns [1974], Burns and Hemakom [1977], Hawkins [1981], PTI [2004], Gerber and Burns [1971], and Scordelis et al. [1959]). The referenced research also shows that analysis using prismatic sections or other approximations of stiffness may provide erroneous results on the unsafe side. Section 14.7.7.4 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 19.10.4. Section 14.7.7.5 is excluded from application to prestressed slab systems because the distribution of moments between column strips and middle strips required by 14.7.7.5 is based on tests for reinforced concrete slabs. Simplified methods of analysis using average coefficients do not apply to prestressed concrete slab systems.

19.12.2 ϕM_n of prestressed slabs required by 9.3 at every section shall be greater than or equal to M_u considering 9.2, 19.10.3, and 19.10.4. ϕV_n of prestressed slabs at columns

R19.12.2 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 (refer

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shall be greater than or equal to V_u considering 9.2, 9.3, 11.1, 11.11.2, and 11.11.6.2.

19.12.3 At service load conditions, all serviceability limitations, including specified limits on deflections, shall be met, with appropriate consideration of the factors listed in 19.10.2.

19.12.4 For uniformly distributed loads, spacing of tendons or groups of tendons in at least one direction shall not exceed eight times the slab thickness, nor 5 ft. Spacing of tendons also shall provide a minimum average effective prestress of 125 psi on the slab section tributary to the tendon or tendon group. For slabs with varying cross section along the slab span, either parallel or perpendicular to the tendon or tendon group, the minimum average effective prestress of 125 psi is required at every cross section tributary to the tendon or tendon group along the span. Concentrated loads and openings in slabs shall be considered when determining tendon spacing.

19.12.5 In slabs with unbonded tendons, bonded reinforcement shall be provided in accordance with 19.9.3 and 19.9.4.

19.12.6 Except as permitted in 19.12.7, in slabs with unbonded tendons, a minimum of two 1/2 in. diameter or larger, seven-wire, post-tensioned strands shall be provided in each direction at columns, either passing through or anchored within the region bounded by the longitudinal reinforcement of the column. Outside column and shear cap faces these two structural integrity tendons shall pass under any orthogonal tendons in adjacent spans. Where the two structural integrity tendons are anchored within the region bounded by the longitudinal reinforcement of the column,

COMMENTARY

to Smith and Burns [1974], Burns and Hemakom [1977], Hawkins [1981], PTI [2004], Gerber and Burns [1971], and Scordelis et al. [1959]).

R19.12.3 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.

R19.12.4 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. The minimum average effective prestress of 125 psi was used in two-way test panels in the early 1970s to address punching shear concerns of lightly reinforced slabs. For this reason, the minimum effective prestress should be provided at every cross section.

For prestressed membrane and controlled slabs-on-ground, the minimum average effective prestress should be in accordance with the provisions of Chapter 21.

If the slab thickness varies along the span of a slab or perpendicular to the span of a slab, resulting in a varying slab cross section, the 125 psi minimum effective prestress and the maximum tendon spacing is required at every cross section tributary to the tendon or group of tendons along the span, considering both the thinner and the thicker slab sections. Note that this may result in higher than the minimum f_{pc} in thinner cross sections, and tendons spaced at less than the maximum in thicker cross sections along a span with varying thickness, due to the practical aspects of tendon placement in the field.

R19.12.6 Unbonded prestressing tendons that pass through the slab-column joint at any location over the depth of the slab suspend the slab following a punching shear failure, provided the tendons are continuous through or anchored within the region bounded by the longitudinal reinforcement of the column and are prevented from bursting through the top surface of the slab (ACI 352.1R). Between column or shear cap faces, structural integrity tendons should pass below the orthogonal tendons from adjacent spans so that vertical movements of the integrity tendons are restrained by the orthogonal tendons. Where tendons are distributed

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the anchorage shall be located beyond the column centroid and away from the anchored span.

19.12.7 Prestressed slabs not satisfying 19.12.6 shall be permitted, provided they contain bottom reinforcement in each direction passing within the region bounded by the longitudinal reinforcement of the column and anchored at exterior supports as required by 14.3.8.5. The amount of bottom reinforcement in each direction shall be not less than 1.5 times that required by Eq. (10-3) and not less than $300b_w d/f_y$, where b_w is the width of the column face through which the reinforcement passes. Minimum extension of these bars beyond the column or shear cap face shall be equal to or greater than the bar development length required by 12.8.2.1.

19.12.8 In lift slabs, bonded bottom reinforcement shall be detailed in accordance with 14.3.8.6.

19.13—Post-tensioned tendon anchorage zones

19.13.1 *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

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in one direction and banded in the orthogonal direction, this requirement can be satisfied by first placing the integrity tendons for the distributed tendon direction and then placing the banded tendons. Where tendons are distributed in both directions, weaving of tendons is necessary and use of 19.12.7 may be an easier approach.

R19.12.7 In some prestressed slabs, tendon layout constraints make it difficult to provide the structural integrity tendons required by 19.12.6. In such situations, the structural integrity tendons can be replaced by deformed bar bottom reinforcement (ACI 352.1R).

R19.13—Post-tensioned tendon anchorage zones

Section 19.13 was extensively revised in ACI 318-99 and was made compatible with the "Standard Specifications for Highway Bridges" (AASTHO 1996) and the recommendations of *NCHRP Report 356* (Breen et al. 1994).

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.

R19.13.1 *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. R19.13.1(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.19.13.1(b)) and behind the anchorage device. The entire shaded region should be considered, as shown in Fig. R19.13.1(b).

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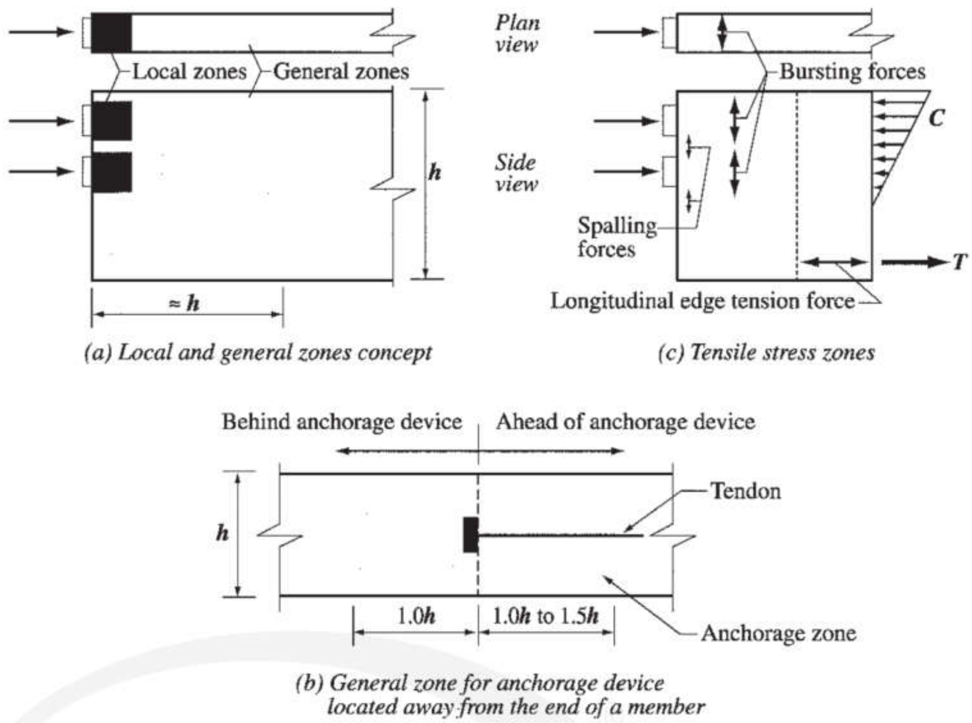

Fig. R19.13.1—Anchorage zones.

19.13.2 *Local zone*

19.13.2.1 Design of local zones shall be based upon the factored prestressing force P_{pu} and the requirements of 9.2.5 and 9.3.2.5.

19.13.2.2 Local-zone reinforcement shall be provided where required for proper functioning of the anchorage device.

19.13.2.3 Local-zone requirements of 19.13.2.2 are satisfied by 19.14.1 or 19.15.1 and 19.15.2.

R19.13.2 *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.

19.13.3 *General zone*

19.13.3.1 Design of general zones shall be based upon the factored prestressing force P_{pu} and the requirements of 9.2.5 and 9.3.2.5.

19.13.3.2 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.

R19.13.3 *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. R19.13.1(c). Also, the compressive stresses immediately ahead (as shown in Fig. R19.13.1(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.

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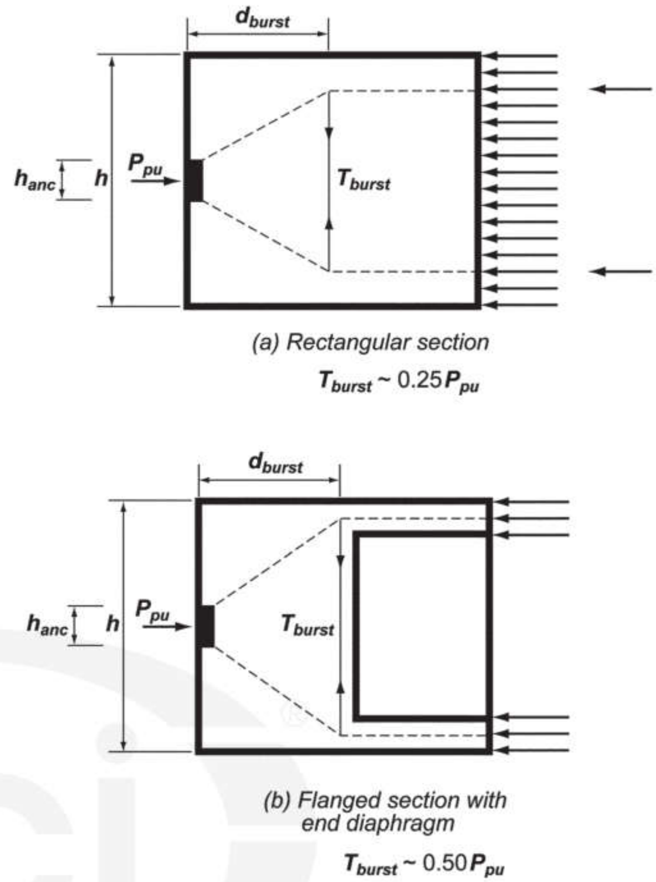

Fig. R19.13.3—Effect of cross section change.

19.13.3.3 The general-zone requirements of 19.13.3.2 are satisfied by 19.13.4, 19.13.5, 19.13.6, and whichever one of 19.14.2 or 19.14.3 or 19.15.3 is applicable.

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. R19.13.3.

19.13.4 *Nominal material strengths*

19.13.4.1 Tensile stress 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**.

19.13.4.2 Except for concrete confined within spirals or hoops providing confinement equivalent to that corresponding to Eq. (10-8), compressive strength in concrete at nominal strength in the general zone shall be limited to $0.7\lambda f_{ci}'$.

19.13.4.3 Compressive strength of concrete at time of post-tensioning shall be specified in the contract documents. 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

R19.13.4 *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 (refer to Breen et al. [1994]). The value for nominal tensile strength of bonded prestressing steel is limited to the yield strength of the prestressing steel because Eq. (19-3) may not apply to these nonflexural applications. The value for unbonded prestressing steel is based on the values of 19.7.2(b) and (c) but is somewhat limited for these short-length nonflexural applications. Test results given in Breen et al (1994) 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

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prestressing force, prestressing steel shall not be stressed until compressive strength of concrete, 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.

19.13.5 *Design methods*

COMMENTARY

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 contract documents. 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 one-third to one-half the final prestressing force.

R19.13.5 *Design methods*

The list of design methods in 19.13.5.1 includes those procedures for which fairly specific guidelines have been given in AASHTO (1996) and Breen et al. (1994). These procedures have been shown to be conservative predictors of strength when compared with test results (Breen et al. 1994). The use of strut-and-tie models is especially helpful for general zone design (Breen et al. 1994). In many anchorage applications, where substantial or massive concrete regions surround the anchorages, simplified equations can be used, except in the cases noted in 19.13.5.2.

For many cases, simplified equations based on AASHTO (1996) and Breen et al. (1994) 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. (R19-1) and (R19-2), respectively. The terms of Eq. (R19-1) and (R19-2) are shown in Fig. R19.13.5 for a prestressing force with small eccentricity. In the applications of Eq. (R19-1) and (R19-2), the specified stressing sequence should be considered if more than one tendon is present

$$T_{burst} = 0.25\sum P_{pu}\left[1-\frac{h_{anc}}{P_{pu}}\right] \qquad \text{(R19-1)}$$

$$d_{burst} = 0.5(h - 2e_{anc}) \qquad \text{(R19-2)}$$

where $\sum P_{pu}$ is the sum of the total factored prestressing force for the stressing arrangement considered, lb; h_{anc} is the depth of anchorage device or single group of closely spaced devices in the direction considered, in.; e_{anc} is 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.; and h is 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,

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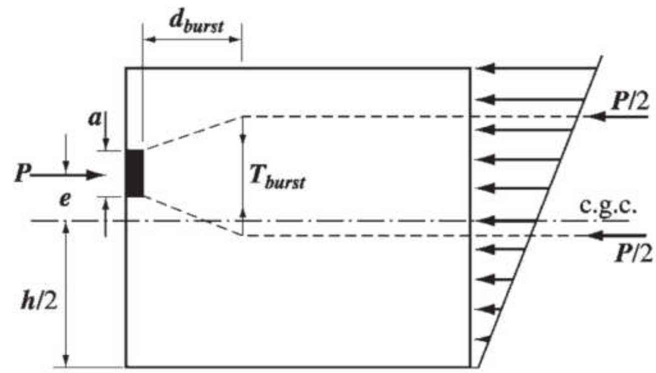

Fig. R19.13.5—Strut-and-tie model example.

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 AASHTO (1996).

19.13.5.1 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.

19.13.5.2 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.

19.13.5.3 The stressing sequence shall be specified on the design drawings and considered in the design.

R19.13.5.3 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.

19.13.5.4 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.

R19.13.5.4 The provision for three-dimensional effects was so that the 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).

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19.13.5.5 For anchorage devices located away from the end of the member, bonded reinforcement shall be provided to transfer at least $\mathbf{0.35}P_{pu}$ 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.

19.13.5.6 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.

19.13.5.7 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.

19.13.5.8 Tensile strength of concrete shall be neglected in calculations of reinforcement requirements.

19.13.6 *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.

19.14—Design of anchorage zones for monostrand or single 5/8 in. diameter bar tendons

19.14.1 *Local zone design*

Monostrand or single 5/8 in. or smaller diameter bar anchorage devices and local zone reinforcement shall meet the requirements of ACI 423.7 or the special anchorage device requirements of 19.15.2.

19.14.2 *General-zone design for slab tendons*

19.14.2.1 For anchorage devices for 0.5 in. or smaller diameter strands in normalweight concrete slabs, minimum reinforcement meeting the requirements of 19.14.2.2 and 19.14.2.3 shall be provided unless a detailed analysis satisfying 19.13.5 shows such reinforcement is not required.

19.14.2.2 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 $\mathbf{1/2}h$ ahead of each device. Those bars shall extend at least 6 in. either side of the outer edges of each device.

19.14.2.3 If the center-to-center spacing of anchorage devices is 12 in. or less, the anchorage devices shall be

COMMENTARY

R19.13.5.5 Where anchorages are located away from the end of a member, local tensile stresses are generated behind these anchorages (refer to Fig. R19.13.1(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 $\mathbf{0.35}P_{pu}$ was developed using 25 percent of the unfactored prestressing force being resisted by reinforcement at $\mathbf{0.6}f_y$.

R19.14—Design of anchorage zones for monostrand or single 5/8 in. diameter bar tendons

R19.14.2 *General-zone design for slab tendons*

For monostrand slab tendons, the general-zone minimum reinforcement requirements are based on the recommendations of ACI 423.3R, which shows typical details. The horizontal bars parallel to the edge required by 19.14.2.2 should be continuous where possible.

The tests on which the recommendations of Breen et al. (1994) 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 423.3R 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.

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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.

19.14.2.4 For anchorage devices not conforming to 19.14.2.1, minimum reinforcement shall be based upon a detailed analysis satisfying 19.13.5.

19.14.3 *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 19.13.3 through 19.13.5.

19.15—Design of anchorage zones for multistrand tendons

19.15.1 *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.

19.15.2 *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.

COMMENTARY

Both ACI 423.3R and Breen et al. (1994) 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 19.14.2.3 have the meaning shown in Fig. R19.13.1.

In those cases where multistrand anchorage devices are used for slab tendons, 19.15 is applicable.

The bursting reinforcement perpendicular to the plane of the slab required by 19.14.2.3 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.

R19.14.3 *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 R19.13.5 is allowed, unless 19.13.5.2 governs. More complex conditions can be designed using strut-and-tie models. Detailed recommendations for use of such models are given in ACI 352.1R and ACI 423.7, as well as in R19.13.5.

R19.15—Design of anchorage zones for multistrand tendons

R19.15.1 *Local zone design*

Refer to R19.13.2.

R19.15.2 *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.

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19.15.3 *General-zone design*

Design for general zones for multistrand tendons shall meet the requirements of 19.13.3 through 19.13.5.

19.16—Corrosion protection for unbonded single-strand prestressing tendons

19.16.1 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.

19.16.2 Sheathing shall be extruded 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.

19.16.3 The sheathing shall be connected to all stressing, intermediate and fixed anchorages in a 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-strand tendon systems shall meet the hydrostatic pressure testing requirements of ACI 423.7 except with a hydrostatic pressure of 10 psi, instead of the specified 1.25 psi.

19.16.4 Unbonded single-strand tendons shall be protected against corrosion in accordance with the provisions of ACI 423.7 except as noted above and as follows:

1) The anchorages shall be fully encapsulated including a watertight plastic coating of the anchorage.

2) Cuts and other damage to the plastic sheathing shall be repaired to be watertight prior to concrete placement.

3) The hydrostatic testing that supplements ACI 423.7 shall also include an intentional "nick" removed from the sheathing measuring at least 1/4 in. (6 mm) wide by 1 in. long repaired with the same waterproof plastic tape and methods proposed to be used in the field.

4) After performing the hydrostatic test, wipe the exterior of the tendon down and air dry at a temperature between 65 and 75°F (18 and 24°C) for a maximum of 24 hours. Remove end caps, sleeves, and the sheathing over the strand length to examine the tendon for any evidence of

COMMENTARY

R19.16—Corrosion protection for unbonded single-strand prestressing tendons

R19.16.1 For walls of noncylindrical liquid-containing structures using unbonded tendons, consideration should be given to increasing the minimum reinforcement requirement of 15.4 and 19.9. 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 ACI 423.7-07.

R19.16.2 Typically, sheathing is a continuous, seamless, high-density polyethylene 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.

R19.16.3 A liquid-tight connection may be achieved either by using special connector pieces, which provide a 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 23 ft head of water. More restrictive requirements for liquid-tightness may be specified for special applications where a high hydrostatic pressure is anticipated.

R19.16.4 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. Since 2015, ACI 423.7 (the successor to ACI 423.6), requires "full encapsulation" for all unbonded single-strand tendons. The additional provision on filling of voids in the end and intermediated anchorage area is to prevent the possible accumulation of water and the associated corrosion in the anchorage area. Care should be taken when tying unbonded tendons to not cut through the plastic sheathing.

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water infiltration, including staining of the white post-tensioning coating.

Tendon systems that were certified under the provisions of Section 18.16.4 of ACI 350-06 need not be retested when installed within 2 years of the adoption date of this Code.

19.16.5 The foreman of the installation crew and the Inspector shall be certified by the PTI as a "Level 2 Unbonded PT Ironworker" or "Level 2 Unbonded PT Inspector"; at least one-half of installation personnel shall have "Level I Unbonded PT Field Installation" certification or equivalent acceptable to the licensed design professional.

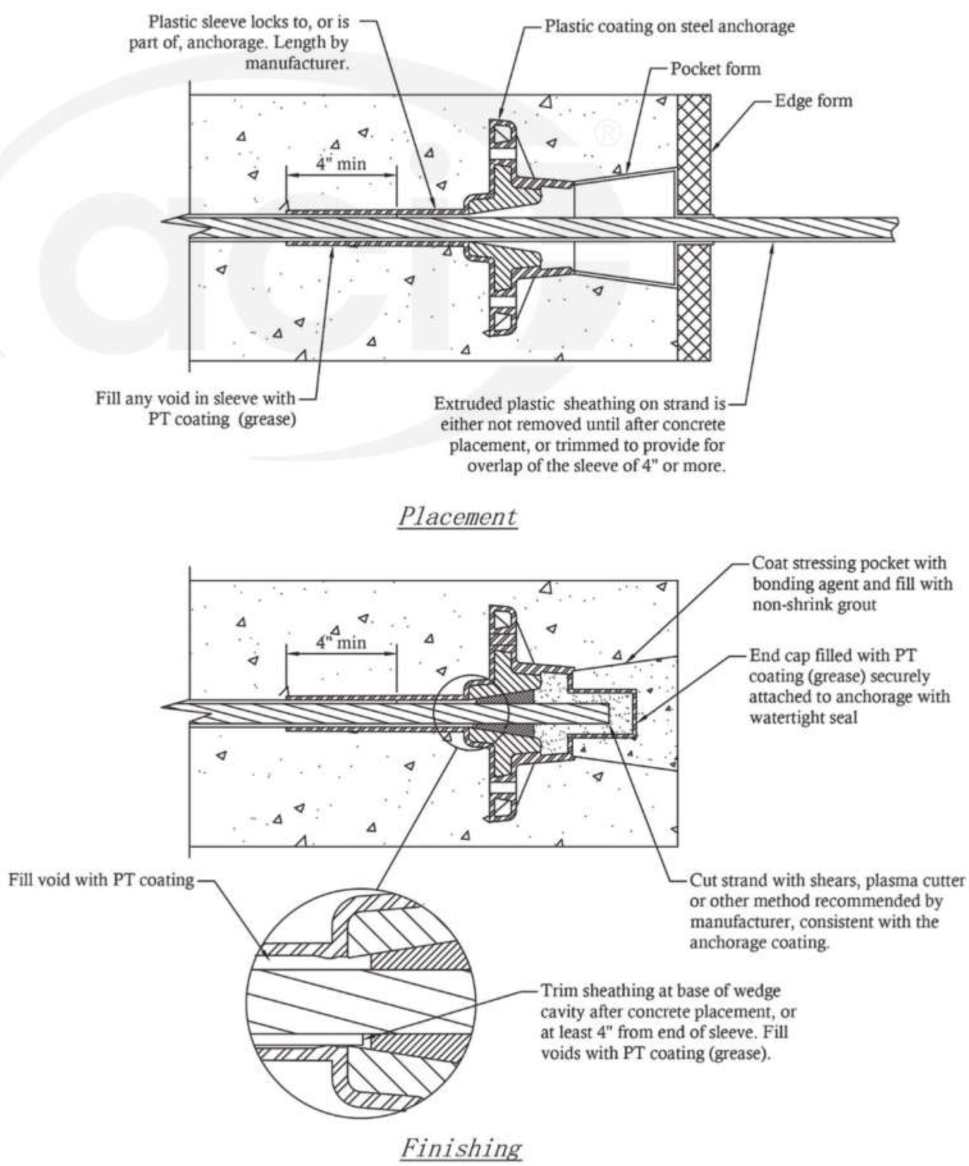
19.16.6 Sheathing shall either:

1) Be trimmed at the base of the wedge cavity inside the anchorage, after placement of the concrete, and immedi-

COMMENTARY

R19.16.5 Unbonded single-strand tendons should be installed, overseen, and inspected by individuals certified by an independent training and certification program acceptable to the licensed design professional.

R19.16.6 The corrosion protection features required for all unbonded tendons at a stressing end used in environmental engineering concrete structures are illustrated in Fig. R19.6.6.



R19.6.6 — Post-tensioning ducts.

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ately prior to placing wedges and stressing the tendon at stressing ends, or

2) If they are to be trimmed prior to concrete placement at stressing or dead ends, and temperature changes between the time of placing the tendons in the forms and placing the concrete cause the end of the sheathing to pull out of the stressing or dead-end sleeves, they shall be reinserted, or additional sheathing placed, to the overlap required by ACI 423.7, and sealed as for sheathing repairs, and the sleeve refilled with PT coating material prior to concrete placement.

19.16.7 After stressing, strand tails shall be removed in a manner acceptable to the anchorage manufacturer and shall not damage the anchorage, wedges, or cap-anchorage seal. The end cap shall be placed, voids in the wedge cavity area shall be filled with corrosion-protective material in the same manner as used for the pressure testing certification of the anchorages, and the stressing pocket shall be cleaned of any contamination, coated with an epoxy bonding agent and filled with non-shrink grout.

19.16.8 Ducts provided for insertion of unbonded tendons in precast wall, roof, or other precast elements shall be filled with post-tensioning grout prior to stressing.

19.17—Post-tensioning ducts

19.17.1 Ducts for grouted tendons shall be mortar-tight and nonreactive with concrete, prestressing steel, grout, and corrosion inhibitor.

19.17.2 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.

19.17.3 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.

19.17.4 Ducts shall be maintained free of liquid if members to be grouted are exposed to temperatures below freezing prior to grouting.

19.18—Grout for bonded tendons

COMMENTARY

R19.17—Post-tensioning ducts

R19.17.4 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 (AASHTO 1989).

R19.18—Grout for bonded tendons

Proper grout and grouting procedures are critical to post-tensioned construction (Gerwick 1971; PTI 2003). 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 3.2) for use with this grout is deemed inappropriate because of a lack of experience or tests with

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cementitious materials other than portland cement and a concern that some cementitious materials might introduce chemicals listed as harmful to tendons in R19.18.2. Thus, "portland cement" in 19.18.1 and "water-cement ratio" in 19.18.3.3 are retained in the Code.

19.18.1 Grout shall consist of portland cement and water; or portland cement, sand, and water; or a 100-percent-solids, two-component epoxy resin system. The use of admixtures in the grout mixture is acceptable if permitted by the licensed design professional.

R19.18.1 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.

19.18.2 Materials for grout shall conform to 19.18.2.1 through 19.18.2.4

R19.18.2 The limitations on admixtures in 3.8 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. 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.

19.18.2.1 Portland cement shall conform to 3.2.

19.18.2.2 Water shall conform to 3.4.

19.18.2.3 Sand, if used, shall conform to ASTM C144 except that gradation shall be permitted to be modified as necessary to obtain satisfactory workability.

19.18.2.4 Admixtures conforming to 3.8 and known to have no injurious effects on grout, steel, or concrete shall be permitted. Calcium chloride shall not be used.

19.18.2.5 Epoxy grout shall be moisture-insensitive with a minimum compressive strength of 125 percent of the design concrete compressive strength.

19.18.3 *Selection of grout proportions*

19.18.3.1 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

(b) Prior documented experience with similar materials and equipment and under comparable field conditions.

R19.18.3 *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 approximately 4000 psi. The handling and placing properties of grout are usually given more consideration than strength when designing grout mixtures.

19.18.3.2 Cement used in the Work shall correspond to that on which selection of grout proportions was based.

19.18.3.3 Water content shall be minimum necessary for proper pumping of grout; however, water-cement ratio (***w/c***) shall not exceed 0.45 by weight.

19.18.3.4 Water shall not be added to increase grout flowability that has been decreased by delayed use of grout.

19.18.3.5 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.

19.18.4 *Mixing and pumping grout*

R19.18.4 *Mixing and pumping grout*

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19.18.4.1 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.

19.18.4.2 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.

19.18.4.3 Grout temperatures shall not be above 90°F during mixing and pumping.

19.19—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.

19.20—Application and measurement of prestressing force

19.20.1 Tendon prestressing force shall be determined by both of (a) and (b):

(a) Measurement of steel elongation. Required elongation shall be determined from average load-elongation curves for the prestressing steel used

(b) Observation of jacking force on a calibrated gauge 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

19.20.2 External wrapping prestressing force shall be determined in accordance with the following:

(a) A calibrated force recording device that can be readily recalibrated shall be used to determine force levels in prestressing steel throughout the wrapping process. At least one force reading for every vertical foot of wall per layer shall be taken after the prestressing wire or strand has been applied on the wall.

(b) The total initial prestress force per vertical foot of wall height shall not be less than the total design minimum

COMMENTARY

In an ambient temperature of 35°F, grout with an initial minimum temperature of 60°F may require as many 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 temperature of 35°F. Quickset grouts, if 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 exceeding 90°F will lead to difficulties in pumping.

R19.20—Application and measurement of prestressing force

R19.20.1 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" (PCI 1999).

Section 19.18.1 of ACI 318-89 was revised to permit 7 percent tolerance in tendon force determined by gauge 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 ACI 318-63 was proposed by Joint ACI-ASCE Committee 423 in 1958, 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.

R19.20.2 Measurements 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 measurements should be made on straight lengths of wire.

A written record of stress measurements, including location and layer, should be maintained. This submission should be reviewed by the licensed design professional 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

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initial force nor more than 5 percent greater than the total design minimum initial force.

(c) If the force in the installed prestressed reinforcement is less than the specified initial prestress, additional wire or strand shall be applied to correct the deficiency. If the force exceeds 1.07 times the initial prestress, the wrapping operation shall be discontinued, overstressed wire or strand shall be removed, and cause determined and corrected before wrapping is resumed.

19.20.3 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.

19.20.4 Long lengths of exposed pretensioned strand shall be cut near the member to minimize shock to concrete.

19.20.5 Total loss of prestress due to unreplaced broken prestressing steel shall not exceed 2 percent of total prestress.

19.21—Post-tensioning anchorages and couplers

19.21.1 Anchorages and couplers for bonded and unbonded tendons shall develop at least 95 percent of the f_{pu}, when tested in an unbonded condition, without exceeding anticipated set. For bonded tendons, anchorages and couplers shall be located so that 100 percent of f_{pu} shall be developed at critical sections after the prestressing steel is bonded in the member.

COMMENTARY

allows no loss of tension between the reading and final placement on the wall.

R19.20.4 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.

R19.21—Post-tensioning anchorages and couplers

R19.21.1 In the 1986 interim ACI 318 code provisions, the separate provisions for strength of unbonded and bonded tendon anchorages and couplers presented in 19.19.1 and 19.19.2 of ACI 318-83 were combined into a single, revised 19.21 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 and ACI 423.7. 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 (Salmons

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19.21.2 Couplers shall be placed in areas approved by the licensed design professional and enclosed in housing long enough to permit necessary movements.

19.21.3 In unbonded construction subject to repetitive loads, special attention shall be given to the possibility of fatigue in anchorages and couplers.

19.21.4 Anchorages, couplers, and end fittings shall be permanently protected against corrosion.

19.22—External post-tensioning

19.22.1 Post-tensioning tendons shall be permitted to be external to any concrete section of a member, provided they are adequately protected against corrosion. The strength and serviceability design methods of this Code shall be used in evaluating the effects of external tendon forces on the concrete structure.

19.22.2 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.

19.22.3 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.

19.22.4 The details of the corrosion-protection method shall be indicated in the contract documents.

COMMENTARY

and McCrate 1977), or by bond tests on other tendon materials, as appropriate.

R19.21.3 For discussion on fatigue loading, refer to ACI 215R.

For detailed recommendations on tests for static and cyclic loading conditions for tendons and anchorage fittings of unbonded tendons, refer to Section 4.1.3 of 423.3R-05 and Section 15.2.2 of ACI 301-10.

R19.21.4 Requirements for protection of anchorages, couplers, and end fittings are given in ACI 423.7.

R19.22—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 upgrade existing structures and permits a wide variety of tendon arrangements.

Additional information on external post-tensioning is given in Barth (1997).

R19.22.3 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.

R19.22.4 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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CHAPTER 20—SHELLS AND FOLDED PLATE MEMBERS

20.1—Scope and definitions

20.1.1 Provisions of Chapter 20 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 wall construction.

20.1.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of Chapter 20, shall apply to thin-shell structures.

20.1.3 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.

20.1.4 folded plates—a special class of shell structures formed by joining flat, thin slabs along their edges to create a three-dimensional spatial structure.

COMMENTARY

CHAPTER R20—SHELLS AND FOLDED PLATE MEMBERS

R20.1—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 (ACI 334.1R), and continued with the inclusion of Chapter 20 in the 1971 Code. The 1982 revision of ACI 334.1R reflected additional experience in design, analysis, and construction and was 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 (IASS Working Group No. 5 1979).

Because Chapter 20 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. Performance of shells and folded plates requires special attention to detail (Tedesko 1980).

R20.1.1 Chapter 20 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 ACI 334.1R and ACI 372R.

R20.1.3 Common types of thin shells are domes (surfaces of revolution) (Billington 1982; ASCE Task Committee 1963), cylindrical shells (ASCE Task Committee 1963), barrel vaults, conoids, elliptical paraboloids (*Concrete Thin Shells* 1971), hyperbolic paraboloids, and groined vaults (Esquillan 1960). 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 freeform shells.

R20.1.4 Folded plates may be prismatic (Billington 1982; ASCE Task Committee 1963), nonprismatic (ASCE Task Committee 1963), or faceted. The first two types consist generally of planar thin slabs joined along their longitu-

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20.1.5 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.

20.1.6 auxiliary members—ribs or edge beams that serve to strengthen, stiffen, or support the shell; usually, auxiliary members act jointly with the shell.

20.1.7 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.

20.1.8 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.

COMMENTARY

dinal 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.

R20.1.5 Ribbed shells (*Concrete Thin Shells* 1971; Esquillan 1960) 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.

R20.1.6 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.

R20.1.7 Elastic analysis of thin shells and folded plates can be performed using 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 (*Hyperbolic Paraboloid Shells* 1988), finite differences (*Concrete Thin Shells* 1971), or numerical integration techniques (*Concrete Thin Shells* 1971; Billington 1990), 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 (*Concrete Thin Shells* 1971) or folded plate (ASCE Task Committee 1963).

R20.1.8 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 (Scordelis 1990; Schnobrich 1991).

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20.1.9 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.

20.2—Analysis and design

20.2.1 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.

20.2.2 Inelastic analyses shall be permitted to be used where 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.

20.2.3 Equilibrium checks of internal resistances and external loads shall be made to ensure consistency of results.

20.2.4 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.

20.2.5 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.

COMMENTARY

R20.2—Analysis and design

R20.2.1 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. In such cases, it may be assumed 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.

R20.2.2 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. Scordelis (1990) and Schnobrich (1991) indicate possible solution methods.

R20.2.4 Experimental analysis of elastic models (Sabnis et al. 1983) 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 stages should be considered for important shells of unusual size, shape, and/or complexity.

For model analysis, only those portions of the structure that significantly affect 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.

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.

R20.2.5 Solutions that include both membrane and bending effects and satisfy conditions of compatibility and equilibrium are encouraged. 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

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COMMENTARY

satisfied, while the strain compatibility equations are not. In complex structures where several shells join, 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.

20.2.6 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.

R20.2.6 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 shell-supporting members should be taken into account.

20.2.7 The thickness of a shell, 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. 3, except for thin shell spherical domes with prestressed dome rings as discussed in 20.2.12.

The provisions of Chapter 17 shall apply if elements are precast. If composite action is involved, the provisions of Chapter 18 shall be satisfied.

R20.2.7 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. Reinforcement sufficient to minimize cracking under service load conditions should be provided. The thickness of the shell is often dictated by the required reinforcement and the construction constraints, by the requirements of 20.2.8, cover, by being subjected to an environmental exposure, or by the Code minimum thickness requirements.

When shell or folded plate elements are precast and connected by cast-in-place segments, composite action is involved.

20.2.8 Shell instability shall be investigated and shown by design to be precluded.

R20.2.8 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 behavior must be considered in determining safety against instability (IASS Working Group No. 5 1979).

The resistance to buckling for thin shell spherical domes with prestressed dome rings is addressed in 20.2.12.2.

Investigation of thin shells for stability should consider the effect of: 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 successfully used to improve resistance to buckling include the provision of two mats of reinforcement—(one near each outer surface of the shell), a local increase

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COMMENTARY

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 (IASS Working Group No. 5 1979).

20.2.9 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 8.12, 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 8.12.

R20.2.9 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.

20.2.10 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.

R20.2.10 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.

20.2.11 In a region where membrane cracking is predicted, the nominal compressive strength parallel to the cracks shall be taken as $\mathbf{0.4}f_c'$.

R20.2.11 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 (Gupta 1984; Vecchio and Collins 1986). For the alternate design method, the compressive strength f_c' parallel to the cracks should be replaced by $\mathbf{0.4}f_c'$ in calculations involving I.3.1(a) or I.6.1.

20.2.12 *Thin shell spherical dome with prestressed dome ring*

20.2.12.1 *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.

R20.2.12.1 Refer to ACI 372R, *Concrete Shell Buckling* (1981), Baker et al. (1972), Billington (1982), Flugge (1960), Ghali (1979), and Heger et al. (1982) for design aids relating to elastic shell analysis. Zarghamee and Heger (1983) provides methods for designing thin concrete domes against buckling.

20.2.12.2 *Thickness*

Dome shell thickness is governed either by buckling resistance, by minimum thickness for practical construction, or by corrosion protection of reinforcement.

(a) The dome minimum thickness is the largest of the thicknesses calculated from Eq. (20-1).

$$h_d = r_d\sqrt{\frac{1.5}{\phi B_i E_c}\left(\frac{p_u}{B_c}+E_v\right)}\text{ , in.} \qquad (20\text{-}1)$$

(b) The value $\boldsymbol{p_u}$ used in Eq. (20-1) for such domes shall be the largest of (1), (2) and (3):

(1) $$p_u = U_1 = 1.4D\text{, lb/ft}^2 \qquad (20\text{-}2)$$

where $\boldsymbol{B_c} = \mathbf{0.44}$ and $\boldsymbol{E_v} = \mathbf{0}$.

R20.2.12.2 A method for determining the minimum thickness of a monolithic concrete spherical dome shell to provide adequate buckling resistance is given in Zarghamee and Heger (1983). 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 $\boldsymbol{B_i}$, $\boldsymbol{B_c}$, and $\boldsymbol{r_i}$ are discussed in Zarghamee and Heger (1983). The values for these factors given in Paragraphs (3) and (4) are recommended for use in Eq. (20-1) 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 350.5. When the dome thickness design is not governed

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(2) $$p_u = U_2 = 1.2D + 1.6L\text{, lb/ft}^2 \tag{20-3}$$

where $\boldsymbol{B_c = 0.44 + 0.003L}$ but not greater than 0.53 and $\boldsymbol{E_v = 0}$.

(3) $$p_u = U_3 = 1.2D + 0.2S\text{, lb/ft}^2 \tag{20-4}$$

where $\boldsymbol{B_c = 0.44 + 0.000375S}$.

In calculating $\boldsymbol{E_v}$, an importance factor $\boldsymbol{I = 1.0}$, response modification factor $\boldsymbol{R = 1.0}$, and vertical acceleration equal to $\boldsymbol{2/3S_{DS}}$ of the lateral mapped acceleration from ASCE/SEI 7 or site-specific vertical acceleration $\boldsymbol{A_{vss}}$ shall be used.

For site-specific ground motions:

$$E_v = A_{vss}(D + 0.2S) \tag{20-5}$$

For mapped ground motions:

$$E_v = 2/3S_{DS}(D + 0.2S) \tag{20-6}$$

The remaining terms in Eq. (20-1) are determined in accordance with Eq. (20-7) and (20-8).

$$\phi = 0.6 \tag{20-7}$$

$$B_i = \left(\frac{r_d}{r_i}\right)^2 \tag{20-8}$$

In the absence of other criteria, the maximum $\boldsymbol{r_i}$ shall be equal to $\boldsymbol{1.4r_d}$. For this case, where $\boldsymbol{B_i = 0.5}$, $\boldsymbol{r_i}$ is based on an imperfection of diameter $\boldsymbol{d}$ in accordance with Eq. (20-9) and as shown in Fig. 20.2.12.2.

$$d = 2.5(r_d h_d/12)^{0.5}\text{, ft} \tag{20-9}$$

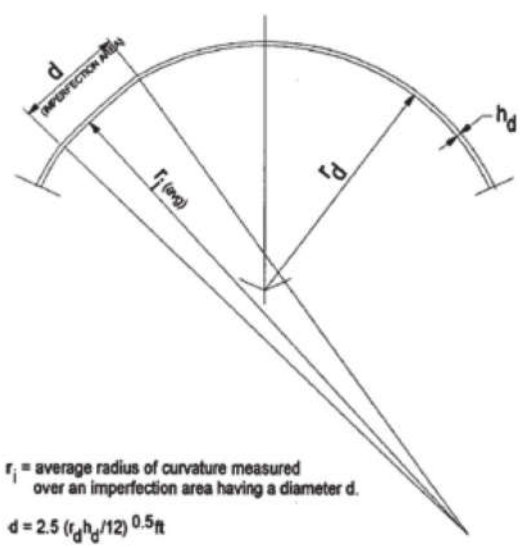

Fig. 20.2.12.2—Geometry of dome imperfection (adapted from Heger et al. [1982]).

COMMENTARY

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(c) Thickness of precast concrete panel dome shells shall not be less than the thickness obtained using Eq. (20-1) when joints between the panels are at least 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 if the minimum thickness of the panel is increased above the value given in Eq. (20-1) 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 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. (20-1).

(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 the deflection due to self-weight exceeds 10 percent of the thickness prior to their final incorporation as a part of the complete dome.

(g) The thickness of domes shall not be less than 3 in. for monolithic concrete and shotcrete, 4 in. for precast concrete, and 3 in. for the outer shell of a ribbed dome.

20.2.12.3 *Reinforcement*

For monolithic domes, the minimum ratio of temperature and shrinkage nonprestressed reinforcement area to concrete area shall be 0.0025 in both the circumferential and meridional 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 in compliance with the minimum concrete cover and reinforcement requirements of Chapter 12.

20.2.12.4 *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.

(b) The dome ring reinforcement shall have sufficient strength to meet the requirements given in Appendix D 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.

COMMENTARY

by buckling, the forms may be removed prior to 7 days if the required design strength has been obtained.

R20.2.12.3 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.

R20.2.12.4 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 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.

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stressed reinforcement shall meet the minimum concrete cover and reinforcement requirements of Chapter 12.

(d) Maximum initial prestress in wires and strands shall comply with Chapter 19.

(e) Maximum initial compression stress in dome rings shall comply with Chapter 19.

20.3—Design strength of materials

20.3.1 Specified compressive strength of concrete, f_c', at 28 days shall not be less than 3000 psi.

20.3.2 Specified yield strength of nonprestressed reinforcement, f_y, shall not exceed 60,000 psi.

20.4—Shell reinforcement

20.4.1 Shell reinforcement shall be provided to resist tensile stresses from internal membrane forces, 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.

Reinforcement for thin shell spherical domes with prestressed dome rings shall be in accordance with 20.2.12.

20.4.2 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, μ, shall not exceed that specified in 11.6.4.3.

COMMENTARY

R20.3—Design strength of materials

R20.3.1 Refer to Chapter 4 for minimum 28-day compressive strength f_c' requirements for concrete that is in contact with various exposure conditions.

R20.4—Shell reinforcement

R20.4.1 At any point in a shell, two different kinds of internal forces may occur simultaneously: those associated 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. Limiting control of membrane crack width and spacing due to shrinkage, temperature, and service load conditions is a major design consideration.

R20.4.2 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 (Gupta 1984; Fialkow 1991; Medwadowski 1989) 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 equating 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

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COMMENTARY

at factored loads. It is not necessary to calculate principal stresses if the alternative approach is used.

20.4.3 The area of shell reinforcement at any section as measured in two orthogonal directions or each principal direction shall not be less than 0.0028 times the cross-sectional area.

R20.4.3 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.

20.4.4 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 14.

20.4.5 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.

R20.4.5 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.

20.4.6 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.

R20.4.6 Generally, for all shells, and particularly in regions of substantial tension, the orientation of reinforcement approximates the directions of the principal tensile membrane forces. However, in some structures it is not possible to detail the reinforcement to follow the stress trajectories. For such cases, orthogonal component reinforcement is allowed.

20.4.7 If the direction of reinforcement varies more than 10 degrees from the direction of principal tensile membrane force, the amount of reinforcement shall be reviewed in relation to cracking at service loads.

R20.4.7 When the directions of reinforcement deviate significantly (more than 10 degrees) 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 limited if necessary.

Permissible crack widths for service loads under different environmental conditions are given in ACI 224R. 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.

20.4.8 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.

R20.4.8 The practice of concentrating tensile reinforcement in the regions of maximum tensile stress has led to numerous 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 limit crack width and spacing.

20.4.9 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

R20.4.9 The design method should ensure that the concrete sections, including consideration of the reinforcement, are capable of developing the internal forces required by the equations of equilibrium (Gupta 1986). The sign of

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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.

20.4.10 Shell reinforcement in any direction shall not be spaced farther apart than 12 in. nor farther apart than three times the shell thickness.

20.4.11 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 $\mathbf{1.2\ell_d}$ but not less than 18 in.

20.4.12 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 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 $\boldsymbol{\ell_d}$ with not more than one-third of the reinforcement spliced at any section.

20.5—Construction

20.5.1 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 $\boldsymbol{E_c}$ used 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 licensed design professional.

For thin shell spherical domes, the modulus of elasticity used to evaluate form removal shall be determined as provided in 8.5.1 and forms shall not be removed until the concrete reaches its required design strength.

20.5.2 Contract documents 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.

20.5.3 For tolerance of thin shell spherical domes, the average maximum radius of any area on the surface of the dome with a diameter of $\mathbf{2.5}\sqrt{\boldsymbol{r_d}\,\boldsymbol{h_d}}$ shall not be greater than $\mathbf{1.4}\boldsymbol{r_d}$.

COMMENTARY

bending moments may change rapidly from point to point of a shell. For this reason, reinforcement to resist bending, 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.

R20.4.10 The value of ϕ to be used is that prescribed in 9.3.2.1 for axial tension.

R20.4.11 and R20.4.12 On curved shell surfaces, it is difficult to control the alignment of precut reinforcement. This should be considered to avoid insufficient splice and development lengths. Sections 20.4.11 and 20.4.12 specify extra reinforcement length to maintain the minimum lengths on curved surfaces.

R20.5—Construction

R20.5.1 When early removal of forms is necessary, the magnitude of the modulus of elasticity at the time of proposed form removal must be investigated to ensure safety of the shell with respect to buckling, and to restrict deflections (Tedesko 1953, 1980). The value of the modulus of elasticity $\boldsymbol{E_c}$ 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 $\boldsymbol{f_c'}$ is determined for the field-cured specimen.

R20.5.2 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. Attention is needed when using air-supported form systems.

R20.5.3 In thin shell spherical domes, screeds and formwork are typically dimensioned to a precision of 1/16 in. Tolerance for the dome formwork and finishing operation should be as required to meet the geometrical imperfection coefficient $\boldsymbol{\beta_i}$ for thin shell spherical domes and the cover requirements of Chapter 12. The tolerance requirements provided in the Code for thin shell domes are depicted in Fig. 20.2.12.2.

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CHAPTER 21—LIQUID-CONTAINING GROUND-SUPPORTED SLABS

21.1—Scope

21.1.1 Provisions of this chapter shall apply to the design of membrane slabs and controlled slabs-on-ground that are used for liquid-tight barriers under fluid loads and subjected to uniform or concentrated vertical loads that induce a negligible flexural response in the slab. Requirements for the design of structural slabs on ground are covered in other sections of the code.

21.1.2 The subsurface conditions at a site shall be known to determine the soil-bearing capacity, compressibility, shear strength, and drainage characteristics. This information is obtained from soil borings, test pits, auger probes, load tests, sampling, laboratory testing, nondestructive test methods, and analysis by a geotechnical professional.

COMMENTARY

CHAPTER R21—LIQUID-CONTAINING GROUND-SUPPORTED SLABS

R21.1—Scope

Membrane slabs are thin, flexible slabs that accept gradual differential foundation settlements without flexural cracking, thus maintaining durability. The flexibility of a membrane slab allows uniform loads on the slab from stored liquid to be directly transmitted to the foundation soil without lateral distribution. Membrane slabs may also be designed to accommodate concentrated loads of limited magnitude. Flexural tensile stresses in membrane slabs are limited by 21.4.2 for the purpose of mitigating cracking of these liquid-containing elements.

Membrane slabs are highly reinforced concrete slabs directly supported by foundation soils. The high level of reinforcement is necessary to control cracking due to shrinkage in slabs without adequate movement joints. The high level of reinforcement controls cracking while still allowing for deflection. These slabs are commonly used as a liquid-tight barrier in circular prestressed concrete liquid-containing tanks. They are also used in rectangular tanks and other configurations. Membrane slabs are typically used in storage tanks where the floor is continually fully saturated.

Controlled slabs-on-ground are reinforced concrete elements supported by foundation soils with relatively uniform stiffness characteristics and are capable of support and distribution of loads. Controlled slabs-on-ground are frequently used for conditions where greater thickness and relative stiffness are required compared to a membrane slab, due to concentrated loads imposed on the controlled slab-on-ground. Controlled slabs-on-ground are expected to provide a liquid-tight barrier under fluid loads.

When membrane slabs and controlled slabs-on-ground are not saturated under normal service conditions, they may be exposed to more severe thermal and moisture gradients that should be considered in the design.

Structural slabs-on-ground distribute applied loads over large areas of the subgrade. They also are used to bridge applied loadings over soft spots or loss of support in the subgrade, to distribute wall and column loads to the subgrade, to transmit hydrostatic uplift forces to the walls and columns above the slab, or transmit pressures from the subgrade to the structure above, such as from expansive soils. Mat foundations are an example of structural slabs-on-ground.

R21.1.2 Geotechnical design criteria are generally obtained from one or more of the following: soil borings; test pits; auger probes; load tests; sampling; laboratory testing; or nondestructive test methods, along with analysis by a geotechnical professional. Recommendations on geotechnical requirements, allowable settlements and additional information on membrane slabs are provided in Appendix A of ACI 372R-13. Equation (A3-1) of ACI 372R-13 provides guidance concerning the maximum tolerable differential settlement for a given slab thickness and radius, limiting the

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CODE

COMMENTARY

tensile stresses to within acceptable levels for a membrane slab system.

21.2—Slab support

21.2.1 The subsurface conditions at a site shall be investigated to determine the design soil-bearing capacity, compressibility, shear strength, and drainage characteristics.

21.2.2 Potential settlements of the foundation soils shall be accounted for in the slab design. Local hard and soft spots, if not avoidable, shall be considered in the slab design. Consideration shall be given to slabs founded on more than one type of foundation soil condition, such as part cut and part fill.

21.2.3 Disturbed subgrade or loosely consolidated soil, or foundation material that is otherwise unsatisfactory for the imposed loadings or control of settlement shall be corrected with in-place soil modification or removed and replaced with acceptable fill material in accordance with the recommendations of a geotechnical professional.

21.2.4 Provisions shall be made to prevent erosion of the subgrade due to water flowing below the slab. If the in-place soils cannot be made acceptable, they shall be removed and replaced with an acceptable fill material in accordance with the recommendations of a geotechnical professional. The gradation of fill and subgrade shall permit free drainage without loss of fines, or a geotextile fabric shall be provided.

21.2.5 Tolerances for the elevation of the prepared subgrade directly beneath controlled slabs-on-ground, and prestressed membrane slabs 5 in. or greater in thickness, shall be +0 in. and –1 in. For other membrane slabs, the tolerances for the elevation of the prepared base material directly beneath the membrane slab shall be +0 in. and –1/2 in. All transitions in elevation shall be smooth and gradual.

21.2.6 A granular base material shall be used beneath the membrane slab. This material shall be stable, compactable angular or subangular with 100 percent passing the 1 in. sieve size and not more than 8 percent passing the No. 200 sieve size. Alternatively, a road base material can be used that satisfies the requirements for construction and long-term stability based on the recommendations of a licensed

R21.2—Slab support

R21.2.2 Soft spots are generally removed and replaced with a lean concrete, controlled low-strength material (CLSM), or compacted structural fill. A layer of compacted structural fill of suitable thickness should be placed over the entire slab area when the slab is placed in a cut-fill area or where partial or full rock excavation is required. The intent of the compacted structural fill layer is to provide a uniform bearing support for the slab and avoid the effect of abrupt changes in the foundation soil stiffness, which may cause high stress concentrations.

R21.2.3 Compaction of the subgrade and structural fill material 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 D1557 or ASTM D698, 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 D1556. In-place soil modification may include preconsolidation, geosynthethic reinforcing materials, grout injection, vibro-compaction, vibro-replacement, or rammed aggregate piers. Additional information on in-place soil modification may be found in ACI 372R-13 Appendix A.

R21.2.4 Erosion of subgrade could be caused by water flow due to groundwater, site runoff, or in the event of leakage of the structure or its piping and connections.

R21.2.6 A well-graded granular base material is critical for membrane slabs to achieve construction requirements for fine grading tolerances and stability for bar supports, and reduce the possibility of base soil disturbance due to foot traffic during construction. The gradation of the base material should be selected to reduce the possibility of degradation of the base material and subgrade soils due to water

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design professional. The granular base material shall have a minimum thickness of 6 in.

flow caused by rain, site runoff or, in the event of leakage, to maintain long-term stability. Refer to ACI 372R for further guidance on granular base material. A controlled slab-on-ground has higher cover requirements and less-restrictive tolerances and, therefore, a base material is less critical and not a requirement of the Code.

Base material should be compacted to 95 percent of the maximum laboratory density as determined by ASTM D1557. The field tests for measurement of in-place density should be in conformance with ASTM D1556 or ASTM D6938. If the base material is cohesionless, the relative density should be measured in accordance with ASTM D4253 and ASTM D4254. A relative density of 70 to 75 percent is normally desirable. Alternatively, if the base layer is relatively thin (8 in. or less), ASTM compaction and density tests can be replaced by compaction performance criteria in which the maximum lift, number of passes in each direction, type, and weight of equipment are specified by the licensed design professional.

21.2.7 A minimum 6 mil-thick plastic sheeting shall be placed on top of the granular base material and used directly beneath a membrane slab.

R21.2.7 The plastic sheeting beneath a membrane slab provides a separation between the concrete and granular base material described in 21.2.7 and is not intended to act as a vapor barrier. This prevents the intermixing of the concrete and base material during placement of the concrete. The plastic sheeting also reduces the frictional drag between the bottom of the slab and the base material. The reduction in frictional resistance should be considered by the licensed design professional in high seismic regions and in situations with differential backfill where the slab is used as part of the load path for transferring lateral loading into the soil.

21.3—Slab thickness

21.3.1 The minimum thicknesses for controlled slabs-on-ground are:

(a) 5 in. for slabs with one layer of nonprestressed reinforcement
(b) 5 in. for slabs with prestressed reinforcement
(c) 6 in. for slabs with top and bottom nonprestressed reinforcement

21.3.2 The maximum thicknesses of controlled slabs-on-ground shall be 8 in. Slabs in excess of 8 in. shall be considered structural slabs.

21.3.3 The minimum thicknesses for membrane slabs shall be 4 in. for slabs with nonprestressed or prestressed reinforcement.

21.3.4 The maximum thicknesses of membrane slabs shall be:

(a) 6 in. for nonprestressed slabs
(b) 7 in. for slabs with prestressed reinforcement.

R21.3—Slab thickness

R21.3.4 Concrete membrane slabs should be as thin as practical with consideration given to construction methods and concrete cover of reinforcement.

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21.3.5 Tolerances for finished controlled slab-on-ground or membrane slab surface elevation shall be –0 in. and +3/4 in. with no greater difference than ±1/4 in. in 10 ft.

21.4—Reinforcement

21.4.1 Liquid-containing ground-supported slabs shall meet the requirements for minimum temperature and shrinkage reinforcement of structural slabs as described in Chapter 12. The following additional provisions shall apply for each type of slab:

(a) Controlled slabs-on-ground with nonprestressed reinforcement shall be reinforced in one or two layers.

(b) Membrane slabs with nonprestressed reinforcement shall be reinforced with one layer. The minimum ratio of reinforcement area to concrete area shall be 0.005 in each orthogonal direction. Maximum spacing of bar reinforcement shall be the lesser of 12 in. or two times the slab thickness.

(c) Prestressed membrane and controlled slabs-on-ground shall have sufficient prestressed reinforcement to impart a minimum average effective prestress in the slab in accordance with 19.12.4. The prestressed tendons shall be in each orthogonal direction and shall be located within the center one-third of the slab. The tendons shall be partially or fully tensioned after the concrete compressive strength has been determined to be adequate to resist the anchorage forces in accordance with the requirements of Chapter 19. The minimum ratio of nonprestressed reinforcement area to concrete area shall be 0.0015 in each orthogonal direction.

21.4.2 Flexural tensile stresses in membrane slabs shall be limited to $6\sqrt{f_c'}$. Portions of the slab that do not meet this requirement shall be designed as a structural slab and the minimum cover requirements for a structural slab shall be used.

21.4.3 Reinforcement shall be maintained in correct vertical position by support chairs or concrete cubes. Reinforcement tolerances shall meet the requirements of 12.5.2.

21.5—Joints

21.5.1 Waterstops shall be used in all slab joints. Controlled slabs on ground and membrane slabs shall be placed continuously in sections as large as practical to avoid construction joints. Slab joints shall be designed in accordance with the requirements of Chapter 7 of the Code.

21.5.2 The design of membrane slabs and controlled slabs-on-ground shall take into account the effect of thickened slab sections and transitions where provided in the slab at joints.

COMMENTARY

R21.4—Reinforcement

R21.4.1 The minimum required reinforcement provides crack control for liquid-tight construction. Additional reinforcement should be provided at thickened slab edges, construction joints, other discontinuities, and as required by the design.

Welded-wire reinforcing sheets may be used as reinforcement for membrane slabs; however, caution should be exercised as follows:

(1) Maximum wire spacing for welded wire reinforcement should be 4 in.

(2) Welded-wire reinforcement should be placed in a single mat using flat sheets

(3) Consideration should be given to minimum cover requirements at splice locations when using welded wire reinforcement

(4) Consideration should be given to prevent excessive deformation of the welded wire reinforcement under construction loads

Slabs with prestressed reinforcement may need two stressing stages to prevent early shrinkage cracks from appearing. Typically, the initial stage (partial) stressing is completed within 24 to 48 hours after concrete placement. The partial tendon stress should be based on the compression strength of concrete cylinders stored in the same environmental conditions as the slab or maturity meters. Protection should be provided at the tendon anchorage prior to capping and grouting the end anchorage to prevent infiltration of water.

R21.4.2 The limitation of flexural tensile stresses for membrane slabs are to address edge effects and to enhance durability of these elements.

R21.5—Joints

R21.5.1 Entire membrane slabs and controlled slabs on ground should be cast with no cold joints and no construction joints, if practical. Factors to consider include expected experience of the contractor and weather conditions.

R21.5.2 Thickening of slabs at joints should be detailed to account for the fact that the joint has a greater rigidity than the slab. Locations of the slab that are thickened at joints may change the slab behavior from membrane action to a

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CODE

COMMENTARY

stiffened edge member that is partially or fully restrained. The resulting additional restraint may affect the required amount of reinforcement for volume change or flexural stresses.

21.6—Hydrostatic uplift

21.6.1 Slabs subject to hydrostatic uplift shall be provided with under-slab drainage or hydrostatic pressure relief valves or be designed to resist the uplift pressure. When pressure relief valves are used, the slab shall be designed to resist the uplift pressures required to initially open the pressure relief valve.

R21.6—Hydrostatic uplift

R21.6.1 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. When pressure relief valves are used, periodic inspection, testing, and maintenance is recommended to ensure that they will operate at their rated opening pressure. In addition, groundwater should be assumed to be above the elevation of the relief valve, due to the pressure required to open the valve, in accordance with the recommendations of ACI 350.4. Rock or soil anchors are a means of anchoring the slab against uplift if subgrade conditions permit. Groundwater can be directed to a manhole or other drainage structure where its level can be observed and measured. A high groundwater alarm system may be used to alert personnel and activate filling the tanks.

21.7—Curing

21.7.1 Concrete shall be cured in accordance with Section 5.13.

R21.7—Curing

R21.7.1 Preferred methods of water curing include ponding, soaking, sprinkling, use of wet coverings, or other methods to keep the concrete continuously wet.

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Notes

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CODE

CHAPTER 22—STRENGTH EVALUATION AND CONDITION ASSESSMENT OF STRUCTURES

22.1—General

This chapter covers strength evaluation and condition assessment of environmental concrete structures. Evaluation of strength and condition assessment, including liquid and gas tightness, of structures shall be conducted as directed by a licensed design professional. Conclusions and judgments of the test results shall be made by a licensed design professional.

22.1.1 *Strength evaluation—general*

22.1.1.1 Strength evaluation of a structure shall be conducted when there is a doubt that a part or all of a structure meets the strength and safety requirements of this Code.

22.1.1.2 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 22.2.

COMMENTARY

CHAPTER R22—STRENGTH EVALUATION AND CONDITION ASSESSMENT OF STRUCTURES

R22.1—General

R22.1.1 *Strength evaluation—general*

Chapter 22 does not cover load testing for the approval of design or construction methods. (Refer to 17.10 for recommendations on strength evaluation of precast concrete members.) Provisions of Chapter 22 may be used to evaluate whether a structure or a portion of a structure satisfies the safety requirements of the 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 22 provides requirements for investigating the strength and condition of structures. The strength and structural condition affect 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.

R22.1.1.2 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 22.2.

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22.1.1.3 A structural analysis and evaluation, based on actual existing structural and loading conditions, shall be performed to verify the structural adequacy and safety of the structure. The load-carrying capacity of critical structural components shall be determined by the strength design method or by the design method used for the original design of the structure as required by a licensed design professional.

R22.1.1.3 Structural components supporting nonstructural elements, such as pipe or equipment supports, should be included in the condition assessment.

22.1.1.4 The results of the structural analysis and condition assessment evaluation shall indicate which structural members require repairs and/or strengthening.

22.1.1.5 If the effect of the strength deficiency is not well understood or if it is not feasible to establish the existing dimensions and material properties required for analysis by measurement, a load test shall be required if the structure is to remain in service.

R22.1.1.5 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.

22.1.1.6 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.

R22.1.1.6 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 and/or maximum liquid level to a level determined to be appropriate.

The length of the specified time period should be based on consideration of: 1) the nature of the problem; 2) environmental and load effects; 3) service history of the structure; and 4) scope of the periodic inspection program. At the end of a specified 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 22.

22.1.2 *Condition assessment—general*

R22.1.2 *Condition assessment—general*

Chapter 22 covers both the strength and durability conditions of existing environmental engineering structures. The owner and a licensed design professional will determine if a condition assessment of a particular structure is required and what testing and field checking should be included. A condition assessment of a structure should begin with a visual observation using ACI 201.1R. It is beneficial for the owner to engage a licensed design professional who has designed environmental engineering structures, and who also has expertise in the condition assessment and the design of the repairs for concrete structures.

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22.1.2.1 Condition assessment of a structure is an integral part of the strength evaluation. Condition survey and field and laboratory testing shall be performed, as deemed necessary by a licensed design professional or as required by building official, to assess the actual structural condition of the structure, as well as to determine the strength and durability of the structural materials.

22.2—Determination of required dimensions and material properties

22.2.1 Dimensions of the structural elements shall be established at critical sections. Material property testing requirements shall be determined in accordance with 22.4.

22.2.2 Locations and sizes of the reinforcing bars, welded wire reinforcement, or tendons shall be determined in accordance with 22.3.6. It shall be permitted to base reinforcement locations on available drawings if spot checks are made confirming the information on the drawings.

22.2.3 Concrete compressive strength shall be based on results of tests of cores removed from the part of the structure where the strength is in question. The method for obtaining and testing cores shall be in accordance with ASTM C42.

COMMENTARY

R22.1.2.1 Tightness testing of concrete structures for the containment of liquids and low-pressure gases may be necessary to verify liquid or gas tightness and determine that the structure can fulfill its intended purpose.

R22.1.2.2 All available sources of relevant existing information concerning the original design, construction, materials, construction records, design and construction personnel, and service history of the structure should be consulted during the condition assessment and strength evaluation.

R22.1.2.3 Information obtained during the condition assessment should be used in developing appropriate remedial design details to correct deficiencies and/or any distress on the structure.

R22.2—Determination of required dimensions and material properties

This section applies if it is decided to make an analytical evaluation (refer to 22.1.1.2).

R22.2.1 Critical sections are those at which each type of stress calculated for the load in question reaches its maximum design value.

R22.2.2 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 this 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.

R22.2.3 ACI Committee 214 has developed two methods for determining f_c' from cores taken from an existing structure. These methods are described in ACI 214.4R and rely on statistical analysis techniques. The procedures described are only appropriate where the determination of an equivalent f_c' is necessary for the strength evaluation of an existing structure and should not be used to investigate low cylinder strength tests results in new construction, which is considered in 5.7.1.3.

The number of core 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).

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22.2.4 Reinforcement or prestressing steel strength shall be based on tensile tests of representative samples of the material in the structure in question.

22.2.5 Dimensions and material properties are determined through measurements and testing, and if analytical evaluations can be made in accordance with 22.1.1.2, it shall be permitted to increase the ϕ-factor in 9.3, but the ϕ-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: 0.9
Other reinforced members: 0.8
Shear and/or torsion: 0.8
Bearing on concrete: 0.8

22.3—Condition survey of structures

22.3.1 The as-built condition with respect to geometry, structural materials, loadings, and environment shall be verified against the available design documents. Discrepancies shall be recorded and their structural implications shall be evaluated in accordance with 22.2. Surveys indicated in Sections 22.3.2 to 22.3.6 shall be performed as deemed necessary by a licensed design professional to evaluate the existing condition of a structure or elements of the structure being evaluated.

22.3.2 All visible forms and areas of deterioration and structural distress in the structural elements shall be located, inspected, and recorded as to type, location, and degree of severity. Exploratory probing shall be performed to survey hidden conditions.

22.3.3 Suspect concrete surfaces shall be tested to detect voids, separations, and delaminations. Testing shall include sounding and other nondestructive methods, such as impact-echo, impulse response, and ground-penetrating radar methods.

22.3.4 Concrete crack surveys shall be performed. Location, direction, width, and depth of selected cracks shall be reported.

22.3.5 Visual leak survey shall be performed. Record location, severity, and visible chemical deposits. Moisture detection equipment, or infrared thermography shall be used to verify the existence of moisture and leakage at suspect locations.

22.3.6 Concrete cover measurements of the reinforcement shall be performed at selected areas. The size, number, and location of steel reinforcement shall be determined by the following methods or combination thereof: magnetic tests; pulse radar systems; and by removal of concrete cover. The concrete removal method shall be used to verify

COMMENTARY

R22.2.4 The number of tests required depends on the uniformity of the material and is best determined by the licensed design professional for the specific application.

R22.2.5 Strength reduction factors given in 22.2.5 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 22.2.5 were changed for the 2006 Code to be compatible with the load combinations and strength reduction factors of Chapter 9, which were revised at that time. For this edition, the strength reduction factor in 22.2.5 for members with spiral reinforcement was increased to correspond to an increase in this strength reduction factor in Chapter 9.

R22.3—Condition survey of structures

To provide an assessment of the structure's capability to support loads, contain liquids or gases, and to attain the life expectancy for the particular facility, it is most important to determine if the structure was built in accordance with the design plans and specifications. This would include wall heights, lengths, thicknesses; slabs, beams, and columns—spans, thicknesses, and sizes; floor slopes; mat or slab gradients; and drainage. The condition survey should determine the existence of and spacing of relief valves, as well as the existence of joints and waterstops, and their condition. The determination of the size and spacing of reinforcement may require both destructive probing and nondestructive testing. The condition survey should determine whether there is evidence of excessive settlements, deflections, or structure movement.

It is important to identify various forms, areas, and extent of deterioration and structural distress, including, but not limited to: excessive cracking; spalling; reinforcement corrosion; excessive deflection or structural movements; settlements; and gas or liquid leaks. If deemed necessary by the licensed design professional, the condition survey should also include nonstructural elements such as concrete fill, waterproofing, coatings, railings, and roofing, as their condition may affect the overall integrity of the structure. The resulting information will aid in the assessment of the structure for containing liquids or gases as well as to assist the licensed design professional in completing a structural analysis of the particular structure. A structural analysis, based on existing conditions, is completed to verify if the structure can safely support the specified loadings.

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calibrate the results of the nondestructive magnetic and radar methods.

22.4—Field and laboratory testing

Material testing indicated in Sections 22.4.1 to 22.4.9 shall be performed as deemed necessary by a licensed design professional to determine the existing material properties and conditions:

22.4.1 Compressive strength testing by using concrete cores extracted from representative structural elements, and nondestructive tests, such as rebound hammer, penetration resistance, pullout testing, and ultrasonic pulse can be used. Results from nondestructive testing shall be correlated with concrete core testing.

22.4.2 Petrographic and chemical analysis to determine the concrete properties such as: air content; potential for alkali-aggregate reactivity; cementitious materials content; chemical composition; contaminated aggregate; contaminated mixing water; frozen components; porosity; quality of aggregate; resistance to freezing and thawing; soundness; potential to sulfate attack; uniformity; and water-cementitious materials ratio.

22.4.3 Depth of carbonation, pH.

22.4.4 Abrasion resistance of concrete surface in accordance with ASTM C779.

22.4.5 Pullout or direct shear strength testing to determine the load strength of overlays and/or previous repair patches.

COMMENTARY

R22.4—Field and laboratory testing

This section outlines various field and laboratory testing procedures and methods that may be used for the condition assessment of concrete structures prior to rehabilitation. The testing results should be used as a guide and are not intended to replace judgment by the licensed design professional responsible for the evaluation. It should be recognized that there are no generally accepted criteria for evaluating serviceability and durability of existing structures. Engineering judgment is a critical element in the evaluation of reinforced concrete structures. The testing may be destructive or nondestructive and may be performed both in the field and in the laboratory or a combination of both. Depending on the particular function of the structure under investigation, certain tests should be performed to effectively assess the condition of the structure.

R22.4.1 Compressive strength testing is necessary to determine if the structure was built in accordance with specifications and to assist in the completion of the structural analysis.

R22.4.2 Petrographic tests can provide the investigator with information on the concrete such as: the cement type and approximate content; aggregate type, gradation, and approximate content; existence of pozzolans or admixtures; the approximate air content; evidence of alkali-aggregate reaction; or evidence of deterioration by chemical, electrical, or physical action between concrete and the service environment.

R22.4.3 Concrete pH or depth of carbonation defines the extent of protection of reinforcement by the concrete cover. As the concrete pH reduces from a high approximately 13 to below 8, the effectiveness of the concrete to protect the embedded reinforcement from corrosion is greatly reduced. Carbonation reduces the alkalinity of the concrete, which reduces the resistance of the reinforcing steel against corrosion. Carbonation can decrease the pH of the pore solution to 9.5 or lower at which level the corrosion passivation film is not stable and corrosion may initiate.

R22.4.4 Surface erosion is due to the action of abrasive materials carried by flowing water. Surface cavitation action causes erosion where high velocities and negative pressures are present. Abrasion resistance may be an important criterion for a particular structure, such as a culvert or flume.

R22.4.5 It may be advised to complete a pull-out test of an overlay or previous bonded repair to determine if composite action is affected.

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CODE

22.4.6 Chemical tests to determine the chloride content of the concrete at incremental depths from the surface to the closest reinforcement in accordance with ASTM C1218.

22.4.7 Tests for potential for sulfate attack. Test soils and groundwater in contact with concrete structures for sulfates of sodium, magnesium, potassium, and calcium.

22.4.8 The presence of sulfide ions and hydrogen sulfide gas shall be verified.

22.4.9 Half-cell potential measurements to determine the corrosion activity or the potential for corrosion of the reinforcement at the time of testing.

22.4.10 Testing and monitoring of vibrations.

22.5—Tightness testing

Tightness testing of concrete environmental engineering liquid and gaseous containment structures shall be performed when required by a licensed design professional. The types of testing are:

(a) Hydrostatic testing for open or covered structures
(b) Surcharged hydrostatic testing for closed structures
(c) Pneumatic testing for closed structures
(d) Combination of hydrostatic-pneumatic testing for closed structures

The tightness testing shall be performed in accordance with ACI 350.1.

22.6—Evaluation report

The results of the entire condition survey, testing, and structural evaluation shall be summarized in a final engineering report that shall be signed and sealed by a licensed design professional. The report shall include, but not be limited to, a brief description of the following areas: purpose and scope, general description of the existing construction, field observations and condition surveys, field and laboratory testing, structural evaluation, conclusions, and recommendations.

COMMENTARY

R22.4.6 Chloride contents can determine if the threshold for corrosion of reinforcement is met.

R22.4.8 Sulfide ions and hydrogen sulfide gas may generate biochemical activity, as certain microorganisms convert these to sulfuric acid, which has been known to attack the concrete in manholes and wastewater treatment structures. This condition should be noted in the condition survey.

R22.4.9 Corrosion rate and half-cell potential testing can determine if active corrosion is taking place at the time of the investigation.

R22.4.10 Environmental structures are often subjected to externally induced vibrations such as equipment vibrations, and vibrations caused by hydrodynamic loads developed by the flow of liquids. Structures that are subjected to equipment-induced vibrations and/or hydrodynamic loads may be susceptible to structural damage, depending on the magnitude of vibration. The vibration characteristics, such as peak velocities, accelerations, and displacements, should be measured and evaluated with respect to the structural integrity and long-term durability of the structure.

A licensed design professional should determine the acceptable limits of vibrations, which can cause damage to the structure, including immediate and long-term effects.

R22.5—Tightness testing

Tightness testing of concrete structures for the containment of liquids and low-pressure gases is provided in ACI 350.1. When tightness testing is required, such testing may need to be successfully performed before or after proceeding with the load testing for strength evaluation as determined by the licensed design professional.

R22.6—Evaluation report

The preceding information, along with test data, photographs, and field notes, is included in a condition survey report for recommended repairs and estimated quantities of recommended repairs. A suggested report outline includes:

(1) **Introduction:** This section includes a brief description of the structure and an explanation why the investigation was needed.

(2) **Purpose and scope:** This section includes a description of the purpose and a detailed scope of work as agreed

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CODE

COMMENTARY

with the owner. Modification to the scope of work over the course of the investigation should also be included.

(3) **Personnel:** This section includes the names of key personnel who worked on the project and their main responsibilities.

(4) **Authorization:** This section includes information on who authorized the investigation.

(5) **General description of the project:** This section includes information on site locations, type of construction, structural system, and history of construction including the original construction team and subsequent repair and rehabilitation programs.

(6) **Discussion:** This section includes information in relation to the review of available construction documents, field studies and testing, laboratory testing, and structural analysis and evaluation.

(7) **Conclusions:** This section includes conclusions on the cause of the damage or deterioration, the structural condition, the quality and durability of the existing structural materials, and the structural adequacy of the various structural components.

(8) **Recommendations:** This section includes the appropriate course of action, such as required repairs due to deterioration, and required strengthening of the structure to correct the identified deficiencies. If requested by the owner, this section shall include repair alternatives, cost estimates, project-specific constraints associated with working around existing operations and site access, and project repair scheduling.

22.7—Load testing

Load testing in accordance with ACI 437.2 shall be permitted to supplement the analysis and demonstrate the strength of the existing structure for resisting vertical gravity loads only. The licensed design professional shall determine if, and to what extent, testing for lateral loads, including liquid and earth pressures, shall be required.

R22.7—Load testing

Load testing is intended to verify whether a structure or portion of a structure satisfies the strength requirements of the Code for the specified loads. The procedures and requirements of the ACI 437.2 code were developed to be applied to structural members subjected to gravity loads and where the structural response is dominated by flexure, such as typical elements of the gravity load-carrying system in a building. ACI 437.2 is not applicable to structures where the response involves significant in-plane or membrane action, where soil-structure interaction is present, or in situations where the structural behavior is not typical of building structural elements. Hence, ACI 437.2 is not applicable for lateral liquid and earth pressure loads. Furthermore, for such loads, it is commonly impractical to apply test loads greater than the service loads, although it may be possible to increase the liquid depth above the normal operating depth if deemed appropriate for testing. Caution is advisable for any testing that may be required for lateral earth pressure because lateral earth pressure depends upon soil-structure interaction and can also vary over time due to the moisture content of the soil and the elevation of any groundwater.

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Notes

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CODE

APPENDIX A—ALTERNATE DESIGN METHOD

A.1—Scope

A.1.1 Nonprestressed reinforced concrete members shall be permitted to be designed using service loads (without load and environmental durability factors) and allowable stresses at service loads in accordance with provisions of Appendix A.

A.1.2 For design of members not covered by Appendix A, appropriate provisions of this Code shall apply.

A.1.3 All applicable provisions of this Code for nonprestressed concrete, except 8.4, shall apply to members designed by the Alternate Design Method.

A.1.4 Flexural members shall meet requirements for deflection control in 9.5, and requirements of 10.4, 10.5 and 10.7 of this Code.

A.1.5 The use of Alternate Design Method (Appendix I) of the 2006 Code is permissible instead of applicable sections of this Code.

COMMENTARY

APPENDIX RA—ALTERNATE DESIGN METHOD

RA.1—Scope

As an alternate to the strength design method of this Code, the design provisions of Appendix A 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 allowable 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 (refer to A.5).

The alternate design method is generally similar to the "working stress design method" of earlier ACI building codes (for example, ACI 318-63). However, significant differences in procedure occur in design of compression members with or without flexure (refer to A.6) and bond stress and development of reinforcement (refer to A.4). For shear, the shear strengths provided by concrete for the strength design method are divided by a factor of safety, and the resulting allowable stresses are restated in Appendix A (refer to A.7). 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.

RA.1.1 Design by Appendix A does not apply to prestressed members. (Chapter 19 permits linear stress-strain assumptions for computing service load stresses and prestress transfer stresses for investigation of behavior at service conditions.)

RA.1.3 All other provisions of this code, except those permitting moment redistribution, apply to the alternate design method. These include control of deflections as well as all the provisions related to slenderness effects in compression members in Chapter 10.

RA.1.4 The general serviceability requirements of this Code, such as the requirements for deflection control (refer to 9.5), must be met regardless of whether the strength method or the alternate method is used for design.

RA.1.5 Appendix I of the 2006 Code was based on the Gergely-Lutz equation (Gergely and Lutz 1968) and generally provided results that were slightly more conservative than designs by the current Appendix A based on the Frosch (1999) equation.

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A.2—General

A.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.

A.2.2 Allowable stresses may be increased by one-third where permitted by the governing building code, except that maximum steel stress shall not exceed $0.6f_y$.

A.2.3 For liquid retention, normal and severe environmental exposures are defined in 10.6.4.5.

A.2.4 Calculated flexural tensile stress in reinforcement at service load, f_s, shall be computed as the unfactored moment divided by the product of steel area and internal moment arm.

A.3—Allowable stresses at service loads

A.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 $2\sqrt{f_c'}$

(c) Bearing on loaded area§: $0.3f_c'$

(d) Tensile hoop stress in concrete: The service level tensile hoop stress in the transformed section of concrete in walls of nonprestressed circular tanks shall be limited to

$$f_{c,tension} = \frac{C_s E_s A_s + T_{hoop}}{A_g + \dfrac{E_s}{E_c} A_s} \le 0.1 f_c' \qquad \text{(A-1)}$$

where C_s is equal to 0.0003.

*For more detailed calculation of shear stress carried by concrete v_c and shear values for lightweight aggregate concrete, refer to A.7.4.

†Designed in accordance with 8.11 of this Code.

‡If shear reinforcement is provided, refer to A.7.7.4 and A.7.7.5.

§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, A2 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

RA.2—General

RA.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, except as required by the governing building code. Where load combinations are not otherwise defined, it is recommended to use the load combinations defined for allowable stress design in ASCE/SEI 7. When using the moment and shear equations of 8.3.3 and Chapter 14, the factored load w_u must be replaced by the service load w.

RA.3—Allowable stresses at service loads

Allowable 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 8.13 of the Code is already included in the $1.2\sqrt{f_c'}$ value for joists.

Refer to R10.14.1 for clarification of the use of areas A_1 and A_2 for increased bearing stress.

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A.3.2 Stresses in steel reinforcement shall not exceed the following:

(a) Flexural tension: Refer to A.3.3.

(b) Direct and hoop tension in normal environmental exposures: $f_{s,max}$ **= 20,000 psi**

(c) Direct and hoop tension in severe environmental exposures: $f_{s,max}$ **= 17,000 psi**

(d) Shear carried by shear reinforcement in normal environmental exposures: $f_{s,max}$ **= 24,000 psi**

(e) Shear carried by shear reinforcement in severe environmental exposures: $f_{s,max}$ **= 20,000 psi**

A.3.3—Distribution of flexural reinforcement

A.3.3.1 This section prescribes rules for distribution of flexural reinforcement and the allowable stresses used to control flexural cracking.

A.3.3.2 Distribution of flexural reinforcement in two-way slabs shall also meet the requirements of 14.3. For the application of A.3.3.4, 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 one-way members.

A.3.3.3 Flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross section as required by A.3.3.4.

A.3.3.4 The maximum allowable tensile stress $f_{s,max}$ in flexural reinforcement at service loads shall not exceed that given by Eq. (A-2) or (A-3) as applicable.

A.3.3.4.1 In normal environmental exposure areas as defined in 10.6.4.5:

$$f_{s,max} = \frac{320}{\beta\sqrt{s^2 + 4(2 + d_b/2)^2}} \tag{A-2}$$

but need not be less than 20,000 psi for one-way and 24,000 psi for two-way members and shall not exceed $\mathbf{0.6}f_y$ but no greater than 36,000 psi.

A.3.3.4.2 In severe environmental exposure areas as defined in 10.6.4.5:

$$f_{s,max} = \frac{260}{\beta\sqrt{s^2 + 4(2 + d_b/2)^2}} \tag{A-3}$$

but need not be less than 17,000 psi for one-way and 20,000 psi for two-way members and shall not exceed $\mathbf{0.6}f_y$ but no greater than 36,000 psi.

A.3.3.4.3 In Eq. (A-1) and (A-2), it shall be permitted to use the value 25 for the term $\mathbf{4(2 + d_b/2)^2}$ as a simplification.

COMMENTARY

RA.3.2 Allowable 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. ACI 224.2R provides guidance for estimating crack widths in axial tension members.

RA.3.3—Distribution of flexural reinforcement

Refer to R10.6.

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A.3.3.4.4 The strain gradient amplification factor shall be given by

$$\beta = \frac{h-c}{d-c} \quad \text{(A-4)}$$

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. (A-2) and (A-3).

A.3.3.5 Where appearance of the concrete surface is of concern and c_c exceeds 3 in., the service load flexural tension stress must not exceed the values given in A.3.3.4, and the spacing s of reinforcement closest to the surface in tension shall not exceed that given by

$$s = 15\left(\frac{36,000}{f_s}\right) - 2.5c_c \quad \text{(A-5)}$$

but not greater than 12 in.

A.3.3.6 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 8.12, 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.

A.3.3.7 Where h of a beam or joist exceeds 36 in., longitudinal skin reinforcement shall be uniformly distributed along both side-faces of the member. Skin reinforcement shall extend for a distance $h/2$ from the tension face. The spacing s shall be as provided in 10.6.4, where c_c is the least distance from the surface of the skin reinforcement or prestressing steel to the side face. 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.

A.4—Development and splices of reinforcement

A.4.1 Development and splices of reinforcement shall be as required in Chapter 12 of this Code.

A.4.2 In satisfying requirements of 12.8.11.3, M_n shall be taken as computed moment capacity assuming all positive moment tension reinforcement at the section to be stressed to the allowable tensile stress f_s, and V_u shall be taken as unfactored shear force at the section.

RA.4—Development and splices of reinforcement

In computing development lengths and splice lengths, the provisions of Chapter 12 govern both the strength method and the alternate design method 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.

A.5—Flexure

For investigation of stresses at service loads, straight-line theory (for flexure) shall be used with the following assumptions.

RA.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

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A.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 4/5 for simple spans, a nonlinear distribution of strain shall be considered. Refer to 10.7 of this Code.

A.5.2 Stress-strain relationship of concrete is a straight line under service loads within permissible service load stresses.

A.5.3 In reinforced concrete members, concrete resists no tension.

A.5.4 It shall be permitted to take the ratio E_s/E_c as the nearest whole number (but not less than 6). Except in calculations for deflections, value of E_s/E_c for lightweight concrete shall be assumed to be the same as for normalweight concrete of the same strength.

A.5.5 In doubly reinforced flexural members, an effective ratio of $2E_s/E_c$ shall be used to transform compression reinforcement for stress computations. Compressive stress in such reinforcement shall not exceed allowable tensile stress.

A.6—Compression members with or without flexure

A.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.

A.6.2 Slenderness effects shall be included according to requirements of 10.10. In Eq. (10-14), (10-21) and (10-22) 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.

A.6.3 Walls shall be designed in accordance with Chapter 15 of this code with flexure and axial load capacities taken as 40 percent of that computed using Chapter 15. In Eq. (15-1), ϕ shall be taken equal to 1.0.

A.7—Shear and torsion

A.7.1 Design shear stress v shall be computed by

$$v = \frac{V}{b_w d} \quad \text{(A-7)}$$

where V is design shear force at section considered.

A.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

COMMENTARY

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 the reinforcement limited to the allowable values.

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 the axial compression may be disregarded, the member should be investigated using A.6.

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 allowable tensile stress is then used to calculate the contribution of the compression reinforcement in computing the resisting moment at service loads.

RA.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 other earlier 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 A.

The permissible service load capacity is taken as 40 percent of the nominal axial load strength P_n at given eccentricity $(\phi = 1.0)$ 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. (10-14), (10-21), and (10-22) 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.

RA.7—Shear and torsion

For convenience, a complete set of design provisions for shear is provided in Appendix A.

The allowable 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 11.11.2 must be applied with the permissible

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shall be permitted to be designed for the same shear v as that computed at a distance d.

A.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.

A.7.4—Shear stress carried by concrete

A.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 A.7.4.4.

A.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 A.7.4.5.

A.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

$$v_c = 1.1\left(1 + 0.004\frac{N}{A_g}\right)\sqrt{f_c'} \tag{A-8}$$

where N is negative for tension. Quantity N/A_g shall be expressed in psi.

A.7.4.4 For members subject to shear and flexure only, it shall be permitted to compute v_c by

$$v_c = \sqrt{f_c'} + 1300\rho_w\frac{Vd}{M} \tag{A-9}$$

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.

A.7.4.5 For members subject to axial compression, it shall be permitted to compute v_c by

$$v_c = 1.1\left(1 + 0.0006\frac{N}{A_g}\right)\sqrt{f_c'} \tag{A-10}$$

Quantity N/A_g shall be expressed in psi.

A.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 $\sqrt{f_c'}$.

COMMENTARY

stresses on the critical section limited to those given in A.7.7.3.

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(b) When f_{ct} is not specified, the value of $\sqrt{f_c'}$ shall be multiplied by 0.75 for "all-lightweight" concrete and by 0.85 for "sand-lightweight" concrete. Linear interpolation shall be permitted when partial sand replacement is used.

A.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.

A.7.5—Shear stress carried by shear reinforcement

A.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 reinforcement with wires located perpendicular to axis of member making an angle of 45 degrees or more with longitudinal tension reinforcement

(c) Longitudinal reinforcement with bent portion making an angle of 30 degrees or more with longitudinal tension reinforcement

(d) Combinations of stirrups and bent longitudinal reinforcement

(e) Spirals

A.7.5.2 Design yield strength of shear reinforcement shall not exceed 60,000 psi.

A.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 12.8.13 of this code to develop design yield strength of reinforcement.

A.7.5.4 *Spacing limits for shear reinforcement*

A.7.5.4.1 Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/2$ nor 12 in.

A.7.5.4.2 Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45-degree 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.

A.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.

A.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

RA.7.5.4 *Spacing limits for shear reinforcement*

A 12 in. maximum spacing of reinforcement is required in liquid-containing structures for crack control.

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A.7.5.5 *Minimum shear reinforcement*

A.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 8.13 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

A.7.5.5.2 Minimum shear reinforcement requirements of A.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.

A.7.5.5.3 Where shear reinforcement is required by A.7.5.5.1 or by analysis, minimum area of shear reinforcement shall be computed by

$$A_v = 50\frac{b_w s}{f_y} \quad \text{(A-11)}$$

where b_w and s are in inches.

A.7.5.6 *Design of shear reinforcement*

A.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 A.7.5.6.2 through A.7.5.6.8.

A.7.5.6.2 When shear reinforcement perpendicular to axis of member is used

$$A_v = \frac{(v - v_c)b_w s}{f_s} \quad \text{(A-12)}$$

A.7.5.6.3 When inclined stirrups are used as shear reinforcement

$$A_v = \frac{(v - v_c)b_w s}{f_s(\sin\alpha + \cos\alpha)} \quad \text{(A-13)}$$

A.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

$$A_v = \frac{(v - v_c)b_w d}{f_s \sin\alpha} \quad \text{(A-14)}$$

where $(v - v_c)$ shall not exceed $1.6\sqrt{f_c'}$.

A.7.5.6.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at

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different distances from the support, required area shall be computed by Eq. (A-13).

A.7.5.6.6 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

A.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.

A.7.5.6.8 Value of $(v - v_c)$ shall not exceed $4.4\sqrt{f_c'}$.

A.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 11.6 of this Code shall be permitted to be applied, with limiting maximum stress for shear taken as 55 percent of that given in 11.6.5. Permissible stress in shear-friction reinforcement shall be that given in A.3.2.

A.7.7—Special provisions for slabs and footings

A.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:

A.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 A.7.1 through A.7.5.

A.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 A.7.7.2 and A.7.7.3.

A.7.7.2 Design shear stress v shall be computed by

$$v = \frac{V}{b_o d} \tag{A-15}$$

where V and b_o shall be taken at the critical section defined in A.7.7.1.2.

A.7.7.3 Design shear stress v shall not exceed v_c given by Eq. (A-16) unless shear reinforcement is provided

$$v_c = \left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'} \tag{A-16}$$

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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 A.7.4.6 shall apply.

A.7.7.4 If shear reinforcement consisting of bars or wires is provided in accordance with 11.11.3 of this code, v_c shall not exceed $\sqrt{f_c'}$, and v shall not exceed $3\sqrt{f_c'}$.

A.7.7.5 If shear reinforcement consisting of steel I- or channel-shaped sections (shearheads) is provided in accordance with 11.11.4 of this Code, v on the critical section defined in A.7.7.1.2 shall not exceed $3.5\sqrt{f_c'}$, and v on the critical section defined in 11.11.4.7 shall not exceed $2\sqrt{f_c'}$. In Eq. (11-35) and (11-36), design shear force V shall be multiplied by 2 and substituted for V_u.

A.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 11.9.6, the design axial load shall be multiplied by 1.2 if compression and 2.0 if tension, and substituted for N_u.

A.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 18.5.3 of this Code.

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APPENDIX B—STRUT-AND-TIE MODELS

B.1—Definitions

B-region—a portion of a member in which the plane sections assumption of flexure theory from 10.2.2 can be applied.

discontinuity—an abrupt change in geometry or loading.

D-region—the portion of a member within a distance ***h*** from a force discontinuity or a geometric discontinuity.

COMMENTARY

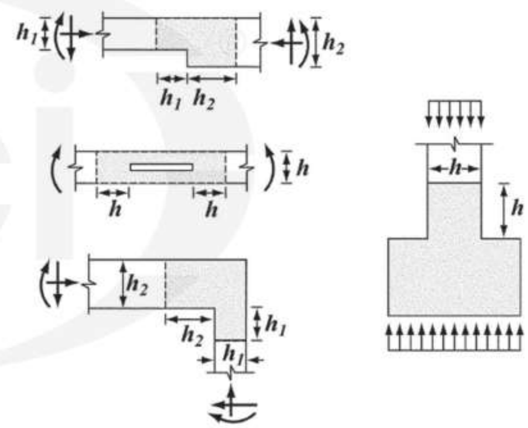
APPENDIX RB—STRUT-AND-TIE MODELS

RB.1—Definitions

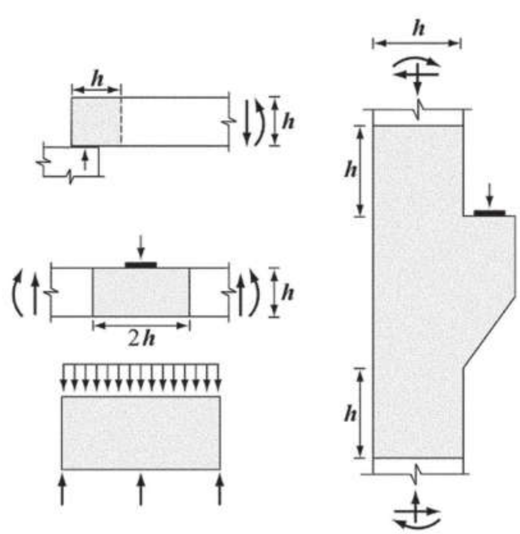
B-region—In general, any portion of a member outside of a D-region is a B-region.

discontinuity—A discontinuity in the stress distribution occurs at a change in the geometry of a structural element or at a concentrated load or reaction. Saint-Venant's principle indicates that the stresses due to axial load and bending approach a linear distribution at a distance approximately equal to the overall height of the member, ***h***, away from the discontinuity. For this reason, discontinuities are assumed to extend a distance ***h*** from the section where the load or change in geometry occurs. Figure RB.1.1(a) shows typical geometric discontinuities, and Fig. RB.1.1(b) shows combined geometrical and loading discontinuities.

D-region—The shaded regions in Fig. RB.1.1(a) and (b) show typical D-regions (Schlaich 1987). The plane sections assumption of 10.2.2 is not applicable in such regions.



(a) Geometric discontinuities



(b) Loading and geometric discontinuities

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Fig. RB.1.1—D-regions and discontinuities

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deep beam—refer to 10.7.1 and 11.8.1.

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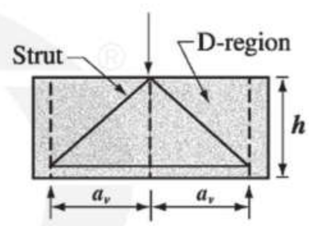
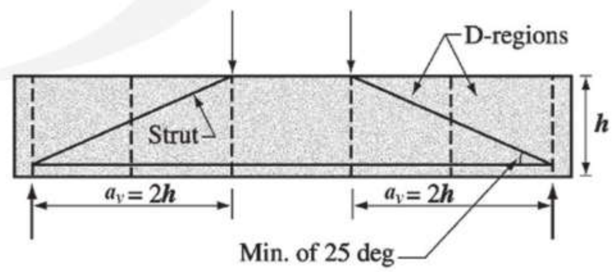
Each shear span of the beam in Fig. RB.1.2(a) is a D-region. If two D-regions overlap or meet as shown in Fig. RB.1.2(b), they can be considered as a single D-region for design purposes. The maximum length-to-depth ratio of such a D-region would be approximately 2. Thus, the smallest angle between the strut and the tie in a D-region is arctan 1/2 = 26.5 degrees, rounded to 25 degrees.

If there is a B-region between the D-regions in a shear span, as shown in Fig. RB.1.2(c), the strength of the shear span is governed by the strength of the B-region if the B- and D-regions have similar geometry and reinforcement (Collins and Mitchell 1991). This is because the shear strength of a B-region is less than the shear strength of a comparable D-region. Shear spans containing B-regions—the usual case in beam design—are designed for shear using the traditional shear design procedures from 11.1 through 11.4 ignoring D-regions.

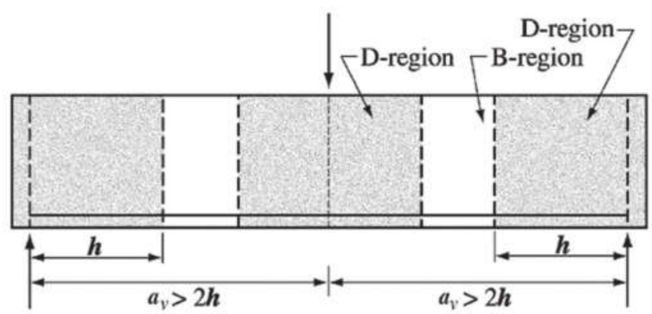
deep beam—Refer to Fig. RB.1.2(a), RB.1.2(b), and RB.1.3, and Sections 10.7 and 11.7.



*(a) Shear span, a_v < 2**h**, deep beam*



*(b) Shear span, a_v = 2**h**, limit for a deep beam*



*(c) Shear span, a_v > 2**h**, slender beam*

Fig. RB.1.2—Description of deep and slender beams.

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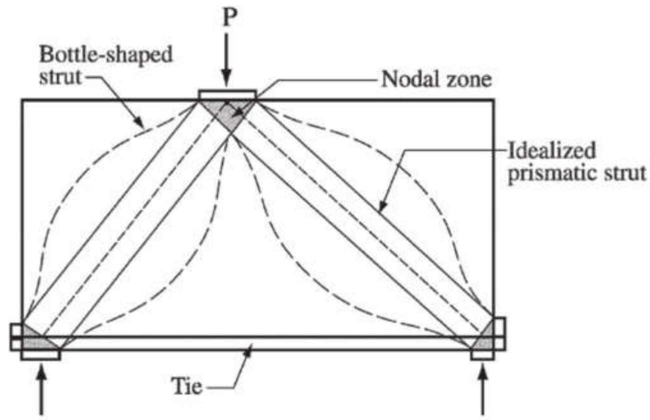
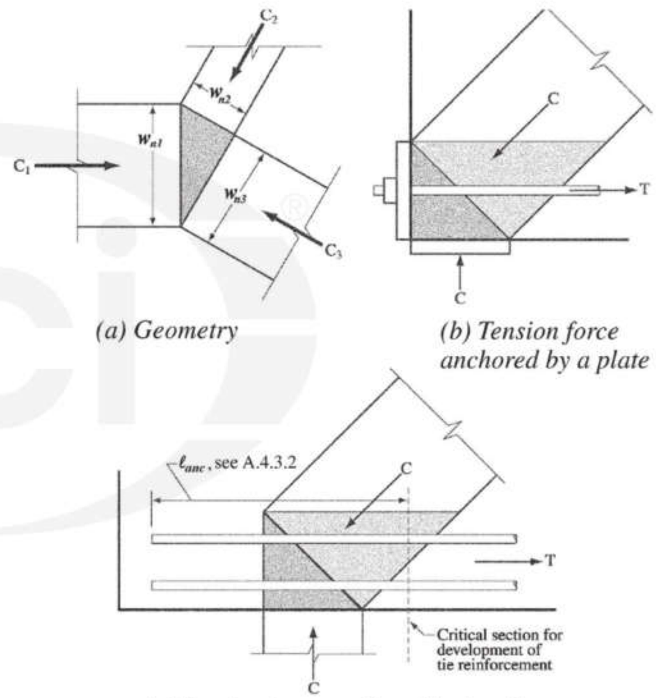


Fig. RB.1.3—Description of strut-and-tie model.



(a) Geometry

(b) Tension force anchored by a plate

(c) Tension force anchored by bond

Fig. RB.1.4—Hydrostatic nodes.

nodal zone—the volume of concrete around a node that is assumed to transfer strut-and-tie forces through the node.

nodal zone—Historically, hydrostatic nodal zones as shown in Fig. RB.1.4 were used. These were largely superseded by what are called extended nodal zones, shown in Fig. RB.1.5.

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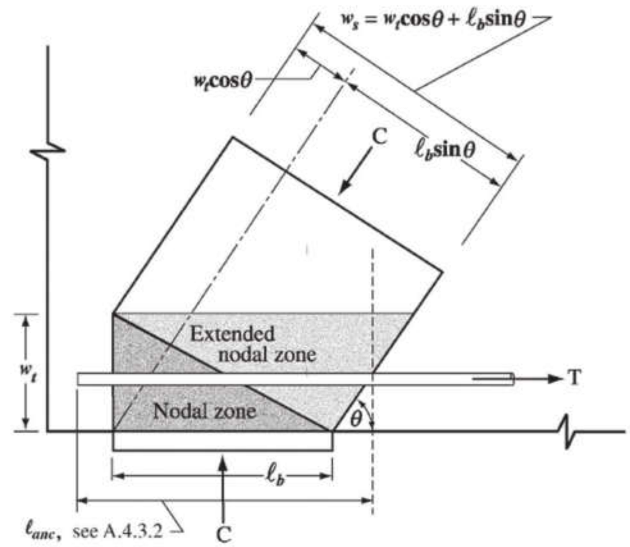
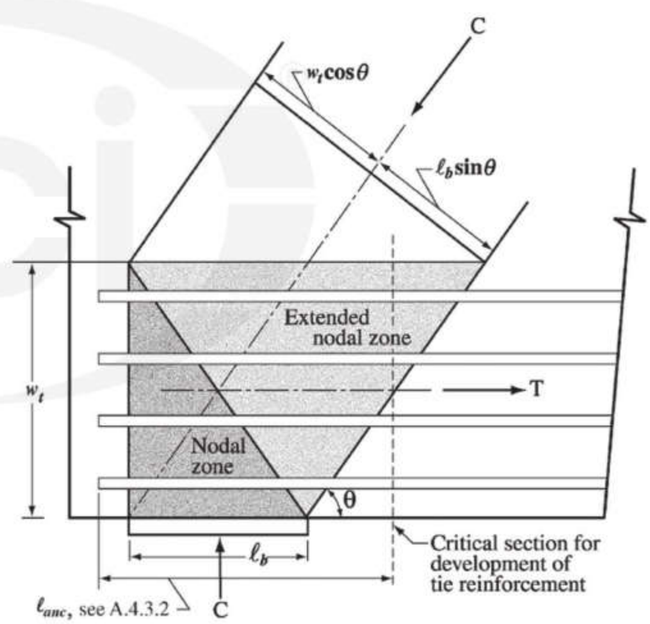


(a) One layer of steel



(b) Distributed steel

Fig. RB.1.5—Extended nodal zone showing the effect of the distribution of the force.

A **hydrostatic nodal zone** has loaded faces perpendicular to the axes of the struts and ties acting on the node and has equal stresses on the loaded faces. Figure RB.1.4(a) shows a **C-C-C** nodal zone. If the stresses on the face of the nodal zone are the same in all three struts, the ratios of the lengths of the sides of the nodal zone, w_{n1}:w_{n2}:w_{n3} are in the same proportions as the three forces C_1:C_2:C_3. The faces of a hydrostatic nodal zone are perpendicular to the axes of the struts and ties acting on the nodal zone.

These nodal zones are called hydrostatic nodal zones because the in-plane stresses are the same in all directions.

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Strictly speaking, this terminology is incorrect because the in-plane stresses are not equal to the out-of-plane stresses.

A **C-C-T** nodal zone can be represented as a hydrostatic nodal zone if the tie is assumed to extend through the node to be anchored by a plate on the far side of the node, as shown in Fig. RB.1.4(b), provided that the size of the plate results in bearing stresses that are equal to the stresses in the struts. The bearing plate on the left side of Fig. RB.1.4(b) is used to represent an actual tie anchorage. The tie force can be anchored by a plate, or through development of straight or hooked bars, as shown in Fig. RB.1.4(c).

The shaded areas in Fig. RB.1.5(a) and (b) are extended nodal zones. An **extended nodal zone** is that portion of a member bounded by the intersection of the effective strut width w_s and the effective tie width w_t (refer to RB.4.2).

In the nodal zone shown in Fig. RB.1.6(a), the reaction R equilibrates the vertical components of the forces C_1 and C_2. Frequently, calculations are easier if the reaction R is divided into R_1, which equilibrates the vertical component of C_1 and R_2, which equilibrates the vertical component of the force C_2, as shown in Fig. RB1.6(b).

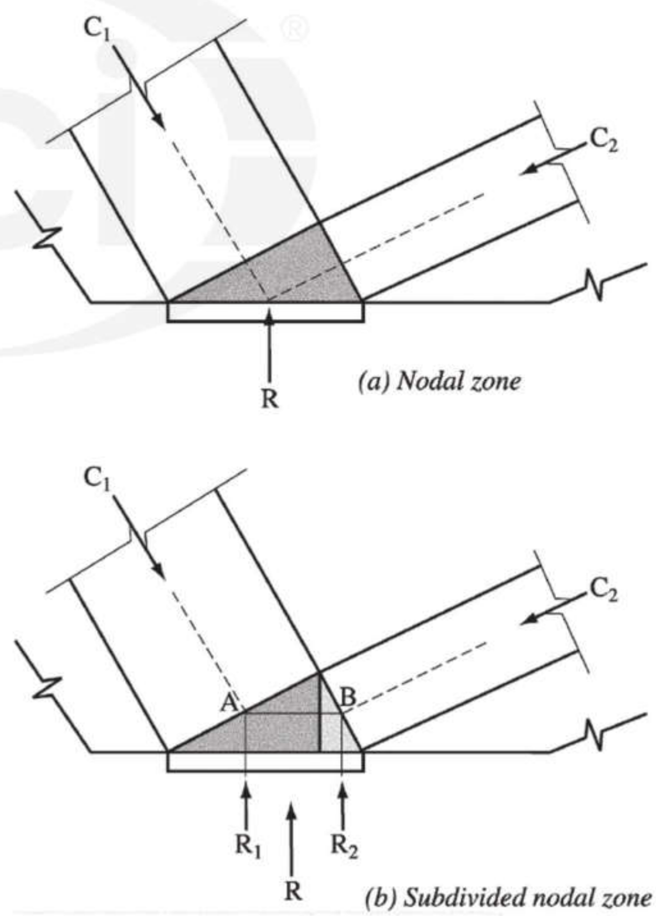

Fig. RB.1.6—Subdivision of nodal zone.

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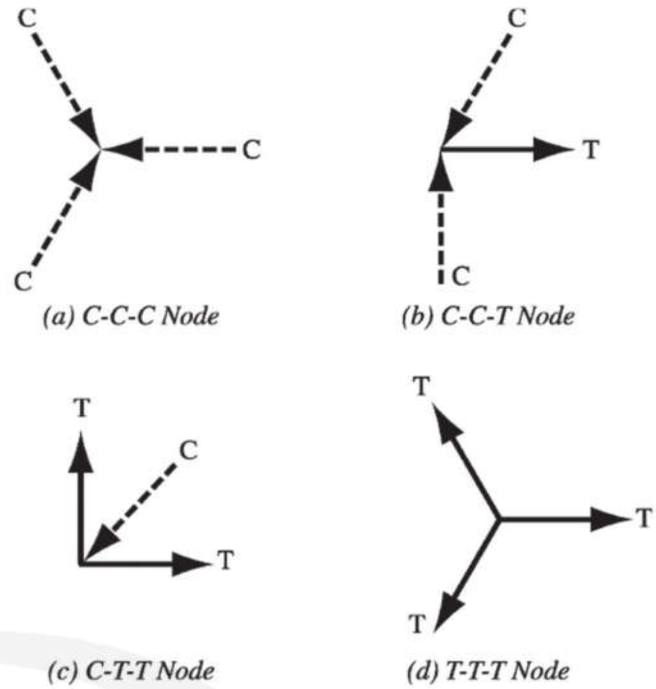

Fig. RB.1.7—Classification of nodes.

node—the point in a joint in a strut-and-tie model where the axes of the struts, ties, and concentrated forces acting on the joint intersect.

strut—a compression member in a strut-and-tie model. A strut represents the resultant of a parallel or a fan-shaped compression field.

bottle-shaped strut—a strut that is wider at mid-length than at its ends.

node—For equilibrium, at least three forces should act on a node in a strut-and-tie model, as shown in Fig. RB.1.7. Nodes are classified according to the signs of these forces. A **C-C-C** node resists three compressive forces, a **C-C-T** node resists two compressive forces and one tensile force, and so on.

strut—In design, struts are usually idealized as prismatic compression members, as shown by the straight-line outlines of the struts in Fig. RB.1.2 and RB.1.3. If the effective compression strength f_{ce} differs at the two ends of a strut, due either to different nodal zone strengths at the two ends, or to different bearing lengths, the strut is idealized as a uniformly tapered compression member.

bottle-shaped struts—A bottle-shaped strut is a strut located in a part of a member where the width of the compressed concrete at midlength of the strut can spread laterally (Schlaich et al. 1987; MacGregor 1997). The curved dashed outlines of the struts in Fig. RB.1.3 and the curved solid outlines in Fig. RB.1.8 approximate the boundaries of bottle-shaped struts. A split cylinder test is an example of a bottle-shaped strut. The internal lateral spread of the applied compression force in such a test leads to a transverse tension that splits the specimen.

To simplify design, bottle-shaped struts are idealized either as prismatic or as uniformly tapered, and crack-control reinforcement from B.3.3 is provided to resist the transverse tension. The amount of confining transverse reinforcement can be computed using the strut-and-tie model shown in Fig. RB.1.8(b) with the struts that represent the spread of the compression force acting at a slope of 1:2 to the axis of the applied compressive force. Alternatively, for f_c' not exceeding 6000 psi, Eq. (B-4a) and (B-4b) can be used. The

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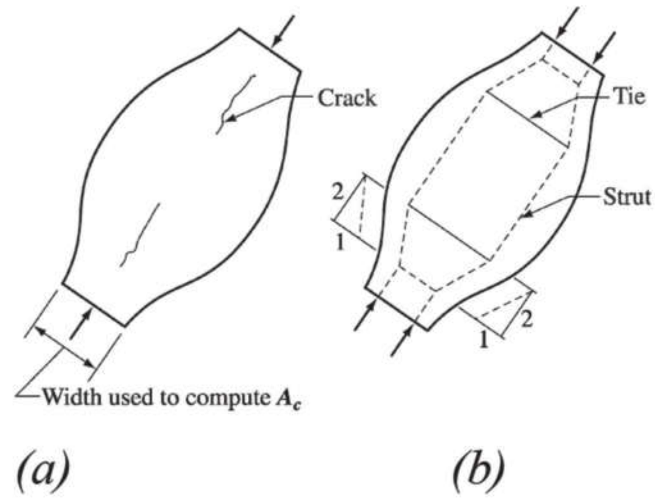

Fig. RB.1.8—Bottle-shaped strut: (a) cracking of a bottle-shaped strut; and (b) strut-and-tie model of a bottle-shaped strut.

cross-sectional area A_c of a bottle-shaped strut is taken as the smaller of the cross-sectional areas at the two ends of the strut; refer to Fig. RB.1.8(a).

strut-and-tie model—a truss model of a structural member, or of a D-region in such a member, made up of struts and ties connected at nodes, capable of transferring the factored loads to the supports or to adjacent B-regions.

strut-and-tie model—The components of a strut-and-tie model of a single-span deep beam loaded with a concentrated load are identified in Fig. RB.1.3. The cross-sectional dimensions of a strut or tie are designated as thickness and width, both perpendicular to the axis of the strut or tie. Thickness is perpendicular to the plane of the truss model, and width is in the plane of the truss model.

tie—a tension member in a strut-and-tie model.

tie—A tie consists of reinforcement or prestressing steel plus a portion of the surrounding concrete that is concentric with the axis of the tie. The surrounding concrete is included to define the zone in which the forces in the struts and ties are to be anchored. The concrete in a tie is not used to resist the axial force in the tie. Although not considered in design, the surrounding concrete will reduce the elongations of the tie, especially at service loads.

B.2—Strut-and-tie model design procedure

B.2.1 It shall be permitted to design structural concrete members, or D-regions in such members, by modeling the member or region as an idealized truss. The truss model shall contain struts, ties, and nodes as defined in B.1. The truss model shall be capable of transferring all factored loads to the supports or adjacent B-regions.

RB.2—Strut-and-tie model design procedure

RB.2.1 The truss model described in B.2.1 is referred to as a strut-and-tie model. Details of the use of strut-and-tie models are given in Schlaich et al. (1987), Collins and Mitchell (1991), MacGregor (1997), FIP Commission 3 (1999), Menn (1990), Muttoni et al. (1997), and ACI 445R. The design of a D-region includes the following four steps:

1. Define and isolate each D-region.
2. Compute resultant forces on each D-region boundary.
3. Select a truss model to transfer the resultant forces across the D-region. The axes of the struts and ties, respectively, are chosen to approximately coincide with the axes of the compression and tension fields. The forces in the struts and ties are computed.
4. The effective widths of the struts and nodal zones are determined considering the forces from Step 3 and the effective concrete strengths defined in B.3.2 and B.5.2, and

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reinforcement is provided for the ties considering the steel strengths defined in B.4.1. The reinforcement should be anchored in the nodal zones.

Strut-and-tie models represent strength limit states and code requirements for serviceability should also be satisfied. Deflections of deep beams or similar members can be estimated using an elastic analysis to analyze the strut-and-tie model. Crack widths in a tie shall be considered acceptable for environmental engineering structures if the service level stress in the tie reinforcement does not exceed limits specified in 9.2.6.2 and 9.2.6.3 for normal and severe exposures. In addition, the crack widths in a tie can be checked using 10.6.4, assuming the tie is encased in a prism of concrete corresponding to the area of tie from RB.4.2.

B.2.2 The strut-and-tie model shall be in equilibrium with the applied loads and the reactions.

B.2.3 In determining the geometry of the truss, the dimensions of the struts, ties, and nodal zones shall be taken into account.

RB.2.3 The struts, ties, and nodal zones making up the strut-and-tie model all have finite widths that should be taken into account in selecting the dimensions of the truss. Figure RB.2.3(a) shows a node and the corresponding nodal zone. The vertical and horizontal forces equilibrate the force in the inclined strut. If the stresses are equal in all three struts, a hydrostatic nodal zone can be used, and the widths of the struts will be in proportion to the forces in the struts.

If more than three forces act on a nodal zone in a two-dimensional structure, as shown in Fig. RB.2.3(b), it is generally necessary to resolve some of the forces to end up with three intersecting forces. The strut forces acting on Faces A–E and C–E in Fig. RB.2.3(b) can be replaced with one force acting on face A–C. This force passes through the node at D.

Alternatively, the strut-and-tie model could be analyzed assuming all the strut forces acted through the node at D, as shown in Fig. RB.2.3(c). In this case, the forces in the

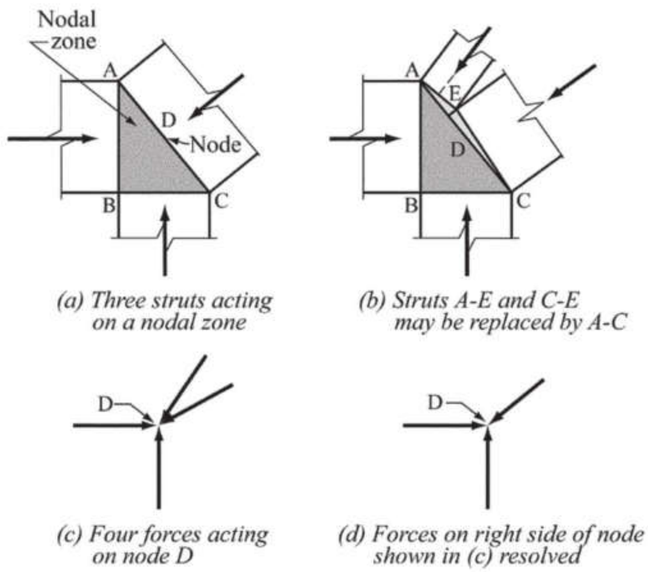

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Fig. RB.2.3—Resolution of forces on a nodal zone.

two struts on the right side of Node D can be resolved into a single force acting through Point D, as shown in Fig. RB.2.3(d).

If the width of the support in the direction perpendicular to the member is less than the width of the member, transverse reinforcement may be required to restrain vertical splitting in the plane of the node. This can be modeled using a transverse strut-and-tie model.

B.2.4 Ties shall be permitted to cross struts. Struts shall cross or overlap only at nodes.

B.2.5 The angle θ between the axes of any strut and any tie entering a single node shall not be taken as less than 25 degrees.

RB.2.5 The angle between the axes of struts and ties acting on a node should be large enough to mitigate cracking and to avoid incompatibilities due to shortening of the struts and lengthening of the ties occurring in almost the same directions. This limitation on the angle prevents modeling the shear spans in slender beams using struts inclined at less than 25 degrees from the longitudinal steel. Refer to Menn (1990).

B.2.6 Design of struts, ties, and nodal zones shall be based on

$$\phi F_n \geq F_u \qquad \text{(B-1)}$$

where F_u is the factored force acting in a strut, in a tie, or on one face of a nodal zone; F_n is the nominal strength of the strut, tie, or nodal zone; and ϕ is specified in 9.3.2.6.

RB.2.6 Factored loads are applied to the strut-and-tie model, and the forces in all the struts, ties, and nodal zones are computed. If several loading cases exist, each should be investigated. The strut-and-tie model, or models, is analyzed for the loading cases and, for a given strut, tie, or nodal zone, F_u is the largest force in that element for all loading cases.

B.3—Strength of struts

B.3.1 The nominal compressive strength of a strut without longitudinal reinforcement, F_{ns}, shall be taken as the smaller value of

$$F_{ns} = f_{ce}A_{cs} \qquad \text{(B-2)}$$

at the two ends of the strut, where A_{cs} is the cross-sectional area at one end of the strut, and f_{ce} is the smaller of (a) and (b):

(a) The effective compressive strength of the concrete in the strut given in B.3.2

(b) The effective compressive strength of the concrete in the nodal zone given in B.5.2

RB.3—Strength of struts

RB.3.1 The width of strut, w_s, used to compute A_{cs} is the smaller dimension perpendicular to the axis of the strut at the ends of the strut. This strut width is illustrated in Fig. RB.1.4(a) and Fig. RB.1.5(a) and (b). In two-dimensional structures, such as deep beams, the thickness of the struts may be taken as the width of the member.

B.3.2 The effective compressive strength of the concrete, f_{ce}, in a strut shall be taken as

$$f_{ce} = 0.85\beta_s f_c' \qquad \text{(B-3)}$$

RB.3.2 The strength coefficient $\mathbf{0.085}f_c'$ in Eq. (B-3), represents the effective concrete strength under sustained compression, similar to that used in Eq. (10-1) and (10-2).

B.3.2.1 For a strut of uniform cross-sectional area over its length: $\beta_s = 1.00$

RB.3.2.1 The value of β_s in B.3.2.1 applies to a strut equivalent to the rectangular stress block in a compression zone in a beam or column.

B.3.2.2 For bottle-shaped struts with reinforcement satisfying B.3.3: $\beta_s = 0.75$

RB.3.2.2 The value of β_s in B.3.2.2 applies to bottle-shaped struts as shown in Fig. RB.1.3. The internal lateral

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B.3.2.3 For struts in tension members, or the tension flanges of members: β_s **= 0.40**

B.3.2.4 For all other cases, β_s **= 0.60λ**, where the value of **λ** is defined in 8.6.1.

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spread of the compression forces can lead to splitting parallel to the axis of the strut near the ends of the strut, as shown in Fig. RB.1.8. Reinforcement placed to resist the splitting force restrains crack width, allows the strut to resist more axial load, and permits some redistribution of force.

RB.3.2.3 The value of β_s in B.3.2.3 applies, for example, to compression struts in a strut-and-tie model used to design the longitudinal and transverse reinforcement of the tension flanges of beams, box girders, and walls. The low value of β_s reflects that these struts need to transfer compression across cracks in a tension zone.

RB.3.2.4 The value of β_s in B.3.2.4 applies to strut applications not included in B.3.2.1, B.3.2.2, and B.3.2.3. Examples are struts in a beam web compression field in the web of a beam where parallel diagonal cracks are likely to divide the web into inclined struts, and struts are likely to be crossed by cracks at an angle to the struts (refer to Fig. RB.3.2(a) and (b)). Section B.3.2.4 gives a reasonable lower limit on β_s except for struts described in B.3.2.3.

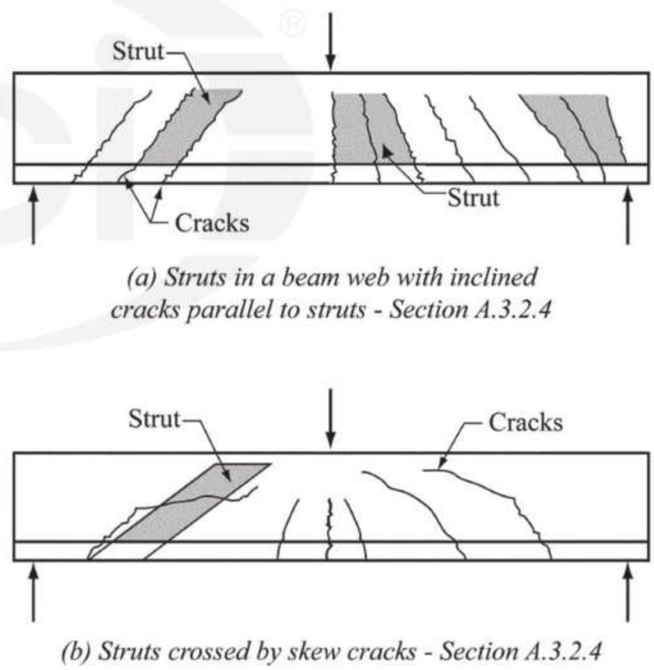

Fig. RB.3.2—Type of struts.

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B.3.3 If the value of β_s specified in B.3.2.2 is used, the axis of the strut shall be crossed by reinforcement proportioned to resist the transverse tensile force resulting from the compression force spreading in the strut. It shall be permitted to assume the compressive force in the strut spreads at a slope of 2 longitudinal to 1 transverse to the axis of the strut.

COMMENTARY

RB.3.3 The reinforcement required by B.3.3 is related to the tension force in the concrete due to the spreading of the strut, as shown in the strut-and-tie model in Fig. RB.1.8(b). Section RB.3.3 allows the use of local strut-and-tie models to compute the amount of transverse reinforcement needed in a given strut. The compressive forces in the strut may be assumed to spread at a 2:1 slope, as shown in Fig. RB.1.8(b). For specified concrete compressive strengths not exceeding 6000 psi, the amount of reinforcement required by Eq. (B-4a) and (B-4b) is deemed to satisfy B.3.3.

Figure RB.3.3 shows two layers of reinforcement crossing a cracked strut. If the crack opens without shear slip along the crack, bars in layer $\boldsymbol{i}$ in the figure will cause a stress perpendicular to the strut of

$$\frac{A_{si} f_{si}}{b_s s_i} \sin \alpha_i$$

where the subscript $\boldsymbol{i}$ takes on the values of 1 and 2 for the vertical and horizontal bars, respectively, as shown in Fig. RB.3.3. Equation (B-4a) is written in terms of a reinforcement ratio rather than a stress to simplify the calculation.

The reinforcement ratio of 0.0042 in Eq. (B-4a) represents the square root of the sum of the squares of minimum vertical and horizontal reinforcement ratios of 0.003. This is consistent with the crack control reinforcement requirements of AASHTO LRFD but is higher than the requirements of CSA A23.3 (0.002 vertical and horizontal with the resultant reinforcement ratio of 0.0028), or Appendix A of ACI 318-11. The second requirement in Eq. (B-4) relates to the shear reinforcement required to maintain equilibrium of the bottle-shaped strut after the formation of the splitting crack. Cracking width within the bottle-shaped strut is controlled with the introduction of environmental durability factor S_d. Permissible tensile stresses f_s for shear reinforce-

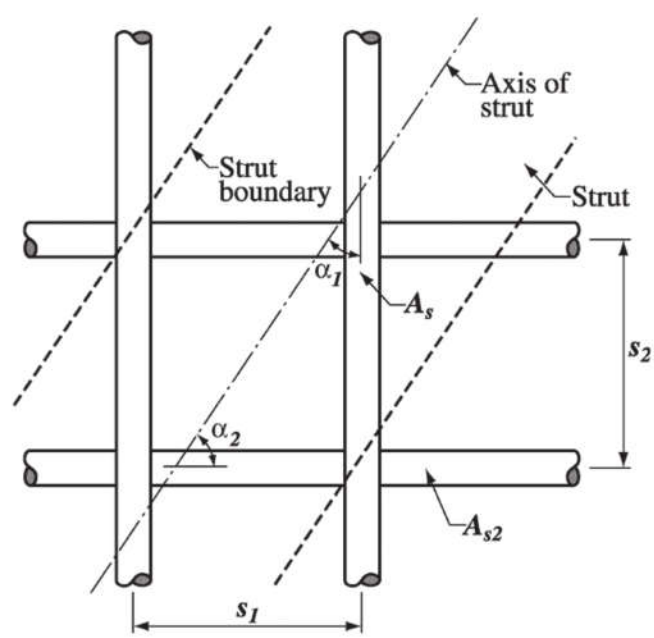

Fig. RB.3.3—Reinforcement crossing a strut.

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ment are defined in 9.2.6.4 and 9.2.6.5 for normal and severe exposure.

Often, the confinement reinforcement given in B.3.3 is difficult to place in three-dimensional structures such as pile caps. If this reinforcement is not provided, the value of f_{ce} given in B.3.2.3 is used.

B.3.3.1 For f_c' not greater than 6000 psi, the requirement of B.3.3 shall be permitted to be satisfied by the axis of the strut being crossed by layers of reinforcement that satisfy Eq. (B-4):

$$\Sigma \frac{A_{si}}{b_s s_i} \sin \alpha_i \geq 0.0042 \quad \text{(B-4a)}$$

and

$$\Sigma \frac{A_{si}}{b_s s_i} \sin \alpha_i \geq \frac{F_{strut} S_d}{2\phi f_y b_s \ell_s} \quad \text{(B-4b)}$$

where A_{si} is the total area of surface reinforcement at spacing s_i in the i-th layer of reinforcement crossing a strut at an angle α_i to the axis of the strut; b_s is the width of the strut perpendicular to the plane of surface reinforcement; F_{strut} is factored strut force; and ℓ_s is the length of strut between nodes. Values for f_s in Eq. (9-8) for S_d shall be taken as 24,000 psi for normal exposure and 20,000 psi for severe exposure.

B.3.3.2 The reinforcement required in B.3.3 shall be placed in either two orthogonal directions at angles α_1 and α_2 to the axis of the strut, or in one direction at an angle α to the axis of the strut. If the reinforcement is in only one direction, α shall not be less than 40 degrees.

RB.3.3.2 In a corbel with a shear span-to-depth ratio less than 1.0, the confinement reinforcement required to satisfy B.3.3 is usually provided in the form of horizontal stirrups crossing the inclined compression strut, as shown in Fig. R11.8.2.

B.3.4 If documented by tests and analyses, it shall be permitted to use an increased effective compressive strength of a strut due to confining reinforcement.

RB.3.4 The design of tendon anchorage zones for prestressed concrete sometimes uses confinement to enhance the compressive strength of the struts in the local zone. Confinement of struts is discussed in FIP Commission 3 (1999) and Bergmeister et al. (1991).

B.3.5 The use of compression reinforcement shall be permitted to increase the strength of a strut. Compression reinforcement shall be properly anchored, parallel to the axis of the strut, located within the strut, and enclosed in ties or spirals satisfying 12.10.1. In such cases, the nominal strength of a longitudinally reinforced strut is:

$$F_{ns} = f_{ce} A_{cs} + A_s' f_s' \quad \text{(B-5)}$$

RB.3.5 The strength added by the reinforcement is given by the last term in Eq. (B-5). The stress f_s' in the reinforcement in a strut at nominal strength can be obtained from the strains in the strut when the strut crushes. For Grade 40 or 60 reinforcement, f_s' can be taken as f_y.

B.4—Strength of ties

B.4.1 The nominal strength of a tie, F_{nt}, shall be taken as

$$F_{nt} = A_{ts} f_y / S_d + A_{tp}(f_{se} + \Delta f_p) \quad \text{(B-6)}$$

RB.4—Strength of ties

RB.4.1 Environmental durability factor S_d is used only for nonprestressed reinforcement. Permissible stresses f_s for the tie reinforcement are based on the hoop tensile stresses defined in 9.2.6.4 and 9.2.6.5 for normal and severe exposure.

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where environmental durability factor S_d is defined in Eq. (9-8); $(f_{se} + \Delta f_p)$ shall not exceed f_{py}, and A_{tp} is zero for nonprestressed members. Values for f_s in Eq. (9-8) shall be taken as 20,000 psi for normal exposure and 17,000 psi for severe exposure.

In Eq. (B–6), it shall be permitted to take Δf_p equal to 60,000 psi for bonded prestressed reinforcement, or 10,000 psi for unbonded prestressed reinforcement. Other values of Δf_p shall be permitted when justified by analysis.

B.4.2 The axis of the reinforcement in a tie shall coincide with the axis of the tie in the strut-and-tie model.

B.4.3 Tie reinforcement shall be anchored by mechanical devices, post-tensioning anchorage devices, standard hooks, or straight bar development as required by B.4.3.1 through B.4.3.4.

B.4.3.1 Nodal zones shall develop the difference between the tie force on one side of the node and the tie force on the other side.

B.4.3.2 At nodal zones anchoring one tie, the tie force shall be developed at the point where the centroid of the reinforcement in a tie leaves the extended nodal zone and enters the span.

B.4.3.3 At nodal zones anchoring two or more ties, the tie force in each direction shall be developed at the point where the centroid of the reinforcement in the tie leaves the extended nodal zone.

B.4.3.4 The transverse reinforcement required by B.3.3 shall be anchored in accordance with Section 12.8.13.

COMMENTARY

RB.4.2 The effective tie width assumed in design, w_t, can vary between the following limits, depending on the distribution of the tie reinforcement:

(a) If the bars in the tie are in one layer, the effective tie width can be taken as the diameter of the bars in the tie plus twice the cover to the surface of the bars, as shown in Fig. RB.1.5(a)

(b) A practical upper limit of the tie width can be taken as the width corresponding to the width in a hydrostatic nodal zone, calculated as

$$w_{t,max} = F_{nt}/(f_{ce}b_s)$$

where f_{ce} is computed for the nodal zone in accordance with B.5.2. If the tie width exceeds the value from (a), the tie reinforcement should be distributed approximately uniformly over the width and thickness of the tie, as shown in Fig. RB.1.5(b).

RB.4.3 Anchorage of ties often requires special attention in nodal zones of corbels or in nodal zones adjacent to exterior supports of deep beams. The reinforcement in a tie should be anchored before it leaves the extended nodal zone at the point defined by the intersection of the centroid of the bars in the tie and the extensions of the outlines of either the strut or the bearing area. This length is ℓ_{anc}. In Fig. RB.1.5(a) and (b), this occurs where the outline of the extended nodal zone is crossed by the centroid of the reinforcement in the tie. Some of the anchorage may be achieved by extending the reinforcement through the nodal zone, as shown in Fig. RB.1.4(c), and developing it beyond the nodal zone. If the tie is anchored using 90-degree hooks, the hooks should be confined within the reinforcement extending into the beam from the supporting member to avoid cracking along the outside of the hooks in the support region.

In deep beams, hairpin bars spliced with the tie reinforcement can be used to anchor the tension tie forces at exterior supports, provided the beam width is large enough to accommodate such bars.

Figure RB.4.3 shows two ties anchored at a nodal zone. Development is required where the centroid of the tie crosses the outline of the extended nodal zone.

The development length of the tie reinforcement can be reduced through hooks, mechanical devices, additional confinement, or by splicing it with several layers of smaller bars.

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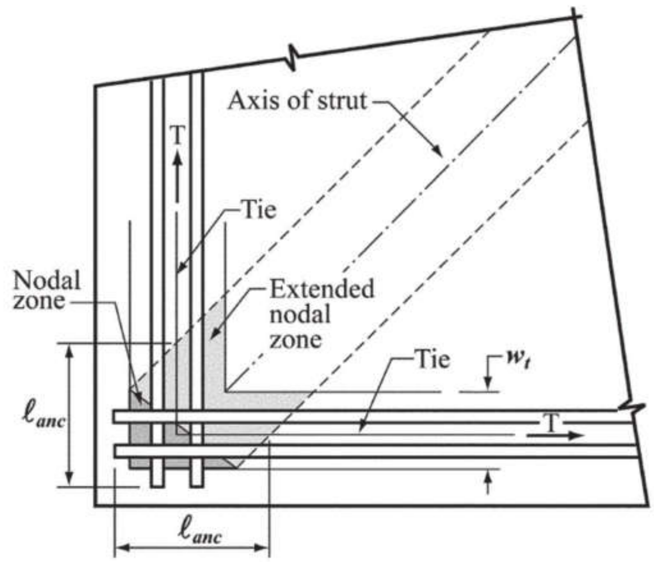

Fig. RB.4.3—Extended nodal zone anchoring two ties.

B.5—Strength of nodal zones

B.5.1 The nominal compression strength of a nodal zone, F_{nn}, shall be

$$F_{nn} = f_{ce}A_{nz} \tag{B-7}$$

where f_{ce} is the effective compressive strength of the concrete in the nodal zone as given in B.5.2 and A_{nz} is the smaller of (a) and (b):

(a) The area of the face of the nodal zone on which F_u acts, taken perpendicular to the line of action of F_u

(b) The area of a section through the nodal zone, taken perpendicular to the line of action of the resultant force on the section

RB.5—Strength of nodal zones

RB.5.1 If the stresses in all the struts meeting at a node are equal, a hydrostatic nodal zone can be used. The faces of such a nodal zone are perpendicular to the axes of the struts, and the widths of the faces of the nodal zone are proportional to the forces in the struts.

Assuming the principal stresses in the struts and ties act parallel to the axes of the struts and ties, the stresses on faces perpendicular to these axes are principal stresses, and B5.1(a) is used. If, as shown in Fig. RB.1.5(b), the face of a nodal zone is not perpendicular to the axis of the strut, there will be both shear stresses and normal stresses on the face of the nodal zone. Typically, these stresses are replaced by the normal (principal compression) stress acting on the cross-sectional area A_c of the strut, taken perpendicular to the axis of the strut as given in B.5.1(a).

In some cases, B.5.1(b) requires that the stresses be checked on a section through a subdivided nodal zone. The stresses are checked on the least area section, which is perpendicular to a resultant force in the nodal zone. In Fig. RB.1.6(b), the vertical face which divide the nodal zone into two parts is stressed by the resultant force acting along A-B. The design of the nodal zone is governed by the critical section from B.5.1(a) or B.5.1(b), whichever gives the highest stress.

B.5.2 Unless confining reinforcement is provided within the nodal zone and its effect is supported by tests and analysis, the calculated effective compressive stress f_{ce} on a face of a nodal zone due to the strut-and-tie forces shall not exceed the value given by:

$$f_{ce} = 0.85\beta_n f_c' \tag{B-8}$$

where the value of β_n is given in B.5.2.1 through B.5.2.3.

RB.5.2 The nodes in two-dimensional members, such as deep beams, can be classified as **C-C-C** if all the members intersecting at the node are in compression; as **C-C-T** nodes if one of the members acting on the node is in tension; and so on, as shown in Fig. RB.1.7. The effective compressive strength of the nodal zone is given by Eq. (B-8), as modified by B.5.2.1 through B.5.2.3 apply to **C-C-C** nodes, **C-C-T** nodes, and **C-T-T** or **T-T-T** nodes, respectively.

The β_n values reflect the increasing degree of disruption of the nodal zones due to the incompatibility of tension strains in the ties and compression strains in the struts. The stress

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B.5.2.1 In nodal zones bounded by struts or bearing areas, or both: $\boldsymbol{\beta_n} = \mathbf{1.00}$

B.5.2.2 In nodal zones anchoring one tie: $\boldsymbol{\beta_n} = \mathbf{0.80}$ or

B.5.2.3 In nodal zones anchoring two or more ties: $\boldsymbol{\beta_n} = \mathbf{0.60}$.

B.5.3 In a three-dimensional strut-and-tie model, the area of each face of a nodal zone shall not be less than that given in B.5.1, and the shape of each face of the nodal zones shall be similar to the shape of the projection of the end of the struts onto the corresponding faces of the nodal zones.

COMMENTARY

on any face of the nodal zone or on any section through the nodal zone should not exceed the value given by Eq. (B-8), as modified by B.5.2.1 through B.5.2.3.

RB.5.3 This description of the shape and orientation of the faces of the nodal zones is introduced to simplify the calculations of the geometry of a three-dimensional strut-and-tie model.

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APPENDIX C—ALTERNATIVE PROVISIONS FOR REINFORCED AND PRESTRESSED CONCRETE FLEXURAL AND COMPRESSION MEMBERS

C.1—Scope

Design for flexure and axial load by provisions of Appendix C shall be permitted. When Appendix C is used in design, C.8.4, C.8.4.1, C.8.4.2, and C.8.4.3 shall replace the corresponding numbered sections in Chapter 8; C.10.3.3 shall replace 10.3.3, 10.3.4, and 10.3.5, except 10.3.5.1 shall remain; C.19.1.3, C.19.8.1, C.19.8.2, and C.19.8.3 shall replace the corresponding numbered sections in Chapter 19; C.19.10.4, C.19.10.4.1, C.19.10.4.2, and C.19.10.4.3 shall replace 19.10.4, 19.10.4.1, and 19.10.4.2. 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.

C.8.4—Redistribution of moments in continuous nonprestressed flexural members

For criteria on moment redistribution for prestressed concrete members, refer to C.19.10.4.

C.8.4.1 Except where approximate values for moments are used, it shall be permitted to decrease factored moments calculated by elastic theory at sections of maximum negative or maximum positive moment and in any span of continuous flexural members for any assumed loading arrangement by not more than

$$20\left(1-\frac{\rho-\rho'}{\rho_b}\right) \text{ percent}$$

C.8.4.2 Redistribution of moments shall be made only when the section at which moment is reduced is so designed that $\boldsymbol{\rho}$ or $\boldsymbol{\rho - \rho'}$ is not greater than $\mathbf{0.50}\boldsymbol{\rho}_b$, where

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{87,000}{87,000+f_y}\right) \quad \text{(C-1)}$$

C.8.4.3 The reduced moment shall be used for calculating redistributed moments at all other sections within the spans. Static equilibrium shall be maintained after redistribution of moments for each loading arrangement.

COMMENTARY

APPENDIX RC—ALTERNATIVE PROVISIONS FOR REINFORCED AND PRESTRESSED CONCRETE FLEXURAL AND COMPRESSION MEMBERS

RC.1—Scope

Reinforcement limits, strength reduction factors ϕ, and moment redistribution in Appendix C differ from those in the main body of the Code. Appendix C contains the reinforcement limits, strength reduction factors ϕ, and moment redistribution used in the Code for many years. Designs using the provisions of Appendix C 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 D are applicable.

RC.8.4—Redistribution of moments in continuous nonprestressed flexural members

Moment redistribution is dependent on adequate ductility in plastic hinge regions. These plastic hinge regions develop at sections of maximum positive or negative moment and cause a shift in the elastic moment diagram. The usual results are reduction in the values of maximum negative moments in the support regions and an increase in the values of positive moments between supports from those computed by elastic analysis. However, because negative moments are determined for one loading arrangement and positive moments for another (refer to 14.7.6 for an exception), economies in reinforcement can sometimes be realized by reducing maximum elastic positive moments and increasing negative moments, thus narrowing the envelope of maximum negative and positive moments at any section in the span. The plastic hinges permit the utilization of the full capacity of more cross sections of a flexural member at ultimate loads.

Editions of the ACI 318 building code prior to 2008 addressed moment redistribution by permitting an increase or decrease of factored negative moments above or below elastically calculated values, within specified limits. A decrease in negative moment capacity implies inelastic behavior in the negative moment area at the support. By increasing the negative moment capacity, the moments at the positive sections can be reduced but the result is that inelastic behavior will occur in the positive moment region and the percentage change in the positive moment section could be much large than the 20 percent permitted for negative moment sections (Bondy 2003). The current Code places the same percentage limitations on both the positive and negative moment sections.

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 (refer to Fig. RC.8.4). Studies by Cohn (1965) and

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COMMENTARY

Mattock (1959) 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. 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 A, nor may it be used for slab systems designed by the direct design method (refer to 14.6.1.7).

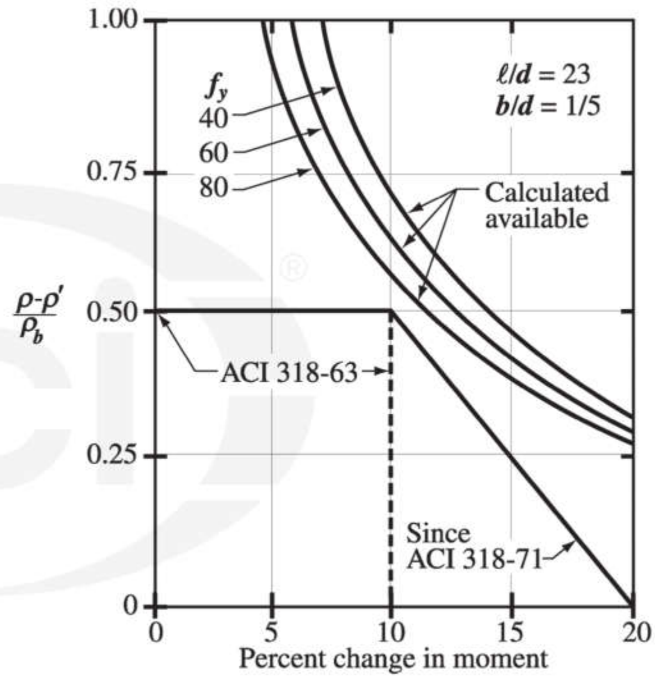

Fig. RC8.4—Permissible moment redistribution for minimum rotation capacity.

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C.10.3—General principles and requirements

C.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.

C.19.1—Scope

C.19.1.3 The following provisions of this code shall not apply to prestressed concrete, except as specifically noted: Sections 12.6.5, C.8.4, 8.12.2, 8.12.3, 8.12.4, 8.13, C.10.3.3, 10.5, 10.6, 10.9.1, and 10.9.2; Chapter 14; and Sections 15.4, 15.6, and 15.7.1.

COMMENTARY

RC.10.3—General principles and requirements

RC.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 C.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.

RC.19.1—Scope

RC.19.1.3 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 12.6.5: Section 12.6.5 of the Code is excluded from application to prestressed concrete because the requirements for bonded reinforcement and unbonded tendons for cast-in-place members are provided in 19.9 and 19.12, respectively.

Section C.8.4: Moment redistribution for prestressed concrete is provided in C.19.10.4.

Sections 8.12.2, 8.12.3, and 8.12.4: The empirical provisions of 8.12.2, 8.12.3, and 8.12.4 for T-beams were developed for nonprestressed reinforced concrete and, if applied to prestressed concrete, would exclude many standard

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prestressed products in satisfactory use today. Hence, proof by experience permits variations.

By excluding 8.12.2, 8.12.3, and 8.12.4, 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 licensed design professional. Where possible, the flange widths in 8.12.2, 8.12.3, and 8.12.4 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 8.12.2.

Sections 8.12.1 and 8.12.5 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 8.13: The empirical limits established for conventionally reinforced concrete joist floors are based on successful past performance of joist construction using standard joist forming systems. Refer to R8.13. For prestressed joist construction, experience and judgment should be used. The provisions of 8.13 may be used as a guide.

Sections C.10.3.3, 10.5, 10.9.1, and 10.9.2: For prestressed concrete, the limitations on reinforcement given in C.10.3.3, 10.5, 10.9.1, and 10.9.2 are replaced by those in C.19.8, C.19.9, and C.19.11.2.

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 14: 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 14. Because of these unique properties associated with prestressing, many of the design procedures of Chapter 14 are not appropriate for prestressed concrete structures and are replaced by the provisions of 19.12.

Sections 15.6 and 15.7.1: The requirements for wall design in 15.6 and 15.7.1 are largely empirical, utilizing considerations not intended to apply to prestressed concrete.

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C.19.8—Limits for reinforcement of flexural members

C.19.8.1 Ratio of prestressed and nonprestressed reinforcement used for computation of moment strength of a member, except as provided in C.19.8.2, 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 $0.36\beta_1$, except as permitted in C.19.8.2.

Ratio ω_p is computed as $\rho_b f_{ps}/f_c'$. Ratios ω_w and ω_{pw} are computed as ω and ω_p, respectively, except that when computing ρ and ρ_b, b_w shall be used in place of b and the area of reinforcement or prestressing steel required to develop the compressive strength of the web only shall be used in place of A_s or A_{ps}. Ratio ω_w' is computed as ω', except that when computing ρ', b_w shall be used in place of b.

C.19.8.2 When a reinforcement ratio that exceeds the limit specified in C.19.8.1 is provided, design moment strength shall not exceed the moment strength based on the compression portion of the moment couple.

C.19.8.3 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. This provision shall be permitted to be waived for:

(a) two-way, unbonded post-tensioned slabs; and

(b) flexural members with shear and flexural strength at least twice that required by 9.2.

C.19.10—Statically indeterminate structures

C.19.10.1 Frames and continuous construction of prestressed concrete shall be designed for satisfactory performance at service load conditions and for adequate strength.

C.19.10.2 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.

COMMENTARY

RC.19.8—Limits for reinforcement of flexural members

RC.19.8.1 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 block 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 C.19.8.1.

RC.19.8.2 Design moment strength of over reinforced members may be computed using strength equations similar to those for nonprestressed concrete members. ACI 318R-83 provided strength equations for rectangular and flanged sections.

RC.19.8.3 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.

Due to the very limited extent of initial cracking in the negative moment region near columns of two-way flat plates, deflection under load does not reflect any abrupt change in stiffness as the modulus of rupture of concrete is reached.

Only at load levels beyond the factored loads is the additional cracking extensive enough to cause an abrupt change in the deflection under load. Tests have shown that it is not possible to rupture (or even yield) unbonded post-tensioning tendons in two-way slabs before a punching shear failure (PTI 2004; Gerber and Burns 1971; Smith and Burns 1974; Burns and Hemakom 1977, 1985; Kosut et al. 1985). The use of unbonded tendons in combination with the minimum bonded reinforcement requirements of 19.9.3 and 19.9.5 has been shown to ensure post-cracking ductility and that a brittle failure mode will not develop at first cracking.

RC.19.10—Statically indeterminate structures

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C.19.10.3 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 C.19.10.4.

C.19.10.4 *Redistribution of moments in continuous prestressed flexural members*

C.19.10.4.1 Where bonded reinforcement is provided at supports in accordance with 19.9, negative or positive moments calculated by elastic theory for any assumed loading shall be permitted to be increased or decreased by not more than

$$20\left[1-\frac{\omega_p+\dfrac{d}{d_p}(\omega-\omega')}{0.36\beta_1}\right]\text{ percent}$$

C.19.10.4.2 Redistribution of moments shall be made only when the section at which moment is reduced is so designed that $\boldsymbol{\omega_p}$, $\boldsymbol{[\omega_p + (d/d_p)(\omega - \omega')]}$, or $\boldsymbol{[\omega_{pw} + (d/d_p)(\omega_w - \omega_w')]}$, whichever is applicable, is not greater than $\mathbf{0.24}\beta_1$.

C.19.10.4.3 The reduced moments shall be used for calculating redistributed moments at all other sections within the spans. Static equilibrium shall be maintained after redistribution of moments for each loading arrangement.

COMMENTARY

RC.19.10.3 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 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: 1) determine moments due to dead and live load; 2) modify by algebraic addition of secondary moments; and 3) 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.

RC.19.10.4 *Redistribution of 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. However, 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 safety 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 19.4. The choice of $\mathbf{0.24}\boldsymbol{\beta_1}$ as the largest tension reinforcement index, $\boldsymbol{\omega_p}$, $\boldsymbol{[\omega_p + (d/d_p)(\omega - \omega')]}$, or $\boldsymbol{[\omega_{pw} + (d/d_p)(\omega_w - \omega_w')]}$, for which redistribution of moments is allowed, is in agreement with the requirements for conventionally reinforced concrete of $\mathbf{0.5}\boldsymbol{\rho_b}$ stated in C.8.4.

The terms $\boldsymbol{\omega_p}$, $\boldsymbol{[\omega_p + (d/d_p)(\omega - \omega')]}$, or $\boldsymbol{[\omega_{pw} + (d/d_p)(\omega_w - \omega_w')]}$ that appear in C.19.10.4.1 and C.19.10.4.3 and are each equal to $\mathbf{0.85}\boldsymbol{a/d_p}$, where $\boldsymbol{a}$ is the depth of the equivalent rectangular stress block 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 C.19.10.4.1 and to check compliance with the limitation on flexural reinforcement contained in C.19.10.4.3.

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COMMENTARY

For the moment redistribution principles of C.19.10.4 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 19.9 will serve this purpose.

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APPENDIX D—ALTERNATIVE LOAD FACTORS, STRENGTH REDUCTION FACTORS, AND DISTRIBUTION OF FLEXURAL REINFORCEMENT

D.1—General

D.1.1 Structural concrete shall be permitted to be designed using the load factors, strength reduction factors, and distribution of flexural reinforcement of Appendix D. When Appendix D is used in design, Sections D.9.2 and D.9.3 shall replace Sections 9.2 and 9.3 in Chapter 9 of the Code and Section D.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.

D.1.2 It shall be permitted to use the provisions of Appendix in conjunction with the provisions of Appendix D.

D.9.2—Required strength

D.9.2.1 Required strength ***U*** to resist dead load ***D*** and live load ***L*** shall not be less than

$$U = 1.4D + 1.7L \tag{D-1}$$

D.9.2.2 For structures that resist ***W***, wind load ***U*** shall not be less than the larger of Eq. (D-1), (D-2) or (D-3)

$$U = 0.75(1.4D + 1.7L) + (1.6W) \tag{D-2}$$

$$U = 0.9D + 1.6W \tag{D-3}$$

The above load combinations shall include both full value and zero value of ***L*** to determine the more severe condition, but for any combination of ***D***, ***L***, and ***W***, required strength ***U*** shall not be less than Eq. (D-1). Where ***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. (D-2) and (D-3).

D.9.2.3 For structures that resist ***E***, the load effects of earthquake, ***U*** shall not be less than the larger of Eq. (D-1), (D-4), (D-5a), and (D-5b):

$$U = 0.75(1.4D + 1.7L + 1.7H + 1.7F) + 1.0E \tag{D-4}$$

$$U = 0.9D + 0.6H + 1.4F + 1.0E \tag{D-5a}$$

$$U = 0.9D + 1.4H + 1.0F + 1.0E \tag{D-5b}$$

In Eq. (D-4), where ***H*** or ***F*** reduce the effect of ***D***, ***L***, or each other, **0.8*H*** or **1.3*F*** shall be substituted for **1.7*H*** or **1.7*F***, as applicable, but ***U*** shall not be less than Eq. (D-1).

The above load combinations shall include both full value and zero value of ***L*** and ***F*** to determine the most severe condition.

COMMENTARY

APPENDIX RD—ALTERNATIVE LOAD FACTORS, STRENGTH REDUCTION FACTORS, AND DISTRIBUTION OF FLEXURAL REINFORCEMENT

RD.1—General

RD.1.1 In the ACI 350-06 Code, 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 D are considered reliable for concrete construction of environmental engineering concrete structures.

When this appendix is used, the corresponding commentary sections apply.

RD.9.2—Required strength

RD.9.2.2 The wind load equation in ASCE 7-98 and IBC 2000 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.

RD.9.2.3 The load ***E*** represents strength-level earthquake forces. 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 the previous load factor for earthquake loads, approximately 1.4, when service-level earthquake forces from earlier editions of such codes are used.

The load combinations in Eq. (D-5a) and (D-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, 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.

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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 $\boldsymbol{E}$ is based on service-level seismic forces, $\mathbf{1.4}\boldsymbol{E}$ shall be used in place of $\mathbf{1.0}\boldsymbol{E}$ in Eq. (D-4), (D-5a), and (D-5b).

D.9.2.4 For structures that resist $\boldsymbol{H}$, loads due to weight and pressure of soil, water in soil, or other related materials, $\boldsymbol{U}$ shall not be less than

$$U = 1.4D + 1.7L + 1.7H \qquad \text{(D-6)}$$

In Eq. (D-6), where $\boldsymbol{D}$ or $\boldsymbol{L}$ reduce the effect of $\boldsymbol{H}$, $\mathbf{0.9}\boldsymbol{D}$ shall be substituted for $\mathbf{1.4}\boldsymbol{D}$, and zero value of $\boldsymbol{L}$ shall be used to determine the greatest required strength $\boldsymbol{U}$. For any combination of $\boldsymbol{D}$, $\boldsymbol{L}$, and $\boldsymbol{H}$, required strength $\boldsymbol{U}$ shall not be less than Eq. (D-1).

D.9.2.5 For structures that resist $\boldsymbol{F}$, loads due to weight and pressure of fluids with well-defined densities, the load factor for $\boldsymbol{F}$ shall be 1.7 and $\boldsymbol{F}$ shall be added to all loading combinations, except as shown in D.9.2.3, such that the effect of $\boldsymbol{L}$ or $\boldsymbol{W}$ does not reduce the effect of $\boldsymbol{F}$.

D.9.2.6 If resistance to impact effects is taken into account in design, such effects shall be included with $\boldsymbol{L}$.

D.9.2.7 Where structural effects of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change $\boldsymbol{T}$ are significant, $\boldsymbol{U}$ shall not be less than the larger of Eq. (D-7) and (D-8):

$$U = 0.75(1.4D + 1.4T + 1.7L) \qquad \text{(D-7)}$$

$$U = 1.4(D + T) \qquad \text{(D-8)}$$

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.

COMMENTARY

RD.9.2.4 If effects $\boldsymbol{H}$ caused by earth pressure, groundwater 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 $\boldsymbol{D}$ or $\boldsymbol{L}$ reduce the effect of $\boldsymbol{H}$

$$U = 0.9D + 1.7H$$

but for any combination of $\boldsymbol{D}$, $\boldsymbol{L}$, or $\boldsymbol{H}$

$$U = 1.4D + 1.7L$$

RD.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 $\boldsymbol{D}$ or $\boldsymbol{L}$ reduce the effect of $\boldsymbol{F}$

$$U = 0.9D + 1.7F$$

but for any combination of $\boldsymbol{D}$, $\boldsymbol{L}$, or $\boldsymbol{F}$

$$U = 1.4D + 1.7L$$

RD.9.2.6 If the live load is applied rapidly, as may be the case for vehicle loads and cranes, impact effects should be considered. In all equations, substitute ($\boldsymbol{L}$ + impact) for $\boldsymbol{L}$ when impact must be considered.

RD.9.2.7 The effects of differential settlement, creep, shrinkage, temperature, and shrinkage-compensating concrete should be considered. 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 (D-8) is to prevent a design for load

$$U = 0.75(1.4D + 1.4T + 1.7L)$$

to approach

$$U = 1.05(D + T)$$

when live load is negligible.

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D.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.

D.9.2.9 Required strength $\boldsymbol{U}$ shall be multiplied by the following environmental durability factors $\boldsymbol{S_d}$ 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 A. For applicable use of the environmental durability factor $\boldsymbol{S_d}$ in conjunction with load combinations that include earthquake loads, refer to 13.1.1.10.

D.9.2.9.1 Flexural stress: $\boldsymbol{S_d = 1.3}$

D.9.2.9.2 Direct tensile stress (including hoop tension): $\boldsymbol{S_d = 1.65}$

D.9.2.9.3 Excess shear stress carried by shear reinforcement: $\boldsymbol{S_d = 1.3}$

D.9.3—Design strength

D.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 in D.9.3.2 and D.9.3.4.

D.9.3.2 Strength reduction factor ϕ shall be as follows:

D.9.3.2.1 Tension-controlled sections, as defined in 10.3.4 (see also D.9.3.2.6): 0.90

COMMENTARY

RD.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.

RD.9.3—Design strength

RD.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 (Winter 1979; ACI Committee 435 1974). 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.

RD.9.3.2.1 In applying D.9.3.2.1 and D.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.

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D.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 ε_t 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 ε_t increases from the compression-controlled strain limit to 0.005.

Alternatively, when Appendix C is used, for members in which f_y does not exceed 60,000 psi, with symmetric reinforcement, and with $(d - d')/h$ 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

RD.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. RD.9.3.2. The concept of net tensile strain ε_t is discussed in R10.3.3.

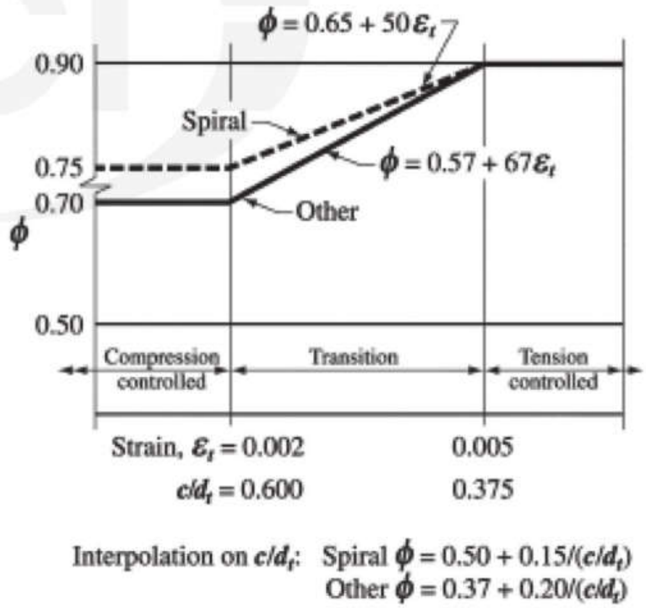

Fig. RD.9.3.2—Variation of ϕ with net tensile strain ε_t and c/d_t for Grade 60 reinforcement and for prestressing steel.

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COMMENTARY

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 distance from the extreme compression fiber to the neutral axis at nominal strength, and d_t is the distance from the extreme compression fiber to the centroid of the extreme layer of longitudinal 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 RD.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 ratio 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.

D.9.3.2.3 Shear and torsion: 0.85

D.9.3.2.4 Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models): 0.70

D.9.3.2.5 Post-tensioned anchorage zones: 0.85

RD.9.3.2.5 The ϕ-factor of 0.85 reflects the wide scatter of results of experimental anchorage zone studies. Because 19.13.4.2 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 $0.85 \times 0.7\lambda f_{ci}' \approx 0.6\lambda f_{ci}'$.

D.9.3.2.6 Flexure sections without axial load in pretensioned members where strand embedment is less than the development length as provided in 12.8.9.1.1: 0.85

RD.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.

D.9.3.3 Development lengths specified in Chapter 12 do not require a ϕ-factor.

D.9.3.4 For structures that rely on intermediate precast structural walls in Seismic Design Category D, E, or F, special moment frames, or special structural walls to resist E, ϕ shall be modified as given in (a) through (c):

(a) For any structural member that is designed to resist E, ϕ for shear shall be 0.60 if the nominal shear strength of the member 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 E

(b) For diaphragms, ϕ for shear shall not exceed the minimum ϕ for shear used for the vertical components of the primary lateral-force-resisting system

(c) For shear in joints and diagonally reinforced coupling beams, ϕ for shear shall be 0.85.

RD.9.3.4 Section D.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 D.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.

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D.10.6—Distribution of flexural reinforcement in beams and one-way slabs

D.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).

D.10.6.2 Distribution of flexural reinforcement in two-way slabs shall be as required by 14.3.

D.10.6.3 Flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross section as required by D.10.6.4.

D.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

$$z = f_s \sqrt[3]{d_c A} \qquad \text{(D-9)}$$

(for reinforcement located in one layer) does not exceed 115 kip/in. for normal environmental exposure and 95 kip/in. 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 flexural stress in reinforcement at service load f_s (kip/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

COMMENTARY

RD.10.6—Distribution of flexural reinforcement in beams and one-way slabs

RD.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 (Gergely and Lutz 1968; Kaar 1966; Base et al. 1966) 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.

RD.10.6.3 Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

RD.10.6.4 Equation (D-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 (Lloyd et al. 1969) have shown that the Gergely-Lutz expression applies reasonably to one-way slabs. The average ratio β_i is approximately 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.

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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.

COMMENTARY

The numerical limitations of z = **115 and 95 kip/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 A is as illustrated in Fig. RD.10.6.4.

For normal environmental exposure, deformed bars or wire should be spaced so that z does not exceed 115 kip/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 kip/in., and surface or other protection or barrier should be provided, suitable for the particular conditions of exposure. Provisions of D.10.6.4 in this Code are intended to provide liquid-tight environmental engineering concrete structures within the scope of this Code.

$$Z = f_s\sqrt[3]{d_c A} \quad \text{or} \quad s = 0.5(Z/f_s)^3/d_c^2$$

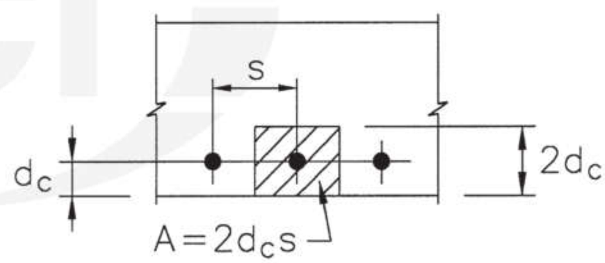

Fig. RD.10.6.4—Effective tension area of concrete.

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D.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 8.12, or a width equal to 1/10 the span, whichever is smaller. If the effective flange width exceeds 1/10 the span, longitudinal reinforcement shall be provided in the outer portions of the flange.

D.10.6.6 If the effective depth $\boldsymbol{d}$ 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 $\boldsymbol{d/2}$ nearest the flexural tension reinforcement. The area of skin reinforcement $\boldsymbol{A_{sk}}$ per foot of height on each side face shall be greater than $\mathbf{0.012(\boldsymbol{d} - 30)}$. The maximum spacing of the skin reinforcement shall not exceed the lesser of $\boldsymbol{d/6}$ 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

RD.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.

RD.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.

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APPENDIX E—ANCHORING TO CONCRETE

E.1—Definitions

adhesive—chemical components formulated from organic polymers, or a combination of organic polymers and inorganic materials that cure when blended together.

adhesive anchor—a post-installed anchor, inserted into hardened concrete with an anchor hole diameter not greater than 1.5 times the anchor diameter, that transfers loads to the concrete by bond between the anchor and the adhesive, and bond between the adhesive and the concrete.

anchor—a steel element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads to the concrete. Cast-in anchors include headed bolts, hooked bolts (J- or L-bolt), and headed studs. Post-installed anchors included expansion anchors, undercut anchors, and adhesive anchors. Steel elements for adhesive anchors include threaded rods, deformed reinforcing bars, or internally threaded steel sleeves with external deformations.

anchor group—numerous similar anchors having approximately equal effective embedment depths with spacing s between adjacent anchors such that the protected areas overlap. Refer to E.3.1.1.

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.

anchor reinforcement—reinforcement used to transfer the full design load from the anchors into the structural member. Refer to E.5.2.9 or E.6.2.9.

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.

COMMENTARY

APPENDIX RE—ANCHORING TO CONCRETE

RE.1—Definitions

adhesive—Organic polymers used in adhesives can include, but are not limited to, epoxies, polyurethanes, polyesters, methyl methacrylates, and vinyl esters.

adhesive anchor—The design model included in Appendix E for adhesive anchors is based on the behavior of anchors with hole diameters not exceeding 1.5 times the anchor diameter. Anchors with hole diameters exceeding 1.5 times the anchor diameter behave differently and are therefore excluded from the scope of Appendix E and ACI 355.4. To limit shrinkage and reduce displacement under load, most adhesive anchor systems require the annular gap to be as narrow as practical while still maintaining sufficient clearance for insertion of the anchor element in the adhesive-filled hole and ensuring complete coverage of the bonded area over the embedded length. The annular gap for reinforcing bars is generally larger than that for threaded rods. The required hole size is provided in the Manufacturer's Printed Installation Instructions (MPII).

anchor group—For all potential failure modes (steel, concrete breakout, pullout, side-face blowout, and pryout), only those anchors susceptible to a particular failure mode should be considered when evaluating the strength associated with that failure mode.

anchor reinforcement—Anchor reinforcement is designed and detailed specifically for the purpose of transferring anchor loads from the anchors into the structural member. Hairpins are generally used for this purpose (refer to RE.5.2.9 and RE.6.2.9); however, other configurations that can be shown to effectively transfer the anchor load are acceptable.

brittle steel element—The 14 percent elongation should be measured over the gauge length specified in the appropriate ASTM standard for the steel.

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concrete pryout strength—the strength corresponding to formation of a concrete spall behind short, stiff anchors 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 A307 shall be considered a ductile steel element. Except as modified by E.3.3.4.3(a)6 for earthquake effects, deformed reinforcing bars meeting the requirements of ASTM A615, A706, or A955 shall be considered ductile steel elements.

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

COMMENTARY

ductile steel element—The 14 percent elongation should be measured over the gauge length specified in the appropriate ASTM standard for the steel. Due to concerns over fracture in cut threads, it should be verified that threaded deformed reinforcing bars satisfy the strength requirement of 12.9.1.3.2.

effective embedment depth—Effective embedment depths for a variety of anchor types are shown in Fig. RE.1.1.

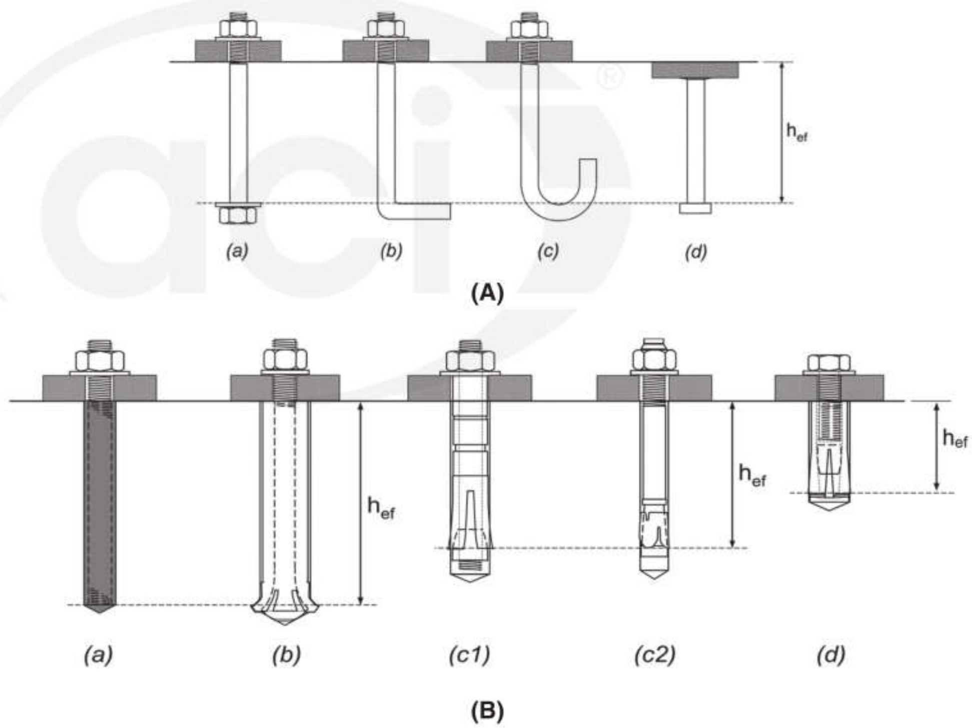

Fig. RE.1.1—Types of anchors: (A) Cast-in anchors: (a) hex head bolt with washer; (b) L-bolt; (c) J-bolt; and (d) welded headed stud; and (B) Post-installed anchors: (a) adhesive anchor; (b) undercut anchor; (c) torque-controlled expan-sion anchors: (c1) sleeve-type and (c2) stud-type; and (d) drop-in type displacement-controlled expansion anchor.

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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.

expansion anchor—a post-installed anchor, 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.

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.

hooked bolt—a cast-in anchor anchored mainly by bearing of the 90-degree bend (L-bolt) or 180-degree bend (J-bolt) against the concrete, at its embedded end, and having a minimum e_h of $3d_a$.

horizontal or upwardly inclined anchor—an anchor installed in a hole drilled horizontally or in a hole drilled at any orientation above horizontal.

COMMENTARY

five percent fractile—The determination of the coefficient K_{05} associated with the 5 percent fractile, $\bar{x} - K_{05}s_s$ depends on the number of tests, n, used to compute the sample mean $\bar{x}$ and sample standard deviation s_s. Values of K_{05} 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 E.4.2 is the same as the characteristic strength in ACI 355.2.

horizontal or upwardly inclined anchor—Figure RE.1.2 illustrates the potential hole orientations for horizontal or upwardly inclined anchors.

Fig. RE.1.2—Possible orientations of horizontal or upwardly inclined anchors.

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Manufacturer's Printed Installation Instructions (MPII)—published instructions for the correct installation of the anchor under all covered installation conditions as supplied in the product packaging.

post-installed anchor—an anchor installed in hardened concrete. Expansion, undercut, and adhesive 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. Refer to E.5.2.1 and E.6.2.1.

projected influence area—the rectilinear area on the free surface of the concrete member that is used to calculate the bond strength of adhesive anchors. Refer to E.5.5.1.

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.

stretch length—length of anchor, extending beyond concrete in which it is anchored, subject to full tensile load applied to anchor, and for which cross-sectional area is minimum and constant.

COMMENTARY

stretch length—Length of an anchor over which inelastic elongations are designed to occur for earthquake loadings. Examples illustrating stretch length are shown in Fig. RE.1.3.

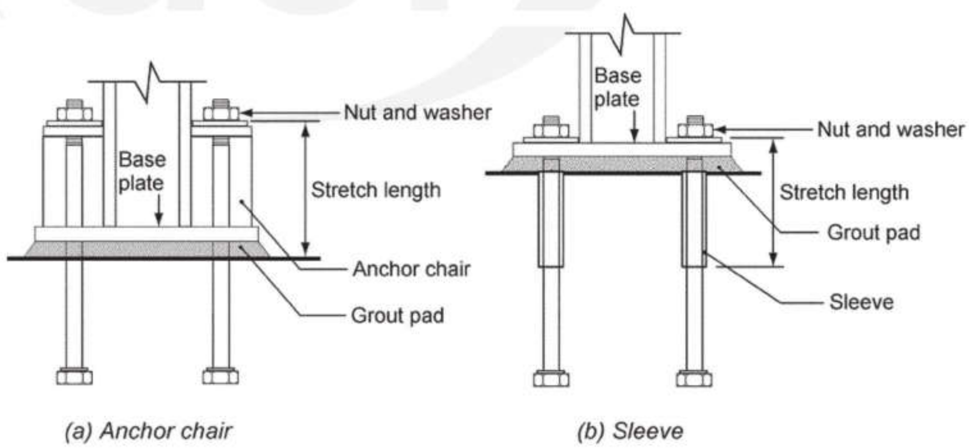

Fig. RE.1.3—Illustrations of stretch length (refer to E.3.3.4.3(a)).

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supplementary reinforcement—reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load from the anchors into 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 achieved with a special drill before installing the anchor or alternatively by the anchor itself during its installation.

E.2—Scope

E.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.

E.2.2 This appendix applies to cast-in anchors and to post-installed expansion (torque-controlled and displacement-controlled), undercut, and adhesive anchors. Adhesive anchors shall be installed in concrete having a minimum age of 21 days at time of anchor installation. Specialty inserts, through-bolts, multiple anchors connected to a single steel plate at the embedded end of the anchors, grouted anchors, and direct anchors such as powder or pneumatic actuated nails or bolts are not included in the provisions of Appendix E. Reinforcement used as part of the embedment shall be designed in accordance with other parts of this Code.

COMMENTARY

supplementary reinforcement—Supplementary reinforcement has a configuration and placement similar to anchor reinforcement but is not specifically designed to transfer loads from the anchors into the structural member. Stirrups, as used for shear reinforcement, may fall into this category.

RE.2—Scope

RE.2.1 Appendix E 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.

RE.2.2 Provisions for design of adhesive anchors were added in ACI 318-11. Adhesive anchors are particularly sensitive to numerous factors, including installation direction and loading type. Where adhesive anchors are used to resist sustained tension, the provisions include testing requirements for horizontal and upwardly inclined installations in E.3.4 and design and certification requirements for sustained tension load cases in E.3.5 and E.9.2.2 through E.9.2.4, respectively. Adhesive anchors qualified in accordance with ACI 355.4 are tested in concrete with compressive strengths within two ranges: 2500 to 4000 psi and 6500 to 8500 psi. Bond strength is in general not highly sensitive to concrete compressive strength. The design performance of adhesive anchors cannot be ensured by establishing a minimum concrete compressive strength at the time of installation in early-age concrete. Therefore, a minimum concrete age of 21 days at the time of adhesive anchor installation was adopted.

Selection of post-installed anchor type and material should consider effects of service conditions on both anchor and host structure durability. Non-adhesive post-installed anchors included in this appendix may not be appropriate in locations subject to being submerged or subject to repeated wet/dry cycles from splash or wash down unless the position of the reinforcing steel is field located and minimum reinforcement cover requirements are maintained between adjacent reinforcement and the anchor hole. Otherwise, to

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protect reinforcement from corrosion, cast-in-place or adhesive anchors are recommended.

Non-adhesive post-installed anchors may not be appropriate in areas subject to freezing where liquid can enter the hole around the anchor and expand damaging the concrete and compromising the anchor capacity.

The wide variety of shapes and configurations of specialty inserts precludes prescription of generalized tests and design equations. Specialty inserts are not covered by Appendix E provisions.

E.2.3 Design provisions are included for the following types of anchors:

(a) Headed studs and headed bolts having a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal to or exceeding $\mathbf{1.4}N_p$, where N_p is given in Eq. (E-14)

(b) Hooked bolts having a geometry that has been demonstrated to result in a pullout strength without the benefit of friction in uncracked concrete equal to or exceeding $\mathbf{1.4}N_p$, where N_p is given in Eq. (E-15)

(c) Post-installed expansion and undercut anchors that meet the assessment criteria of ACI 355.2

(d) Adhesive anchors that meet the assessment criteria of ACI 355.4

RE.2.3 Typical cast-in headed studs and headed bolts with geometries consistent with ANSI/ASME B1.1, B18.2.1, and B18.2.6 have been tested and proven to behave predictably, so calculated pullout strengths are acceptable.

Post-installed anchors do not have predictable pullout strengths, and, therefore, qualification tests to establish the pullout strengths per ACI 355.2 are required. 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. For adhesive anchors, the characteristic bond stress and suitability for structural applications are established by testing in accordance with ACI 355.4.

E.2.4 Load applications that are predominantly high cycle fatigue or impact loads are not covered by this appendix.

RE.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. E.3.3 presents additional requirements for design when seismic loads are included.

E.3—General requirements

E.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 ductile steel elements, provided that deformational compatibility is taken into account.

RE.3—General requirements

RE.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 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 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. Cook and Klinger (1992a,b) and Lotze et al. (2001) discuss nonlinear analysis, using theory of plasticity, for the determination of the capacities of ductile anchor groups.

E.3.1.1 Anchor group effects shall be considered wherever two or more anchors have spacing less than the critical spacing as follows:

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Failure mode under investigation	Critical spacing
Concrete breakout in tension	$3h_{ef}$
Bond strength in tension	$2c_{Na}$
Concrete breakout in shear	$3c_{a1}$

Only those anchors susceptible to the particular failure mode under investigation shall be included in the group.

E.3.2 The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in 9.2 or D.9.2.

E.3.3 *Seismic design requirements*

E.3.3.1 Anchors in structures assigned to Seismic Design Category C, D, E, or F shall satisfy the additional requirements of E.3.3.2 through E.3.3.7.

COMMENTARY

RE.3.3 *Seismic design requirements*

Unless E.3.3.4.1 or E.3.3.5.1 apply, all anchors in structures assigned to Seismic Design Categories C, D, E, or F are required to satisfy the additional requirements of E.3.3.1 through E.3.3.7 regardless of whether earthquake loads are included in the controlling load combination for the anchor design. In addition, all post-installed anchors in structures assigned to Seismic Design Categories C, D, E, or F must meet the requirements of ACI 355.2 or ACI 355.4 for prequalification of anchors to resist earthquake loads. Ideally, for tension loadings, anchor strength should be governed by yielding of the ductile steel element of the anchor. If the anchor cannot meet the specified ductility requirements of E.3.3.4.3(a), then the attachment should be either designed to yield if it is structural or light-gauge steel or designed to crush if it is wood. If ductility requirements of E.3.3.4.3(a) are satisfied, then any attachments to the anchor should be designed not to yield. In designing attachments using yield mechanisms to provide adequate ductility, as permitted by E.3.3.4.3(b) and E.3.3.5.3(a), the ratio of specified yield strength to expected strength for the material of the attachment should be considered in determining the design force. The value used for the expected strength should consider both material overstrength and strain hardening effects. For example, the material in a connection element could yield and, due to an increase in its strength with strain hardening, cause a secondary failure of a sub-element or place extra force or deformation demands on the anchors. For a structural steel attachment, if only the specified yield strength of the steel is known, the expected strength should be taken as approximately 1.5 times the specified yield strength. If the actual yield strength of the steel is known, the expected strength should be taken as approximately 1.25 times the actual yield strength.

Under seismic conditions, the direction of shear may not be predictable. The full shear force should be assumed in any direction for a safe design.

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E.3.3.2 The provisions of Appendix E do not apply to the design of anchors in plastic hinge zones of concrete structures under earthquake forces.

E.3.3.3 Post-installed anchors shall be qualified for earthquake loading in accordance with ACI 355.2 or ACI 355.4. The pullout strength N_p and steel strength in shear V_{sa} of expansion and undercut anchors shall be based on the results of the ACI 355.2 Simulated Seismic Tests. For adhesive anchors, the steel strength in shear V_{sa} and the characteristic bond stresses τ_{uncr} and τ_{cr} shall be based on results of the ACI 355.4 Simulated Seismic Tests.

E.3.3.4 *Requirements for tensile loading*

E.3.3.4.1 Where the tensile component of the strength-level earthquake force applied to a single anchor or group of anchors is equal to or less than 20 percent of the total factored anchor tensile force associated with the same load combination, it shall be permitted to design a single anchor or group of anchors to satisfy E.5 and the tensile strength requirements of E.4.1.1.

E.3.3.4.2 Where the tensile component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor tensile force associated with the same load combination, anchors and their attachments shall be designed in accordance with E.3.3.4.3. The anchor design tensile strength shall be determined in accordance with E.3.3.4.4.

E.3.3.4.3 Anchors and their attachments shall satisfy one of options (a) through (d):

(a) For single anchors, the concrete-governed strength shall be greater than the steel strength of the anchor. For anchor groups, the ratio of the tensile load on the most highly stressed anchor to the steel strength of that anchor shall be equal to or greater than the ratio of the tensile load on tension loaded anchors to the concrete-governed strength of those anchors. In each case:

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RE.3.3.2 The design provisions in Appendix E do not apply for anchors in plastic hinge zones. The possible higher levels of cracking and spalling in plastic hinge zones are beyond the conditions for which the nominal concrete-governed strength values in Appendix E are applicable. Plastic hinge zones are considered to extend a distance equal to twice the member depth from any column or beam face, and also include any other sections in walls, frames, and slabs where yielding of reinforcement is likely to occur as a result of lateral displacements.

Where anchors must be located in plastic hinge regions, they should be detailed so that the anchor forces are transferred directly to anchor reinforcement that is specifically designed to carry the anchor forces into the body of the member beyond the anchorage region. Configurations that rely on concrete tensile strength should not be used.

RE.3.3.3 Anchors that are not suitable for use in cracked concrete should not be used to resist earthquake loads. Qualification of post-installed anchors for use in cracked concrete is an integral part of the qualification for resisting earthquake loads in ACI 355.2 and ACI 355.4. The design values obtained from the Simulated Seismic Tests of ACI 355.2 and ACI 355.4 are expected to be less than those for static load applications.

RE.3.3.4 *Requirements for tensile loading*

RE.3.3.4.1 The requirements of E.3.3.4.3 need not apply where the applied earthquake tensile force is a small fraction of the total factored tension force.

RE.3.3.4.2 If the ductile steel element is ASTM A36 or ASTM A307 steel, the f_{uta}/f_{ya} value is typically about 1.5 and the anchor can stretch considerably before rupturing at the threads. For other steels, calculations may need to be made to ensure that a similar behavior can occur. RE.5.1.2 provides additional information on the steel properties of anchors. Provision of upset threaded ends, whereby the threaded end of the rod is enlarged to compensate for the area reduction associated with threading, can ensure that yielding occurs over the stretch length regardless of the ratio of the yield to ultimate strength of the anchor.

RE.3.3.4.3 Four options are provided for determining the required anchor or attachment strength to protect against nonductile tension failure:

In option (a), anchor ductility requirements are imposed and the required anchor strength is that determined using strength-level earthquake forces acting on the structure. Research (Hoehler and Eligehausen 2008; Vintzeleou and Eligehausen 1992) has shown that if the steel of the anchor yields before the concrete anchorage fails, no reduction in

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1. The steel strength shall be taken as 1.2 times the nominal steel strength of the anchor.
2. The concrete-governed strength shall be taken as the nominal strength considering pullout, side-face blowout, concrete breakout, and bond strength as applicable. For consideration of pullout in groups, the ratio shall be calculated for the most highly stressed anchor.
3. Anchors shall transmit tensile loads via a ductile steel element with a stretch length of at least eight anchor diameters unless otherwise determined by analysis.
4. Where anchors are subject to load reversals, the anchor shall be protected against buckling.
5. Where connections are threaded and the ductile steel elements are not threaded over their entire length, the ratio of f_{uta}/f_{ya} shall not be less than 1.3 unless the threaded portions are upset. The upset portions shall not be included in the stretch length.
6. Deformed reinforcing bars used as ductile steel elements to resist earthquake effects shall be limited to ASTM A615 Grades 40 and 60 satisfying the requirements of 13.1.5.2(a) and (b) or ASTM A706 Grade 60.

(b) The anchor or group of anchors shall be designed for the maximum tension that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects for the attachment. The anchor design tensile strength shall be calculated from E.3.3.4.4.

(c) The anchor or group of anchors shall be designed for the maximum tension that can be transmitted to the anchors by a non-yielding attachment. The anchor design tensile strength shall be calculated from E.3.3.4.4.

(d) The anchor or group of anchors shall be designed for the maximum tension obtained from design load combinations that include $\boldsymbol{E}$, with $\boldsymbol{E}$ increased by $\boldsymbol{\Omega_o}$. The anchor design tensile strength shall satisfy the tensile strength requirements of E.4.1.1.

COMMENTARY

the anchor tensile strength is needed for earthquake loadings. Ductile steel anchors should satisfy the definition for ductile steel elements in E.1. To facilitate comparison between steel strength, which is based on the most highly-stressed anchor, and concrete strength based on group behavior, the design is performed on the basis of the ratio of applied load to strength for the steel and concrete, respectively.

For some structures, anchors provide the best locations for energy dissipation in the nonlinear range of response. The stretch length of the anchor affects the lateral displacement capacity of the structure and, therefore, that length needs to be sufficient such that the displacement associated with the design-basis earthquake can be achieved (FEMA P-750). Observations from earthquakes indicate that the provision of a stretch length of eight anchor diameters results in good structural performance. Where the required stretch length is calculated, the relative stiffness of the connected elements needs to be considered. When an anchor is subject to load reversals, and its yielding length outside the concrete exceeds six anchor diameters, buckling of the anchor in compression is likely. Buckling can be restrained by placing the anchor in a tube. However, care must be taken that the tube does not share in resisting the tensile load assumed to act on the anchor. For anchor bolts that are not threaded over their length, it is important to ensure that yielding occur over the unthreaded portion of the bolt within the stretch length prior to failure in the threads. This is accomplished by maintaining sufficient margin between the specified yield and ultimate strengths of the bolt. It should be noted that the available stretch length may be adversely influenced by construction techniques (for example, the addition of leveling nuts to the examples shown in Fig. RE.1.3).

In option (b), the anchor is designed for the tension force associated with the expected strength of the metal or similar material of the attachment. For option (b), as discussed in RE.3.3, care must be taken in design to consider the consequences of potential differences between the specified yield strength and the expected strength of the attachment. An example is 13.4.3 for the design of connections of intermediate precast walls where a connection not designed to yield should develop at least $\mathbf{1.5}\boldsymbol{S_y}$, where $\boldsymbol{S_y}$ is the nominal strength of the yielding element based on its specified yield strength. Similarly, steel design manuals require structural steel connections that are designated nonyielding and part of the seismic load path to have design strengths that exceed a multiple of the nominal strength. That multiple depends on a factor relating the likely actual to specified yield strength of the material and an additional factor exceeding unity to account for material strain hardening. For attachments of cold-formed steel or wood, similar principles should be used for determining the expected strength of the attachment to determine the required strength of the anchorage.

Additional guidance on the use of options (a) through (d) is provided in FEMA P-750. The design of anchors in accordance with option (a) should be used only where the anchor yield behavior is well defined and where the interaction of

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the yielding anchor with other elements in the load path has been adequately addressed. For the design of anchors per option (b), the force associated with yield of a steel attachment, such as an angle, baseplate, or web tab, should be the expected strength, rather than the specified yield strength of the steel. Option (c) may apply to a variety of special cases, such as the design of sill bolts where the crushing of the wood limits the force that can be transferred to the bolt, or where the provisions of ANSI/AISC 341 specify loads based on member strengths.

E.3.3.4.4 The anchor design tensile strength for resisting earthquake forces shall be determined from consideration of (a) through (e) for the failure modes given in Table E.4.1.1 assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked:

(a) ϕN_{sa} for a single anchor, or for the most highly stressed individual anchor in a group of anchors

(b) $\mathbf{0.75}\phi N_{cb}$ or $\mathbf{0.75}\phi N_{cbg}$, except that N_{cb} or N_{cbg} need not be calculated where anchor reinforcement satisfying E.5.2.9 is provided

(c) $\mathbf{0.75}\phi N_{pn}$ for a single anchor, or for the most highly stressed individual anchor in a group of anchors

(d) $\mathbf{0.75}\phi N_{sb}$ or $\mathbf{0.75}\phi N_{sbg}$

(e) $\mathbf{0.75}\phi N_{a}$ or $\mathbf{0.75}\phi N_{ag}$

where ϕ is in accordance with E.4.3 or E.4.4.

RE.3.3.4.4 The reduced anchor nominal tensile strengths associated with concrete failure modes is to account for increased cracking and spalling in the concrete resulting from seismic actions. Because seismic design generally assumes that all or portions of the structure are loaded beyond yield, it is likely that the concrete is cracked throughout for the purpose of determining the anchor strength. In locations where it can be demonstrated that the concrete does not crack, uncracked concrete may be assumed for determining the anchor strength as governed by concrete failure modes.

E.3.3.4.5 Where anchor reinforcement is provided in accordance with E.5.2.9, no reduction in design tensile strength beyond that specified in E.5.2.9 shall be required.

RE.3.3.4.5 Where anchor reinforcement as defined in E.5.2.9 and E.6.2.9 is used, with the properties as defined in 13.1.5.2, no separation of the potential breakout prism from the substrate is likely to occur, provided the anchor reinforcement is designed for a load greater than the concrete breakout strength.

E.3.3.5 *Requirements for shear loading*

RE.3.3.5 *Requirements for shear loading*

Where the shear component of the earthquake force applied to the anchor exceeds 20 percent of the total anchor shear force, three options are recognized for determining the required shear strength to protect the anchor or group of anchors against premature shear failure. There is no option corresponding to option (a) of E.3.3.4.3 because the cross section of the steel element of the anchor cannot be configured so that steel failure in shear provides any meaningful degree of ductility.

Design of the anchor or group of anchors for the strength associated with force-limiting mechanisms under option (b), such as the bearing strength at holes in a steel attachment or the combined crushing and bearing strength for wood members may be particularly relevant. Tests on typical anchor bolt connections for wood framed shear walls (Fennel et al. 2009) showed that wood components attached to concrete with minimum edge distances exhibited ductile behavior. Wood "yield" (crushing) was the first limiting state and resulted in nail slippage in shear. Nail slippage combined with bolt bending provided the required ductility and toughness for the shear walls and limited the loads acting on the

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bolts. Procedures for defining bearing and shear limit states for connections to cold-formed steel are described in AISI S100-07 and examples of strength calculations are provided in the AISI *Cold-Formed Steel Design Manual* (AISI 2008). In such cases, consideration should be given to whether exceedance of the bearing strength may lead to tearing and an unacceptable loss of connectivity. Where anchors are located far from edges, it may not be possible to design such that anchor reinforcement controls the anchor strength. In such cases, anchors should be designed for overstrength in accordance with option (c).

E.3.3.5.1 Where the shear component of the strength-level earthquake force applied to the anchor or group of anchors is equal to or less than 20 percent of the total factored anchor shear force associated with the same load combination, it shall be permitted to design the anchor or group of anchors to satisfy E.6 and the shear strength requirements of E.4.1.1.

RE.3.3.5.1 The requirements of E.3.3.5.3 need not apply where the applied earthquake shear force is a small fraction of the total factored shear force.

E.3.3.5.2 Where the shear component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor shear force associated with the same load combination, anchors and their attachments shall be designed in accordance with E.3.3.5.3. The anchor design shear strength for resisting earthquake forces shall be determined in accordance with E.6.

E.3.3.5.3 Anchors and their attachments shall be designed using one of options (a) through (c):

(a) The anchor or group of anchors shall be designed for the maximum shear that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects in the attachment.

(b) The anchor or group of anchors shall be designed for the maximum shear that can be transmitted to the anchors by a non-yielding attachment.

(c) The anchor or group of anchors shall be designed for the maximum shear obtained from design load combinations that include $\boldsymbol{E}$, with $\boldsymbol{E}$ increased by $\boldsymbol{\Omega_o}$. The anchor design shear strength shall satisfy the shear strength requirements of E.4.1.1.

E.3.3.5.4 Where anchor reinforcement is provided in accordance with E.6.2.9, no reduction in design shear strength beyond that specified in E.6.2.9 shall be required.

E.3.3.6 Single anchors or groups of anchors that are subjected to both tension and shear forces shall be designed to satisfy the requirements of E.7, with the anchor design tensile strength calculated from E.3.3.4.4.

E.3.3.7 Anchor reinforcement used in structures assigned to Seismic Design Category C, D, E, or F shall

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reinforcement and shall be limited to ASTM A615 Grades 40 and 60 satisfying the requirements of 13.1.5.2(a) and (b) or ASTM A706 Grade 60.

E.3.4 Adhesive anchors installed horizontally or upwardly inclined shall be qualified in accordance with ACI 355.4 requirements for sensitivity to installation direction.

E.3.5 For adhesive anchors subjected to sustained tension loading, E.4.1.2 shall be satisfied. For groups of adhesive anchors, Eq. (E-1) shall be satisfied for the anchor that resists the highest sustained tension load. Installer certification and inspection requirements for horizontal and upwardly inclined adhesive anchors subjected to sustained tension loading shall be in accordance with E.9.2.2 through E.9.2.4.

E.3.6 Modification factor λ_a for lightweight concrete shall be taken as:

Cast-in and undercut anchor concrete failure: **1.0λ**

Expansion and adhesive anchor concrete failure: **0.8λ**

Adhesive anchor bond failure per Eq. (E-22): **0.6λ**

where λ is determined in accordance with 8.6.1. It shall be permitted to use an alternate value of λ_a where tests have been performed and evaluated in accordance with ACI 355.2 or ACI 355.4.

E.3.7 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 with f_c' greater than 8000 psi.

COMMENTARY

RE.3.4 ACI 355.4 includes optional tests to confirm the suitability of adhesive anchors for horizontal and upwardly inclined installations.

RE.3.5 For adhesive anchors subjected to sustained tension loading, an additional calculation for the sustained portion of the factored load for a reduced bond resistance is required to account for possible bond strength reductions under sustained load. The resistance of adhesive anchors to sustained tension load is particularly dependent on correct installation, including hole cleaning, adhesive metering and mixing, and prevention of voids in the adhesive bond line (annular gap). In addition, care should be taken in the selection of the correct adhesive and bond strength for the expected conditions on-site such as the concrete condition during installation (dry or saturated, cold or hot), the drilling method used (rotary impact drill/rock drill or core drill), and anticipated in-service temperature variations in the concrete. Installer certification and inspection requirements associated with the use of adhesive anchors for horizontal and upwardly inclined installations to resist sustained tension loads are addressed in E.9.2.2 through E.9.2.4.

Adhesive anchors are particularly sensitive to installation direction and loading type. Adhesive anchors installed overhead that resist sustained tension loads are of concern because previous applications of this type have led to failures. Other anchor types may be more appropriate for such cases. Where adhesive anchors are used in overhead applications subjected to sustained tension loading, it is essential to meet test requirements of ACI 355.4 for sensitivity to installation direction, use certified installers, and require special inspection.

RE.3.6 The number of tests available to establish the strength of anchors in lightweight concrete is limited. Lightweight concrete tests of cast-in headed studs indicate that the present reduction factor λ adequately captures the influence of lightweight concrete (Shaikh and Yi 1985; Anderson and Meinheit 2005). Anchor manufacturer data developed for evaluation reports on both post-installed expansion and adhesive anchors indicate that a reduced λ is needed to provide the necessary safety factor for the respective design strength. ACI 355.2 and ACI 355.4 provide procedures whereby a specific value of λ_a can be used based on testing, assuming the lightweight concrete is similar to the reference test material.

RE.3.7 A limited number of tests of cast-in and post-installed anchors in high-strength concrete (Primavera et al. 1997) indicate that the design procedures contained in this appendix become unconservative, particularly for cast-in anchors in concrete with compressive strengths in the range

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of 11,000 to 12,000 psi. Until further tests are available, an upper limit on f_c' of 10,000 psi has been imposed in the design of cast-in anchors. This limitation is consistent with Chapters 11 and 12. ACI 355.2 and ACI 355.4 do not require testing of post-installed anchors in concrete with f_c' greater than 8000 psi. Some post-installed expansion anchors may have difficulty expanding in very high-strength concretes and the bond strength of adhesive anchors may be negatively affected by very high-strength concrete. Therefore, f_c' is limited to 8000 psi in the design of post-installed anchors unless testing is performed.

E.4—General requirements for strength of anchors

E.4.1 Strength design of anchors shall be based either on computation using design models that satisfy the requirements of E.4.2, or on test evaluation using the 5 percent fractile of applicable test results for the following:

(a) Steel strength of anchor in tension (E.5.1)
(b) Concrete breakout strength of anchor in tension (E.5.2)
(c) Pullout strength cast-in, post-installed expansion or undercut anchor in tension (E.5.3)
(d) Concrete side-face blowout strength of headed anchor in tension (E.5.4)
(e) Bond strength of adhesive anchor in tension (E.5.5)
(f) Steel strength of anchor in shear (E.6.1)
(g) Concrete breakout strength of anchor in shear (E.6.2)
(h) Concrete pryout strength of anchor in shear (E.6.3)

In addition, anchors shall satisfy the required edge distances, spacings, and thicknesses to preclude splitting failure, as required in E.8.

E.4.1.1 The design of anchors shall be in accordance with Table E.4.1.1. In addition, the design of anchors shall satisfy E.3.3 for earthquake loading and E.4.1.2 for adhesive anchors subject to sustained tensile loading.

RE.4—General requirements for strength of anchors

RE.4.1 This section provides requirements for establishing the strength of anchors in concrete. The various types of steel and concrete failure modes for anchors are shown in Fig. RE.4.1(a) and RE.4.1(b). Comprehensive discussions of anchor failure modes are included in CEB (1997), Fuchs et al. (1995), Eligehausen and Balogh (1995), Eligehausen et al. (2006a), and Cook et al. (1998). Tension failure modes related to concrete capacity include concrete breakout failure in E.5.2 (applicable to all anchor types), pullout failure in E.5.3 (applicable to cast-in anchors and post-installed expansion and undercut anchors), side-face blowout failure in E.5.4 (applicable to headed anchors), and bond failure in E.5.5 (applicable to adhesive anchors). Shear failure modes related to concrete capacity include concrete breakout failure and concrete pryout in E.6.2 and E.6.3, respectively (applicable to all anchor types). Any model that complies with the requirements of E.4.1.3 and E.4.2 can be used to establish the concrete-related strengths. Additionally, anchor tensile and shear strengths are limited by the minimum spacings and edge distances of E.8 as required to preclude splitting. The design of post-installed anchors recognizes that the strength of anchors is sensitive to appropriate installation; installation requirements are included in E.9. Some post-installed anchors are less sensitive to installation errors

Table E.4.1.1—Required strength of anchors, except as noted in E.3.3

Failure mode	Single anchor	Anchor group*: Individual anchor in a group	Anchor group*: Anchors as a group
Steel strength in tension (E.5.1)	$\phi N_{sa} \geq N_{ua}$	$\phi N_{sa} \geq N_{ua,i}$	
Concrete breakout strength in tension (E.5.2)	$\phi N_{cb} \geq N_{ua}$		$\phi N_{cbg} \geq N_{ua,g}$
Pullout strength in tension (E.5.3)	$\phi N_{pn} \geq N_{ua}$	$\phi N_{pn} \geq N_{ua,i}$	
Concrete side-face blowout strength in tension (E.5.4)	$\phi N_{sb} \geq N_{ua}$		$\phi N_{sbg} \geq N_{ua,g}$
Bond strength of adhesive anchor in tension (E.5.5)	$\phi N_a \geq N_{ua}$		$\phi N_{ag} \geq N_{ua,g}$
Steel strength in shear (E.6.1)	$\phi V_{sa} \geq V_{ua}$	$\phi V_{sa} \geq V_{ua,i}$	
Concrete breakout strength in shear (E.6.2)	$\phi V_{cb} \geq V_{ua}$		$\phi V_{cbg} \geq V_{ua,g}$
Concrete pryout strength in shear (E.6.3)	$\phi V_{cp} \geq V_{ua}$		$\phi V_{cpg} \geq V_{ua,g}$

*Required strengths for steel and pullout failure modes shall be calculated for the most highly stressed anchor in the group.

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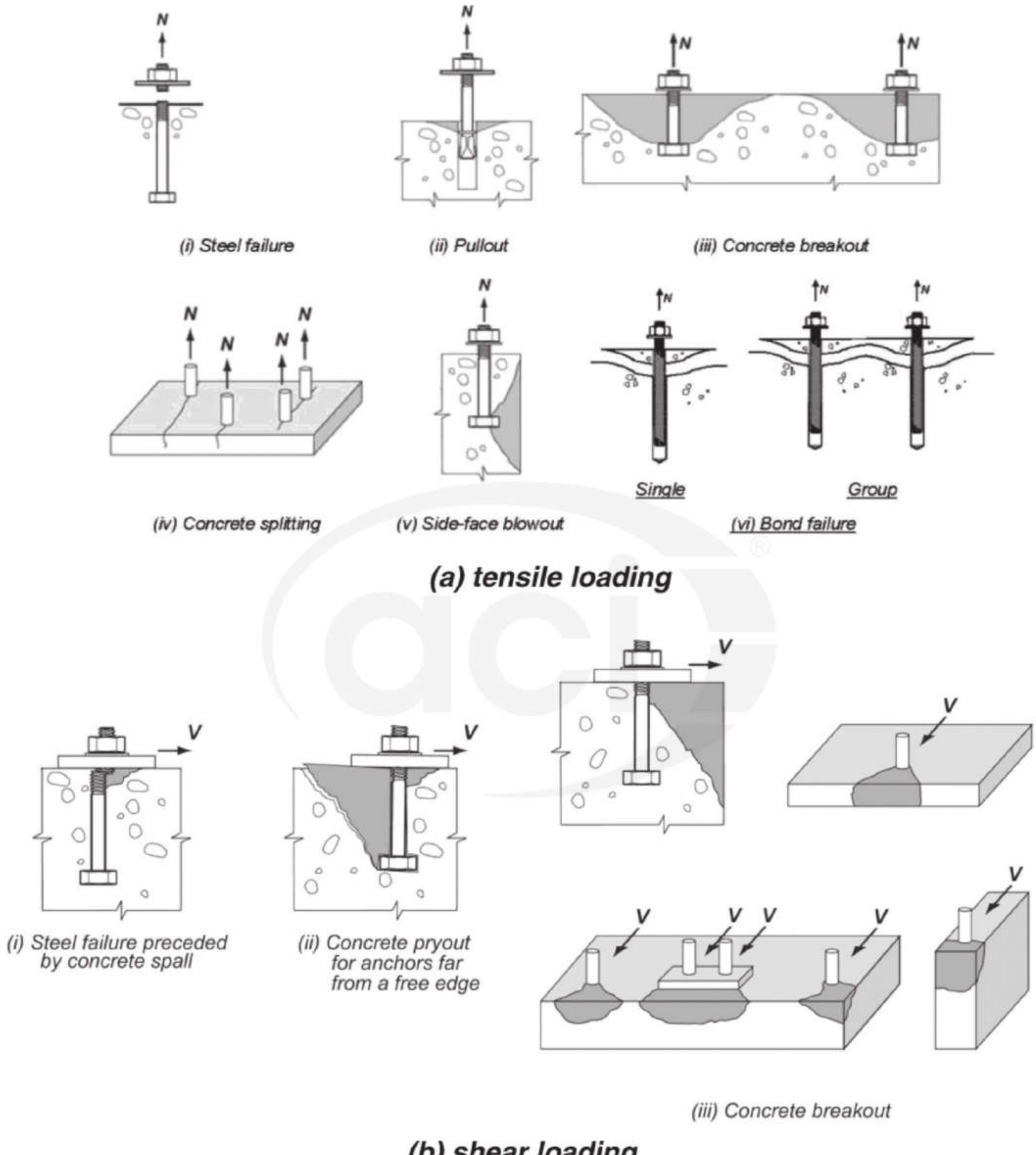

Fig. RE.4.1—Failure modes for anchors.

and tolerances. This is reflected in varied ϕ-factors, given in E.4.3 and E.4.4, based on the assessment criteria of ACI 355.2 and ACI 355.4.

Test procedures can also be used to determine the single-anchor breakout strength in tension and in shear. The test

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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 E.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.

Under combined tension and bending, individual anchors in a group are subjected to different magnitude tensile forces. Similarly, under combined shear and torsion, individual anchors in a group are subjected to different magnitude shear forces. Table E.4.1.1 includes requirements to design single anchors and individual anchors in a group to safeguard against all potential failure modes. For steel and pullout failure modes, the most highly stressed anchor in the group should be checked to ensure it has sufficient capacity to carry its required load, whereas for concrete breakout, the anchors should be checked as a group. Elastic analysis or plastic analysis of ductile anchors as described in E.3.1 may be used to determine the loads carried by each anchor.

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E.4.1.2 For the design of adhesive anchors to resist sustained tensions loads, in addition to E.4.1.1,

$$0.55\phi N_{ba} \geq N_{ua,s} \quad \text{(E-1)}$$

where N_{ba} is determined in accordance with E.5.5.2.

E.4.1.3 When both N_{ua} and V_{ua} are present, interaction effects shall be considered 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 E.7.

E.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.

COMMENTARY

RE.4.1.2 The 0.55 factor used for the additional calculation for sustained loads is correlated with ACI 355.4 test requirements and provides satisfactory performance of adhesive anchors under sustained tension loads when used in accordance with ACI 355.4. Product evaluation according to ACI 355.4 is based on sustained tension loading being present for a minimum of 50 years at a standard temperature of 70°F and a minimum of 10 years at a temperature of 110°F. For longer life spans (for example, greater than 50 years) or higher temperatures, lower factors should be considered.

RE.4.1.3 and RE.4.2 E.4.1.3 and E.4.2 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 E.4.2 as long as sufficient data are available to verify the model.

The method for concrete breakout design included as "considered to satisfy" E.4.2 was developed from the Concrete Capacity Design (CCD) Method (Fuchs et al. 1995; Eligehausen and Balogh 1995), which was an adaptation of the κ Method (Eligehausen et al. 2006a; Eligehausen and Fuchs 1988) and is considered to be accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD Method predicts the strength 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. Experimental and numerical investigations have demonstrated the applicability of the CCD Method to adhesive anchors as well (Eligehausen et al. 2006a).

The breakout strength calculations are based on a model suggested in the κ Method. It is consistent with a breakout

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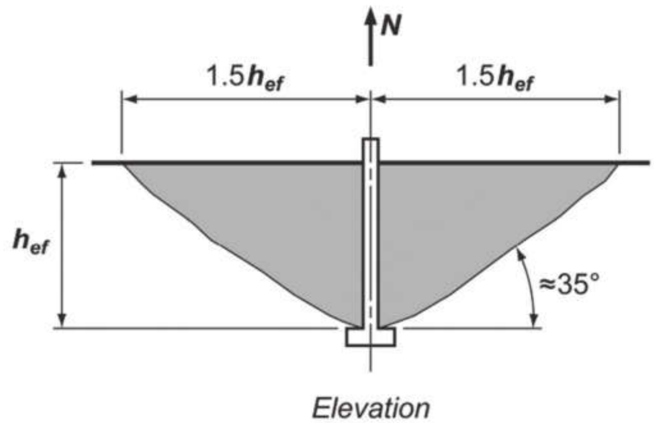

Fig. RE.4.2a—Breakout cone for tension.

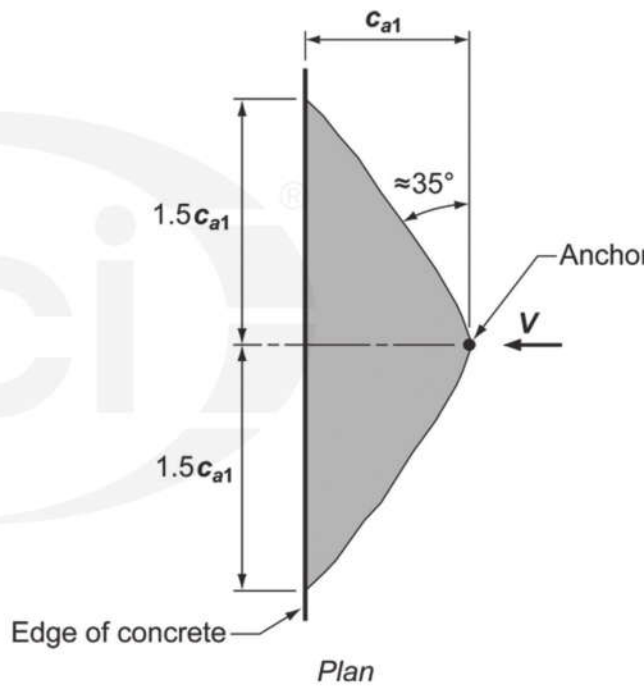

Fig. RE.4.2b—Breakout cone for shear.

prism angle of approximately 35 degrees (Fig. RE.4.2a and RE.4.2b).

E.4.2.1 The effect of reinforcement provided to restrain the concrete breakout shall be permitted to be included in the design models used to satisfy E.4.2. Where anchor reinforcement is provided in accordance with E.5.2.9 and E.6.2.9, calculation of the concrete breakout strength in accordance with E.5.2 and E.6.2 is not required.

RE.4.2.1 The addition of reinforcement in the direction of the load to restrain concrete breakout can greatly enhance the strength and deformation capacity of the anchor connection. Such enhancement is practical with cast-in anchors such as those used in precast sections.

CEB (1994, 1997), Klingner et al. (1982), ACI 349, and Eligehausen et al. (2006a) provide information regarding the effect of reinforcement on the behavior of anchors. The effect of reinforcement is not included in the ACI 355.2 and ACI 355.4 anchor acceptance tests or in the concrete breakout calculation method of E.5.2 and E.6.2. The beneficial effect of supplementary reinforcement is recognized by the Condition A ϕ-factors in E.4.3 and E.4.4. Anchor rein-

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forcement may be provided instead of calculating breakout strength using the provisions of Chapter 12 in conjunction with E.5.2.9 and E.6.2.9.

The breakout strength of an unreinforced connection can be taken as an indication of the load at which significant cracking will occur. Such cracking can represent a serviceability problem if not controlled. (Refer to RE.6.2.1.)

E.4.2.2 For anchors with diameters not exceeding 4 in., the concrete breakout strength requirements shall be considered satisfied by the design procedure of E.5.2 and E.6.2.

RE.4.2.2 The limitation on anchor diameter is based on the current range of test data. In the 2002 through 2008 editions of the ACI 318 code, there were limitations on the diameter and embedment of anchors to compute the concrete breakout strength. These limitations were necessitated by the lack of test results on anchors with diameter larger than 2 in. and embedment length longer than 24 in. In 2011, limitations on anchor diameter and embedment length were revised to limit the diameter to 4 in. diameter based on the results of tension and shear tests on large-diameter anchors with deep embedments (Lee et al. 2007, 2010). These tests included 4.25 in. diameter anchors embedded 45 in. in tension tests and 3.5 in. diameter anchors in shear tests. The reason for this 4 in. diameter limit is that the largest-diameter anchor in ASTM F1554 is 4 in., whereas other ASTM specifications permit up to 8 in. diameter anchors that have not been tested to ensure applicability of the E.5.2 and E.6.2 concrete breakout provisions.

E.4.2.3 For adhesive anchors with embedment depths $4d_a \leq h_{ef} \leq 20d_a$, the bond strength requirements shall be considered satisfied by the design procedure of E.5.5.

RE.4.2.3 ACI 355.4 limits the embedment depth of adhesive anchors to $4d_a \leq h_{ef} \leq 20d_a$, which represents the theoretical limits of the bond model (Eligehausen et al. 2006a).

E.4.3 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 or ACI 355.4—		
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

RE.4.3 The ϕ-factors for steel strength are based on using f_{uta} to determine the nominal strength of the anchor (refer to E.5.1 and E.6.1) rather than f_{ya} as used in the design of reinforced concrete members. Although the ϕ-factors for use with f_{uta} appear low, they result in a level of safety consistent with the use of higher ϕ-factors applied to f_{ya}. The smaller ϕ-factors for shear than for tension do not reflect basic material differences but rather account for the possibility of a nonuniform 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 corresponding to anchor forces no greater than the minimum design strength of the anchors specified in E.3.3 (refer to E.3.3.4.3 and E.3.3.5.3).

For anchors governed by the more brittle concrete breakout or blowout failure, two conditions are recognized. If supplementary reinforcement is present (Condition A), greater deformation capacity is provided than in the case where such supplementary reinforcement is not present (Condition B). An explicit design of supplementary reinforcement is not required. However, the arrangement of supplementary reinforcement should generally conform to that of the anchor reinforcement shown in Fig. RE.5.2.9 and RE.6.2.9(b). Full development is not required.

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Category 3: (High sensitivity to installation and lower reliability)	0.55	0.45

Condition A applies where supplementary reinforcement is present except for pullout and pryout strengths.

Condition B applies where supplementary reinforcement is not present, and for pullout or pryout strength.

E.4.4 Strength reduction factor ϕ for anchors in concrete shall be as follows when the load combinations referenced in Appendix D are used:

a) Anchor governed by strength of a ductile steel element
  i) Tension loads: 0.80
  ii) Shear loads: 0.75

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 or ACI 355.4—		
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

COMMENTARY

The strength reduction factors for anchor reinforcement are given in E.5.2.9 and E.6.2.9. Further discussion of strength reduction factors is presented in RE.4.4.

The ACI 355.2 tests for sensitivity to installation procedures determine the reliability category appropriate for a particular expansion or undercut anchoring device. In the ACI 355.2 tests for expansion and undercut anchors, the effects of variability in anchor torque during installation, tolerance on drilled hole size, and energy level used in setting anchors are considered; for expansion and undercut anchors approved for use in cracked concrete, increased crack widths are considered. ACI 355.4 tests for sensitivity for installation procedures determine the category for a particular adhesive anchor system considering the influence of adhesive mixing and the influence of hole cleaning in dry, saturated and water-filled/underwater bore holes. 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

Category 3—high sensitivity to installation and lower reliability

The strengths 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.

RE.4.4 As noted in R9.1, the 2002 ACI 318 code incorporated the load factors of ASCE/SEI 7-02 and the corresponding strength reduction factors provided in ACI 318-99 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 E were based on the 1999 ACI 318 Sections 9.2 and 9.3 load and strength reduction factors. The resulting ϕ-factors are presented in E.4.4 for use with the load factors of Appendix D, starting with ACI 318-02. The ϕ-factors for use with the load factors of the 1999 ACI 318 Appendix D were determined in a manner consistent with the other ϕ-factors of the 1999 ACI 318 Appendix D. These ϕ-factors are presented in E.4.3 for use with the load factors of 9.2, starting with the 2002 ACI 318 code. Because developmental studies for ϕ-factors to be used with Appendix E for brittle concrete failure modes, were performed for the load and strength reduction factors now given in Appendix D, the discussion of the selection of these ϕ-factors appears in this section.

Even though the ϕ-factor for structural plain concrete in Appendix D is 0.65, the basic factor for brittle concrete failures (ϕ = **0.75**) was chosen based on results of probabilistic studies (Farrow and Klingner (1995) 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

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Category 3: (High sensitivity to installation and lower reliability)	0.65	0.55

Condition A applies where supplementary reinforcement is present except for pullout and pryout strengths. Condition B applies where supplementary reinforcement is not present, and for pullout and pryout strengths.

E.5—Design requirements for tensile loading

E.5.1 *Steel strength of anchor in tension*

E.5.1.1 The nominal strength of an anchor in tension as governed by the steel, N_{sa}, shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

E.5.1.2 The nominal strength of an anchor in tension, N_{sa}, shall not exceed

$$N_{sa} = A_{se,N}f_{uta} \quad \text{(E-2)}$$

where $A_{se,N}$ is the effective cross-sectional area of an anchor in tension, in.^{2}, and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 125,000 psi.

E.5.2 *Concrete breakout strength of anchor in tension*

E.5.2.1 The nominal concrete breakout strength in tension, N_{cb} of a single anchor or N_{cbg} of a group of anchors, shall not exceed:

COMMENTARY

other design procedures and probabilistic studies (Farrow and Klingner 1995) indicated that the choice of $\phi = 0.75$ was justified. Applications with supplementary reinforcement (Condition A) provide more deformation capacity, permitting the ϕ-factors to be increased. The value of $\phi = 0.85$ is compatible with the level of safety for shear failures in concrete beams and has been recommended in the *PCI Design Handbook* (PCI 2010) and by ACI 349.

RE.5—Design requirements for tensile loading

RE.5.1 *Steel strength of anchor in tension*

RE.5.1.2 The nominal strength of anchors in tension is best represented as a function of f_{uta} rather than f_{ya} because most anchor materials do not exhibit a well-defined yield point. The American Institute of Steel Construction (AISC) has based tension strength of anchors on $A_{se,N}f_{uta}$ since the 1986 edition of their specifications. The use of Eq. (E-2) with 9.2 load factors and the ϕ-factors of E.4.3 give design strengths consistent with the AISC "Load and Resistance Factor Design Specification for Structural Steel Buildings."

The limitation of $1.9f_{ya}$ on f_{uta} is to ensure that, under service load conditions, the anchor does not exceed f_{ya}. The limit on f_{uta} of $1.9f_{ya}$ was determined by converting the LRFD provisions to corresponding service level conditions. For Section 9.2, the average load factor of 1.4 (from $1.2D + 1.6L$) divided by the highest ϕ-factor (0.75 for tension) results in a limit of f_{uta}/f_{ya} of 1.4/0.75 = 1.87. For Appendix D, the average load factor of 1.55 (from $1.4D + 1.7L$), divided by the highest ϕ-factor (0.80 for tension), results in a limit of f_{uta}/f_{ya} of 1.55/0.8 = 1.94. For consistent results, the serviceability limitation of f_{uta} was taken as $1.9f_{ya}$. If the ratio of f_{uta} to f_{ya} exceeds this value, the anchoring may be subjected to service loads above f_{ya} under service loads. Although not a concern for standard structural steel anchors (maximum value of f_{uta}/f_{ya} is 1.6 for ASTM A307), the limitation is applicable to some stainless steels.

For post-installed anchors having a reduced cross-sectional area anywhere along the anchor length, such as wedge-type anchors, the effective cross-sectional area of the anchor should be provided by the manufacturer. For threaded rods and headed bolts, ANSI/ASME B1.1E.1 defines $A_{se,N}$ as

$$A_{se,N} = \frac{\pi}{4}\left(d_a - \frac{0.9743}{n_t}\right)^2$$

where n_t is the number of threads per inch.

RE.5.2 *Concrete breakout strength of anchor in tension*

RE.5.2.1 The effects of multiple anchors, spacing of anchors, and edge distance on the nominal concrete breakout

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(a) For a single anchor

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{(E-3)}$$

(b) For a group of anchors

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{(E-4)}$$

Factors $\psi_{ec,N}$, $\psi_{ed,N}$, $\psi_{c,N}$, and $\psi_{cp,N}$ are defined in E.5.2.4, E.5.2.5, E.5.2.6, and E.5.2.7, respectively. A_{Nc} is the projected concrete failure area of a single 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_{Nc} shall not exceed nA_{Nco}, where n is the number of anchors in the group that resist tension. A_{Nco} is the projected concrete failure area of a single anchor with an edge distance equal to or greater than $1.5h_{ef}$

$$A_{Nco} = 9h_{ef}^2 \quad \text{(E-5)}$$

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strength in tension are included by applying the modification factors A_{Nc}/A_{Nco} and $\psi_{ed,N}$ in Eq. (E-3) and (E-4).

Figure RE.5.2.1(a) shows A_{Nco} and the development of Eq. (E-5). A_{Nco} is the maximum projected area for a single anchor. Figure RE.5.2.1(b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. Because A_{Nc} is the total projected area for a group of anchors, and A_{Nco} is the area for a single anchor, there is no need to include n, the number of anchors, in Eq. (E-3) or (E-4). If anchor groups are positioned in such a way that their projected areas overlap, the value of A_{Nc} is required to be reduced accordingly.

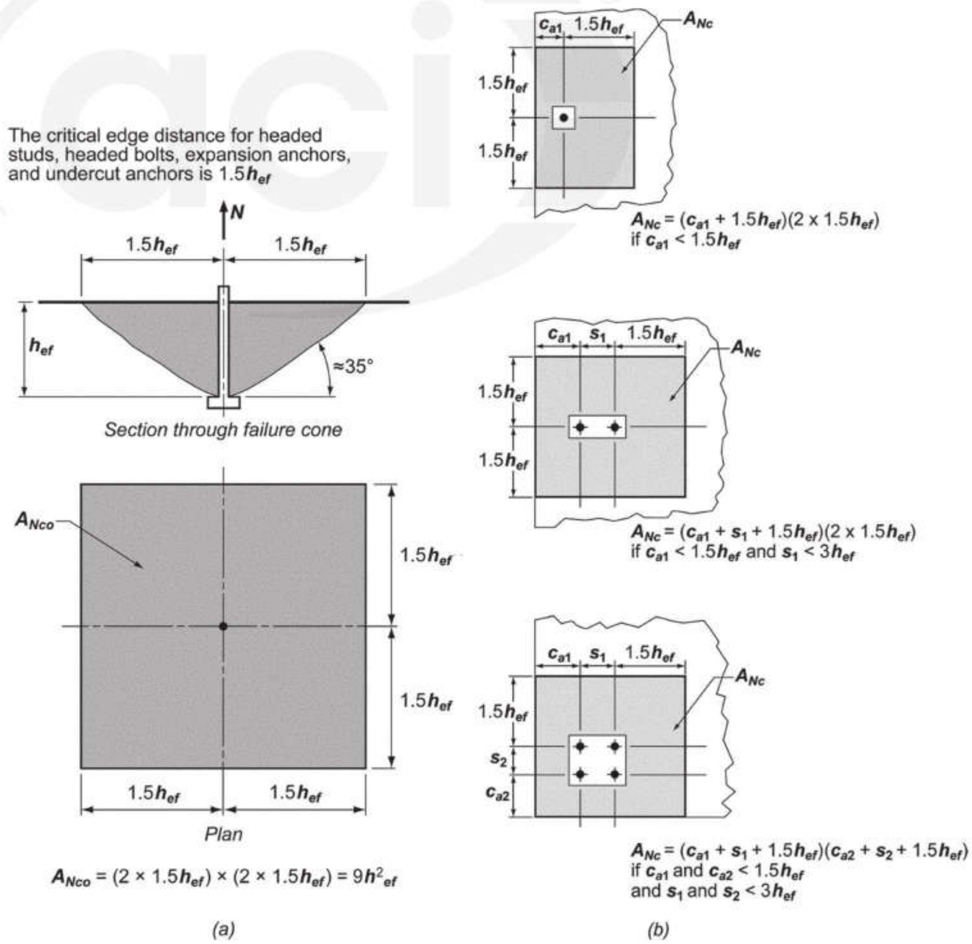

Fig. RE.5.2.1—(a) Calculation of $\mathbf{A_{Nco}}$*; and (b) calculation of* $\mathbf{A_{Nc}}$ *for single anchors and groups of anchors.*

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E.5.2.2 The basic concrete breakout strength of a single anchor in tension in cracked concrete, N_b, shall not exceed

$$N_b = k_c\lambda_a\sqrt{f_c'}\,h_{ef}^{1.5} \quad \text{(E-6)}$$

where k_c = 24 for cast-in anchors and k_c = 17 for post-installed anchors.

The value of k_c for post-installed anchors shall be permitted to be increased above 17 based on ACI 355.2 or ACI 355.4 product-specific tests but shall in no case exceed 24.

Alternatively, for cast-in headed studs and headed bolts with **11 in.** $\leq h_{ef} \leq$ **25 in.**, N_b shall not exceed

$$N_b = 16\lambda_a\sqrt{f_c'}\,h_{ef}^{5/3} \quad \text{(E-7)}$$

E.5.2.3 Where anchors are located less than $1.5h_{ef}$ from three or more edges, the value of h_{ef} used for the calculation of A_{Nc} in accordance with E.5.2.1, as well as in Eq. (E-3) through (E-10) shall be the larger of $c_{a,max}/1.5$ and $s/3$, where s is the maximum spacing between anchors within the group.

COMMENTARY

RE.5.2.2 The equation for the basic concrete breakout strength was derived (Fuchs et al. 1995; Eligehausen and Balogh 1995; Eligehausen and Fuchs 1987, 1988; CEB 1994) assuming a concrete failure prism with an angle of approximately 35 degrees, considering fracture mechanics concepts.

The values of k_c in Eq. (E-6) were determined from a large database of test results in uncracked concrete (Fuchs et al. 1995) at the 5 percent fractile. The values were adjusted to corresponding k_c values for cracked concrete (Eligehausen and Balogh 1995; Goto 1971). Tests have shown that the values of k_c applicable to adhesive anchors are approximately equal to those derived for expansion anchors (Eligehausen et al. 2006a; Zhang and Klingner 2001). Higher k_c values for post-installed anchors may be permitted, provided they have been determined from testing in accordance with ACI 355.2 or ACI 355.4. For anchors with a deeper embedment (h_{ef} > **11 in.**), test evidence indicates the use of $h_{ef}^{1.5}$ can be overly conservative for some cases. An alternative expression (Eq. (E-7)) is provided using $h_{ef}^{5/3}$ for evaluation of cast-in headed studs and headed bolts with **11 in.** $\leq h_{ef} \leq$ **25 in.** This expression can also be appropriate for some undercut post-installed anchors. However, for such anchors, the use of Eq. (E-7) should be justified by test results in accordance with E.4.2. Experimental and numerical investigation indicate that Eq. (E-7) may be unconservative for h_{ef} > **25 in.**, where bearing pressure on the anchor head is at or near the limit permitted by Eq. (E-14) (Ožbolt et al. 2007).

RE.5.2.3 For anchors located less than $1.5h_{ef}$ from three or more edges, the tensile breakout strength computed by the CCD Method, which is the basis for Eq. (E-3) to (E-10), gives overly conservative results (Lutz 1995). This occurs because the ordinary definitions of A_{Nc}/A_{Nco} do not correctly reflect the edge effects. This problem is corrected by limiting the value of h_{ef} used in Eq. (E-3) through (E-10) to $c_{a,max}/1.5$, where $c_{a,max}$ is the largest of the influencing edge distances that are less than or equal to the actual $1.5h_{ef}$. In no case should $c_{a,max}/1.5$ be taken less than one-third of the maximum spacing between anchors within the group. The limit on h_{ef} of at least one-third of the maximum spacing between anchors within the group prevents the use of a calculated strength based on individual breakout prisms for a group anchor configuration.

This approach is illustrated in Fig. RE.5.2.3. In this example, the proposed limit on the value of h_{ef} to be used in the computations where $h_{ef} = c_{a,max}/1.5$, results in $h_{ef} = h_{ef}'$ = **4 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.

The requirement of E.5.2.3 may be visualized by moving the actual concrete breakout surface, which originates at the actual h_{ef}, toward the surface of the concrete parallel to the applied tension load. The value of h_{ef} used in Eq. (E-3) to (E-10) is determined when either: (a) the outer boundaries of the failure surface first intersect a free edge; or (b) the

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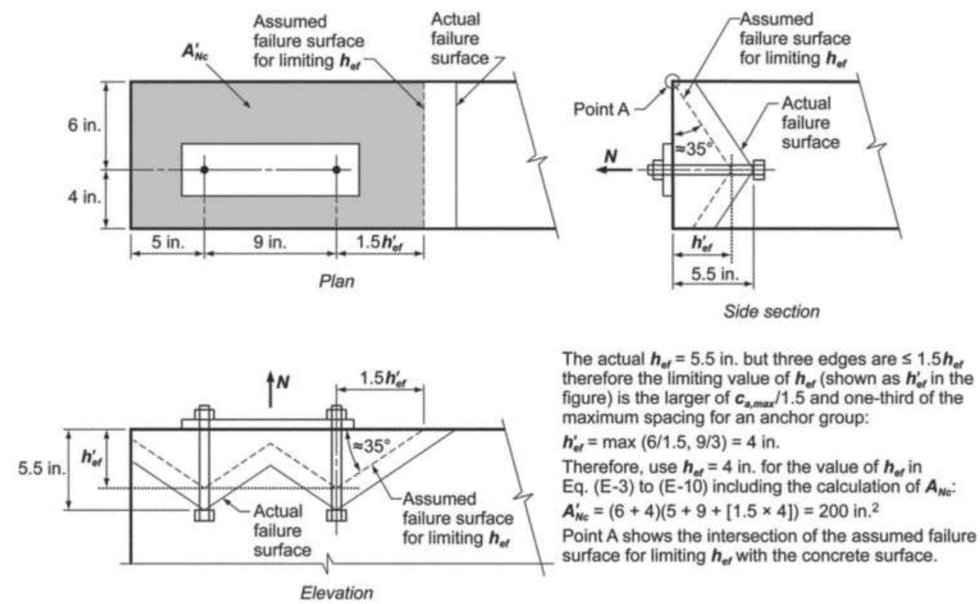

Fig. RE.5.2.3—Example of tension where anchors are located in narrow members.

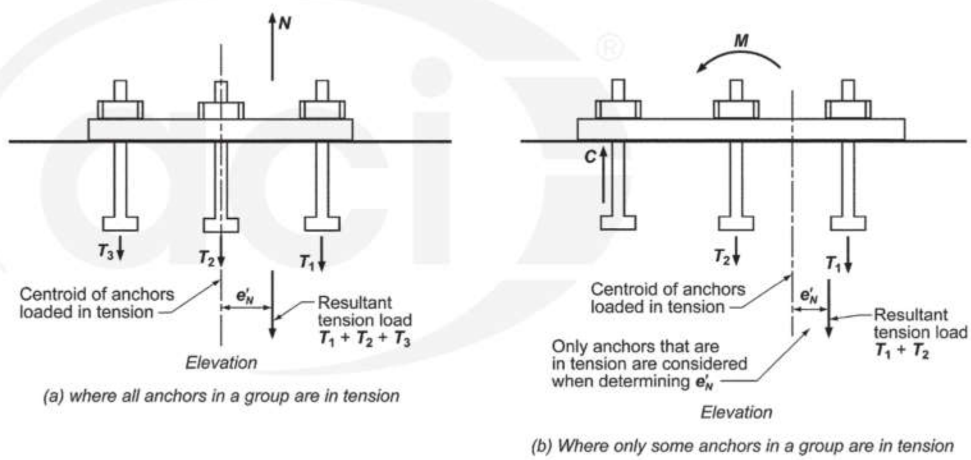

Fig. RE.5.2.4—Definition of $\mathbf{e}_N'$ *for a group of anchors.*

intersection of the breakout surface between anchors within the group first intersects the surface of the concrete. For the example shown in Fig. RE.5.2.3, Point "A" defines the intersection of the assumed failure surface for limiting h_{ef} with the concrete surface.

E.5.2.4 The modification factor for anchor groups loaded eccentrically in tension, $\psi_{ec,N}$, shall be computed as

$$\psi_{ec,N} = \frac{1}{\left(1+\frac{2e'_N}{3h_{ef}}\right)} \quad \text{(E-8)}$$

but $\psi_{ec,N}$ shall not be taken greater than 1.0. 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. (E-8) and for the calculation of N_{cbg} according to Eq. (E-4).

RE.5.2.4 Figure RE.5.2.4(a) shows a group of anchors that are all in tension but the resultant force is eccentric with respect to the centroid of the anchor group. Groups of anchors can also be loaded in such a way that only some of the anchors are in tension (Fig. RE.5.2.4(b)). 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.

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In the case where eccentric loading exists about two axes, the modification factor, $\psi_{ec,N}$, shall be computed for each axis individually and the product of these factors used as $\psi_{ec,N}$ in Eq. (E-4).

E.5.2.5 The modification factor for edge effects for single anchors or anchor groups loaded in tension, $\psi_{ed,N}$, shall be computed as

If $c_{a,min} \geq 1.5h_{ef}$, then

$$\psi_{ed,N} = 1.0 \tag{E-9}$$

If $c_{a,min} < 1.5h_{ef}$, then

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \tag{E-10}$$

E.5.2.6 For anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, the following modification factor shall be permitted:

$\psi_{c,N}$ = **1.25** for cast-in anchors; and

$\psi_{c,N}$ = **1.4** for post-installed anchors, where the value of k_c used in Eq. (E-6) is 17.

Where the value of k_c used in Eq. (E-6) is taken from the ACI 355.2 or ACI 355.4 product evaluation report for post-installed anchors qualified for use in both cracked and uncracked concrete, the values of k_c and $\psi_{c,N}$ shall be based on the ACI 355.2 or ACI 355.4 product evaluation report.

Where the value of k_c used in Eq. (E-6) is taken from the ACI 355.2 or ACI 355.4 product evaluation report for post-installed anchors qualified for use in uncracked concrete, $\psi_{c,N}$ shall be taken as 1.0.

When analysis indicates cracking at service load levels, $\psi_{c,N}$ 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 or ACI 355.4. 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.

E.5.2.7 The modification factor for post-installed anchors designed for uncracked concrete in accordance with E.5.2.6 without supplementary reinforcement to control splitting, $\psi_{cp,N}$, shall be computed as follows using the critical distance c_{ac} as defined in E.8.6

If $c_{a,min} \geq c_{ac}$, then

$$\psi_{cp,N} = 1.0 \tag{E-11}$$

If $c_{a,min} < c_{ac}$, then

$$\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} \tag{E-12}$$

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RE.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 strength of the anchor is further reduced beyond that reflected in A_{Nc}/A_{Nco}. If the smallest side cover distance is greater than or equal to $1.5h_{ef}$, a complete prism can form and there is no reduction ($\psi_{ed,N}$ = **1**). If the side cover is less than $1.5h_{ef}$, the factor $\psi_{ed,N}$ is required to adjust for the edge effect (Fuchs et al. 1995).

RE.5.2.6 Post-installed anchors that have not met the requirements for use in cracked concrete according to ACI 355.2 or ACI 355.4 should be used only in regions that will remain uncracked. The analysis for the determination of crack formation should include the effects of restrained shrinkage (refer to 12.13.1.2). The anchor qualification tests of ACI 355.2 or ACI 355.4 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. (E-6) and (E-7) assume cracked concrete (that is, $\psi_{c,N}$ = **1.0**) with $\psi_{c,N}k_c$ = **24** for cast-in-place, and 17 for post-installed (cast-in 40 percent higher). When the uncracked concrete $\psi_{c,N}$ factors are applied (1.25 for cast-in, and 1.4 for post-installed), the results are $\psi_{c,N}k_c$ 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.

RE.5.2.7 The design provisions in E.5 are based on the assumption that the basic concrete breakout strength can be achieved if the minimum edge distance, $c_{a,min}$, equals $1.5h_{ef}$. However, test results (Asmus 1999) indicate that many torque-controlled and displacement-controlled expansion anchors and some undercut anchors require minimum edge distances exceeding $1.5h_{ef}$ to achieve the basic concrete breakout strength when tested in uncracked concrete without supplementary reinforcement to control splitting. When a tension load is applied, the resulting tensile stresses at the embedded end of the anchor are added to the tensile stresses induced due to anchor installation, and splitting failure may occur before reaching the concrete breakout strength defined in E.5.2.1. To account for this potential splitting mode of failure, the basic concrete breakout strength is reduced by

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but $\psi_{cp,N}$ determined from Eq. (E-12) shall not be taken less than $1.5h_{ef}/c_{ac}$, where the critical distance c_{ac} is defined in E.8.6.

For all other cases, including cast-in anchors, $\psi_{cp,N}$ shall be taken as 1.0.

E.5.2.8 Where 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 the thickness of the washer or plate from the outer edge of the head of the anchor.

E.5.2.9 Where anchor reinforcement is developed in accordance with Chapter 12 on both sides of the breakout surface, the design strength of the anchor reinforcement shall be permitted to be used instead of the concrete breakout strength in determining ϕN_n. A strength reduction factor of 0.75 shall be used in the design of the anchor reinforcement.

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a factor $\psi_{cp,N}$ if $c_{a,min}$ is less than the critical edge distance c_{ac}. If supplementary reinforcement to control splitting is present or if the anchors are located in a region where analysis indicates cracking of the concrete at service loads, then the reduction factor $\psi_{cp,N}$ is taken as 1.0. The presence of supplementary reinforcement to control splitting does not affect the selection of Condition A or B in D.4.3 or D.4.4.

RE.5.2.9 For conditions where the factored tensile force exceeds the concrete breakout strength of the anchor(s) or where the breakout strength is not evaluated, the nominal strength can be that of anchor reinforcement properly anchored as illustrated in Fig. RE.5.2.9. Care needs to be taken in the selection and positioning of the anchor reinforcement. The anchor reinforcement should consist of

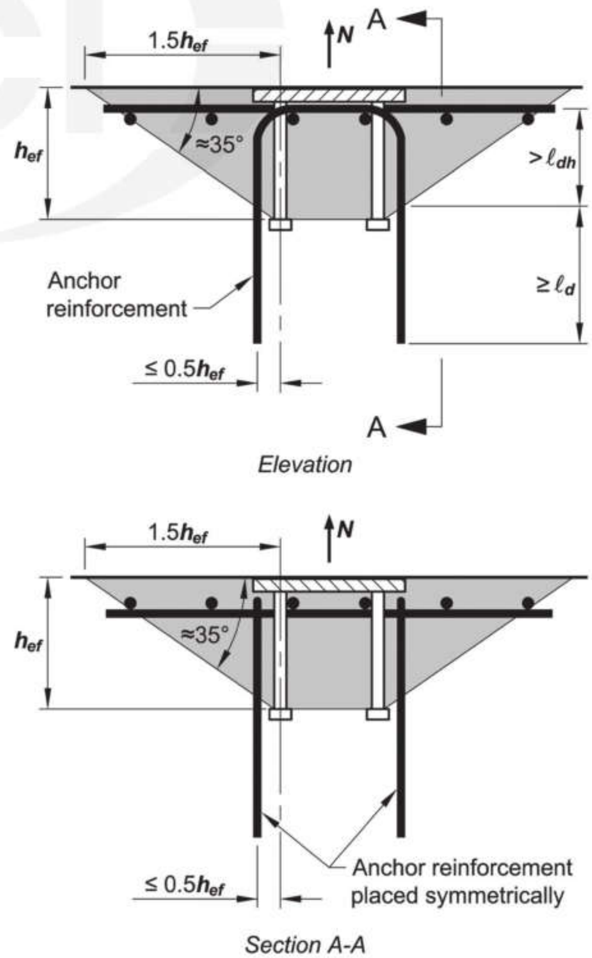

Fig. RE.5.2.9—Anchor reinforcement for tension.

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stirrups, ties, or hairpins placed as close as practicable to the anchor. Only reinforcement spaced less than $0.5h_{ef}$ from the anchor centerline should be included as anchor reinforcement. The research (Eligehausen et al. 2006a) on which these provisions is based was limited to anchor reinforcement with maximum diameter similar to a No. 5 bar. It is beneficial for the anchor reinforcement to enclose the surface reinforcement. In sizing the anchor reinforcement, use of a 0.75 strength reduction factor ϕ is recommended as is used for strut-and-tie models. If the alternate load factors of Appendix D are used, the corresponding strength reduction factor of 0.85 for strut-and-tie models should be used. As a practical matter, use of anchor reinforcement is generally limited to cast-in-place anchors.

E.5.3 *Pullout strength of cast-in, post-installed expansion and undercut anchors in tension*

E.5.3.1 The nominal pullout strength of a single cast-in, post-installed expansion, and post-installed undercut anchor in tension, N_{pn}, shall not exceed

$$N_{pn} = \psi_{c,P} N_p \qquad \text{(E-13)}$$

where $\psi_{c,P}$ is defined in E.5.3.6.

E.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.

E.5.3.3 For single cast-in headed studs and headed bolts, it shall be permitted to evaluate the pullout strength in tension using E.5.3.4. For single J- or L-bolts, it shall be permitted to evaluate the pullout strength in tension using E.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.

E.5.3.4 The pullout strength in tension of a single headed stud or headed bolt, N_p, for use in Eq. (E-13), shall not exceed

$$N_p = 8A_{brg}f_c' \qquad \text{(E-14)}$$

E.5.3.5 The pullout strength in tension of a single hooked bolt, N_p, for use in Eq. (E-13) shall not exceed

$$N_p = 0.9f_c'e_hd_a \qquad \text{(E-15)}$$

where $3d_a \leq e_h \leq 4.5d_a$.

RE.5.3 *Pullout strength of cast-in, post-installed expansion and undercut anchors in tension*

RE.5.3.1 The design requirements for pullout are applicable to cast-in, post-installed expansion, and post-installed undercut anchors. They are not applicable to adhesive anchors, which are instead evaluated for bond failure in accordance with E.5.5.

RE.5.3.2 The pullout strength equations given in E.5.3.4 and E.5.3.5 are only applicable to cast-in headed and hooked anchors (CEB 1997; Kuhn and Shaikh 1996); 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.

RE.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.

RE.5.3.4 The value computed from Eq. (E-14) corresponds to the load at which crushing of the concrete occurs due to bearing of the anchor head (CEB 1997; ACI 349). 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. Local crushing of the concrete greatly reduces the stiffness of the connection, and generally will be the beginning of a pullout failure.

RE.5.3.5 Equation (E-15) for hooked bolts was developed by Lutz based on the results of Asmus (1999). 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

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E.5.3.6 For an anchor located in a region of a concrete member where analysis indicates no cracking at service load levels, the following modification factor shall be permitted

$$\psi_{c,P} = 1.4$$

Where analysis indicates cracking at service load levels, $\boldsymbol{\psi_{c,P}}$ shall be taken as 1.0.

E.5.4—Concrete side-face blowout strength of a headed anchor in tension

E.5.4.1 For a single headed anchor with deep embedment close to an edge ($\boldsymbol{h_{ef} > 2.5c_{a1}}$), the nominal side-face blowout strength, $\boldsymbol{N_{sb}}$, shall not exceed

$$N_{sb} = (160c_{a1}\sqrt{A_{brg}})\lambda_a\sqrt{f_c'} \qquad \text{(E-16)}$$

If $\boldsymbol{c_{a2}}$ for the single headed anchor is less than $\boldsymbol{3c_{a1}}$, the value of $\boldsymbol{N_{sb}}$ shall be multiplied by the factor $\boldsymbol{(1 + c_{a2}/c_{a1})/4}$ where $\boldsymbol{1.0 \leq c_{a2}/c_{a1} \leq 3.0}$.

E.5.4.2 For multiple headed anchors with deep embedment close to an edge ($\boldsymbol{h_{ef} > 2.5c_{a1}}$) and anchor spacing less than $\boldsymbol{6c_{a1}}$, the nominal strength of those anchors susceptible to a side-face blowout failure $\boldsymbol{N_{sbg}}$ shall not exceed

$$N_{sbg} = \left(1 + \frac{s}{6c_{a1}}\right)N_{sb} \qquad \text{(E-17)}$$

where $\boldsymbol{s}$ is the distance between the outer anchors along the edge, and $\boldsymbol{N_{sb}}$ is obtained from Eq. (E-16) without modification for a perpendicular edge distance.

E.5.5 *Bond strength of adhesive anchor in tension*

E.5.5.1 The nominal bond strength in tension—$\boldsymbol{N_a}$ of a single adhesive anchor or $\boldsymbol{N_{ag}}$ of a group of adhesive anchors—shall not exceed

(a) For a single adhesive anchor:

$$N_a = \frac{A_{Na}}{A_{Nao}}\psi_{ed,Na}\psi_{cp,Na}N_{ba} \qquad \text{(E-18)}$$

(b) For a group of adhesive anchors:

$$N_{ag} = \frac{A_{Na}}{A_{Nao}}\psi_{ec,Na}\psi_{ed,Na}\psi_{cp,Na}N_{ba} \qquad \text{(E-19)}$$

Factors $\boldsymbol{\psi_{ec,Na}}$, $\boldsymbol{\psi_{ed,Na}}$, and $\boldsymbol{\psi_{cp,Na}}$ are defined in E.5.5.3, E.5.5.4, and E5.5.5, respectively. $\boldsymbol{A_{Na}}$ is the projected influ-

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$\boldsymbol{e_h}$ are based on the range of variables used in the three tests programs reported in Kuhn and Shaikh (1996).

RE.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 Furche and Eligehausen (1991). 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.

RE.5.4.2 In determining nominal side-face blowout strength for multiple headed anchors, only those anchors close to an edge ($\boldsymbol{h_{ef} > 2.5c_{a1}}$) that are loaded in tension should be considered. Their strength should be compared to the proportion of the tensile load applied to those anchors.

RE.5.5 *Bond strength of adhesive anchor in tension*

RE.5.5.1 Evaluation of bond strength applies only to adhesive anchors. Single anchors with small embedment loaded to failure in tension may exhibit concrete breakout failures, while deeper embedments produce bond failures. Adhesive anchors that exhibit bond failures when loaded individually may exhibit concrete failures when in a group or in a near-edge condition. In all cases, the strength in tension of adhesive anchors is limited by the concrete breakout strength as given by Eq. (E-3) and (E-4) (Eligehausen et al. 2006a). The influences of anchor spacing and edge distance on both bond strength and concrete breakout strength must be evaluated for adhesive anchors. The influences of anchor spacing and edge distance on the nominal bond strength of adhesive

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ence area of a single adhesive anchor or group of adhesive anchors that shall be approximated as a rectilinear area that projects outward a distance c_{Na} from the centerline of the adhesive anchor, or in the case of a group of adhesive anchors, from a line through a row of adjacent adhesive anchors. A_{Na} shall not exceed nA_{Nao}, where n is the number of adhesive anchors in the group that resist tension loads. A_{Nao} is the projected influence area of a single adhesive anchor with an edge distance equal to or greater than c_{Na}:

$$A_{Nao} = (2c_{Na})^2 \tag{E-20}$$

where

$$c_{Na} = 10d_a\sqrt{\frac{\tau_{uncr}}{1100}} \tag{E-21}$$

and constant 1100 carries the unit of lb/in.^{2}

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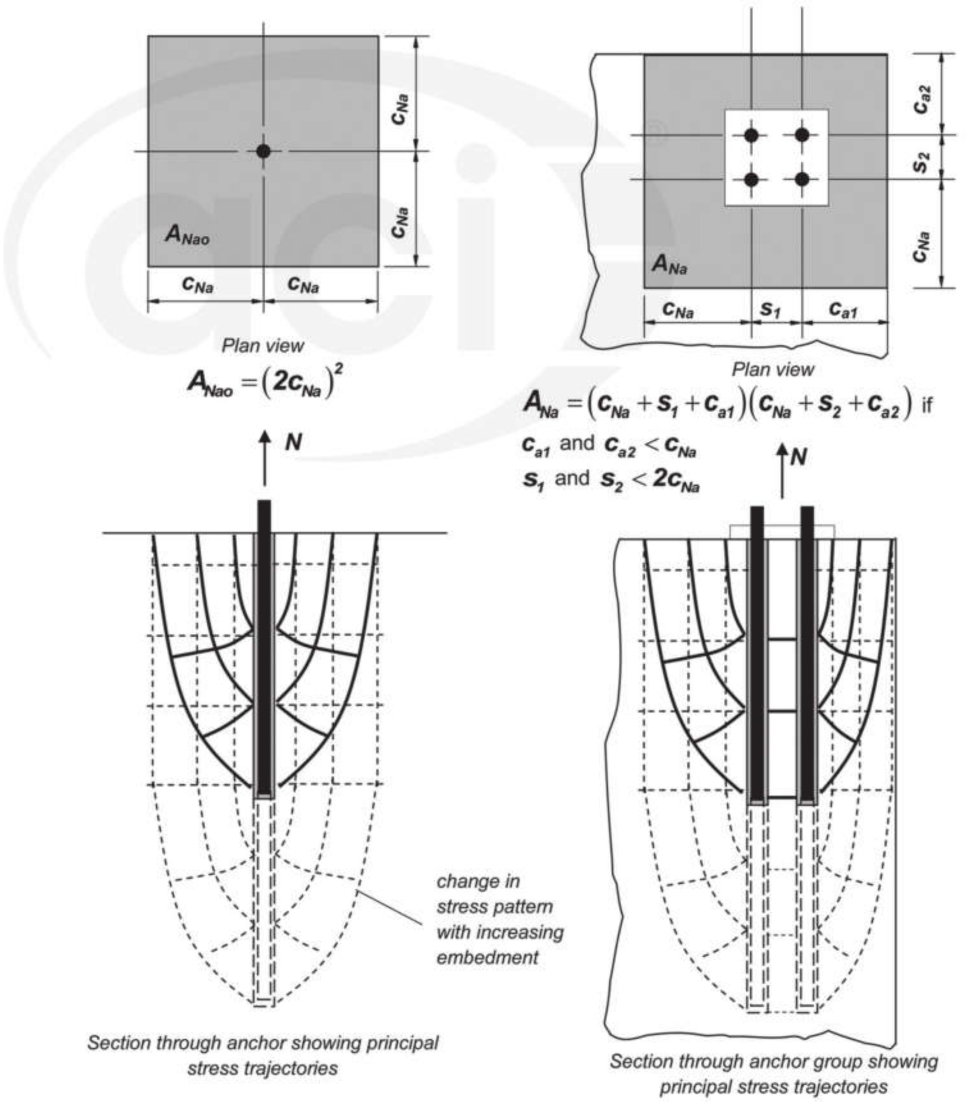
anchors in tension are included in the modification factors A_{Na}/A_{Nao} and $\psi_{ed,Na}$ in Eq. (E-18) and (E-19).

The influence of nearby edges and adjacent loaded anchors on bond strength is dependent on the volume of concrete mobilized by a single adhesive anchor. In contrast to the projected concrete failure area concept used in Eq. (E-3) and (E-4) to compute the breakout strength of an adhesive anchor, the influence area associated with the bond strength of an adhesive anchor used in Eq. (E-18) and (E-19) is not a function of the embedment depth but rather a function of the anchor diameter and the characteristic bond stress. The critical distance c_{Na} is assumed the same whether the concrete is cracked or uncracked; for simplicity, the relationship for c_{Na} in Eq. (E-21) uses τ_{uncr}. This has been verified by experimental and numerical studies (Eligehausen et al. 2006a). Figure RE.5.5.1(a) shows A_{Nao} and the development of Eq. (E-20). A_{Nao} is the projected influence area



(a) Single adhesive anchor away from edges and other anchors

(b) Group of four adhesive anchors located near a corner

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Fig. RE.5.5.1—Calculation of influence areas $\mathbf{A_{Nao}}$ *and* $\mathbf{A_{Na}}$.

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for the bond strength of a single adhesive anchor. Figure RE.5.5.1(b) shows an example of the projected influence area for an anchor group. Because in this case, A_{Na} is the projected influence area for a group of anchors and A_{Nao} is the projected influence area for a single anchor, there is no need to include n, the number of anchors, in Eq. (E-19). If anchors in a group (anchors loaded by a common base plate or attachment) are positioned in such a way that the projected influence areas of the individual anchors overlap, the value of A_{Na} is less than nA_{Nao}.

The tensile strength of closely spaced adhesive anchors with low bond strength may significantly exceed the value given by Eq. (E-19). A correction factor is given in the literature (Eligehausen et al. 2006a) to address this issue, but for simplicity, this factor is not included in the Code.

E.5.5.2 The basic bond strength of a single adhesive anchor in tension in cracked concrete, N_{ba}, shall not exceed

$$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef} \tag{E-22}$$

The characteristic bond stress τ_{cr} shall be taken as the 5 percent fractile of results of tests performed and evaluated according to ACI 355.4.

Where analysis indicates cracking at service load levels, adhesive anchors shall be qualified for use in cracked concrete in accordance with ACI 355.4.

For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} shall be permitted to be used in place of τ_{cr} in Eq. (E-22) and shall be taken as the 5 percent fractile of results of tests performed and evaluated according to ACI 355.4.

It shall be permitted to use the minimum characteristic bond stress values in Table E.5.5.2, provided (a) through (e) are satisfied:

(a) Anchors shall meet the requirements of ACI 355.4
(b) Anchors shall be installed in holes drilled with a rotary impact drill or rock drill
(c) Concrete at time of anchor installation shall have a minimum compressive strength of 2500 psi
(d) Concrete at time of anchor installation shall have a minimum age of 21 days
(e) Concrete temperature at time of anchor installation shall be at least 50°F

Table E.5.5.2—Minimum characteristic bond stresses[*†]

Installation and service environment	Moisture content of concrete at time of anchor installation	Peak in-service temperature of concrete, °F	τ_{cr}, psi	τ_{uncr}, psi
Outdoor	Dry to fully saturated	175	200	650
Indoor	Dry	110	300	1000

[*]Where anchor design includes sustained tension loading, multiply values of τ_{cr} and τ_{uncr} by 0.4.

[†]Where anchor design includes earthquake loads for structures assigned to Seismic Design Category C, D, E, or F, multiply values of τ_{cr} by 0.8 and τ_{uncr} by 0.4.

RE.5.5.2 The equation for basic bond strength of adhesive anchors as given in Eq. (E-22) represents a uniform bond stress model that has been shown to provide the best prediction of adhesive anchor bond strength through numerical studies and comparisons of different models to an international database of experimental results (Cook et al. 1998). The basic bond strength is valid for bond failures that occur between the concrete and the adhesive as well as between the anchor and the adhesive.

Characteristic bond stresses should be based on tests performed in accordance with ACI 355.4 and should reflect the particular combination of installation and use conditions anticipated during construction and during the anchor service life. For those cases where product-specific information is unavailable at the time of design, Table E.5.5.2 provides lower-bound default values. The characteristic bond stresses in Table E.5.5.2 are the minimum values permitted for adhesive anchor systems qualified in accordance with ACI 355.4 for the tabulated installation and use conditions. Use of these values is restricted to the combinations of specific conditions listed; values for other combinations of installation and use conditions should not be inferred. Where both sustained loading and earthquake loading are present, the applicable factors given in the footnotes of Table E.5.5.2 should be multiplied together. The table assumes that all concrete has a minimum age of 21 days and a minimum concrete compressive strength of 2500 psi. Refer to RE.2.2.

The terms "indoor" and "outdoor" as used in Table E.5.5.2 refer to a specific set of installation and service environments. Indoor conditions represent anchors installed in dry concrete with a rotary impact drill or rock drill and subjected to limited concrete temperature variations over the service life of the anchor. Outdoor conditions are assumed to occur, when at the time of anchor installation, the concrete is exposed to weather and may therefore be wet. Anchors installed in outdoor conditions are also assumed to be subject to greater concrete temperature variations such as might be associated with freezing and thawing or elevated temperatures resulting from direct sun exposure. While the indoor/outdoor characterization is useful for many applica-

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tions, there may be situations in which a literal interpretation of the terms "indoor" and "outdoor" do not apply. For example, anchors installed before the building envelope is completed may involve drilling in saturated concrete. As such, the minimum characteristic bond stress associated with the outdoor condition in Table E.5.5.2 applies, regardless of whether the service environment is "indoor" and "outdoor." Rotary impact drills and rock drills produce non-uniform hole geometries that are generally favorable for bond. Installation of adhesive anchors in core drilled holes may result in substantially lower characteristic bond stresses. Because this effect is highly product-dependent, design of anchors to be installed in core drilled holes should adhere to the product-specific characteristic bond stresses established through testing in accordance with ACI 355.4.

The characteristic bond stresses associated with specific adhesive anchor systems are dependent on numerous parameters. Consequently, care should be taken to include all parameters relevant to the value of characteristic bond stress used in the design. These parameters include but are not limited to:

1. Type and duration of loading—bond strength is reduced for sustained tension loading
2. Concrete cracking—bond strength is higher in uncracked concrete
3. Anchor size—bond strength is generally inversely proportional to anchor diameter
4. Drilling method—bond strength may be lower for anchors installed in core drilled holes
5. Degree of concrete saturation at time of hole drilling and anchor installation—bond strength may be reduced due to concrete saturation
6. Concrete temperature at time of installation—installation of anchors in cold conditions may result in retarded adhesive cure and reduced bond strength
7. Concrete age at time of installation—installation in early-age concrete may result in reduced bond strength (refer to RE.2.2)
8. Peak concrete temperatures during anchor service life—under specific conditions (for example, anchors in thin concrete members exposed to direct sunlight), elevated concrete temperatures can result in reduced bond strength
9. Chemical exposure—anchors used in industrial environments may be exposed to increased levels of contaminants that can reduce bond strength over time

Anchors tested and assessed under ACI 355.4 may in some cases not be qualified for all the installation and service environments represented in Table E.5.5.2. Therefore, even where the minimum values given in Table E.5.5.2 are used for design, the relevant installation and service environments should be specified in accordance with E.9.2.1 and only anchors that have been qualified under ACI 355.4 for the installation and service environments corresponding to the characteristic bond stress taken from Table E.5.5.2 should be specified.

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Characteristic bond stresses associated with qualified adhesive anchor systems for a specific set of installation and use conditions may substantially exceed the minimum values provided in Table E.5.5.2. For example, 1/2 in. to 3/4 in. diameter anchors installed in impact-drilled holes in dry concrete where use is limited to indoor conditions in uncracked concrete as described above may exhibit characteristic bond stresses τ_{uncr} in the range of 2000 to 2500 psi.

E.5.5.3 The modification factor for adhesive anchor groups loaded eccentrically in tension, $\psi_{ec,Na}$, shall be computed as

$$\psi_{ec,Na} = \frac{1}{\left(1+\frac{e'_N}{c_{Na}}\right)} \tag{E-23}$$

but $\psi_{ec,Na}$ shall not be taken greater than 1.0.

If the loading on an adhesive anchor group is such that only some adhesive anchors are in tension, only those adhesive anchors that are in tension shall be considered when determining the eccentricity e_N' for use in Eq. (E-23) and for the calculation of N_{ag} according to Eq. (E-19).

In the case where eccentric loading exists about two orthogonal axes, the modification factor $\psi_{ec,Na}$ shall be computed for each axis individually and the product of these factors used as $\psi_{ec,Na}$ in Eq. (E-19).

RE.5.5.3 Refer to RE.5.2.4.

E.5.5.4 The modification factor for edge effects for single adhesive anchors or adhesive anchor groups loaded in tension, $\psi_{ed,Na}$, shall be computed as

If $c_{a,min} \geq c_{Na}$, then

$$\psi_{ed,Na} = 1.0 \tag{E-24}$$

If $c_{a,min} < c_{Na}$, then

$$\psi_{ed,Na} = 0.7 + 0.3\frac{c_{a,min}}{c_{Na}} \tag{E-25}$$

RE.5.5.4 If anchors are located close to an edge, their strength is further reduced beyond that reflected in A_{Na}/A_{Nao}. If the smallest side cover distance is greater than or equal to c_{Na}, there is no reduction ($\psi_{ed,Na} = 1$). If the side cover is less than c_{Na}, the factor $\psi_{ed,Na}$ accounts for the edge effect (Fuchs et al. 1995; Eligehausen et al. 2006a).

E.5.5.5 The modification factor for adhesive anchors designed for uncracked concrete in accordance with E.5.5.2 without supplementary reinforcement to control splitting, $\psi_{cp,Na}$, shall be computed as

If $c_{a,min} \geq c_{ac}$, then

$$\psi_{cp,Na} = 1.0 \tag{E-26}$$

If $c_{a,min} < c_{ac}$, then

$$\psi_{cp,Na} = \frac{c_{a,min}}{c_{ac}} \tag{E-27}$$

but $\psi_{cp,Na}$ determined from Eq. (E-27) shall not be taken less than c_{Na}/c_{ac} , where the critical edge distance c_{ac} is defined in E.8.6. For all other cases, $\psi_{cp,Na}$ shall be taken as 1.0.

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E.6—Design requirements for shear loading

E.6.1 *Steel strength of anchor in shear*

E.6.1.1 The nominal strength of an anchor in shear as governed by steel, V_{sa}, shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor. Where concrete breakout is a potential failure mode, the required steel shear strength shall be consistent with the assumed breakout surface.

E.6.1.2 The nominal strength of an anchor in shear, V_{sa}, shall not exceed (a) through (c):

(a) For cast-in headed stud anchor

$$V_{sa} = A_{se,V}f_{uta} \tag{E-28}$$

where $A_{se,V}$ is the effective cross-sectional area of an anchor in shear, in^2, and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 125,000 psi.

(b) For cast-in headed bolt and hooked bolt anchors and for post-installed anchors where sleeves do not extend through the shear plane

$$V_{sa} = 0.6A_{se,V}f_{uta} \tag{E-29}$$

where $A_{se,V}$ is the effective cross-sectional area of an anchor in shear, in.^{2}, and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 125,000 psi.

(c) For post-installed anchors where sleeves extend through the shear plane, V_{sa} shall be based on the results of tests performed and evaluated according to ACI 355.2. Alternatively, Eq. (E-29) shall be permitted to be used.

E.6.1.3 Where anchors are used with built-up grout pads, the nominal strengths of E.6.1.2 shall be multiplied by a 0.80 factor.

E.6.2 *Concrete breakout strength of anchor in shear*

E.6.2.1 The nominal concrete breakout strength in shear, V_{cb} of a single anchor or V_{cbg} of a group of anchors, shall not exceed:

(a) For shear force perpendicular to the edge on a single anchor

$$V_{cb} = \frac{A_{Vc}}{A_{Vco}}\psi_{ed,V}\psi_{c,V}\psi_{h,V}V_b \tag{E-30}$$

(b) For shear force perpendicular to the edge on a group of anchors

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}}\psi_{ec,V}\psi_{ed,V}\psi_{c,V}\psi_{h,V}V_b \tag{E-31}$$

(c) For shear force parallel to an edge, V_{cb} or V_{cbg} shall be permitted to be twice the value of the shear force determined from Eq. (E-30) or (E-31), respectively, with the shear force assumed to act perpendicular to the edge and with $\psi_{ed,V}$ taken equal to 1.0.

COMMENTARY

RE.6—Design requirements for shear loading

RE.6.1 *Steel strength of anchor in shear*

RE.6.1.1 The shear load applied to each anchor in a group may vary depending on assumptions for the concrete breakout surface and load redistribution (refer to RE.6.2.1).

RE.6.1.2 The nominal shear strength of anchors is best represented as a function of f_{uta} rather than f_{ya} because most anchor materials do not exhibit a well-defined yield point. Welded studs develop a higher steel shear strength than headed anchors due to the fixity provided by the weld between the studs and the base plate. The use of Eq. (E-28) and (E-29) with 9.2 load factors and the ϕ-factors of E.4.3 give design strengths consistent with the AISC "Load and Resistance Factor Design Specifications for Structural Steel Buildings."

The limitation of $1.9f_{ya}$ on f_{uta} is to ensure that, under service load conditions, the anchor stress does not exceed f_{ya}. The limit on f_{uta} of $1.9f_{ya}$ was determined by converting the LRFD provisions to corresponding service level conditions as discussed in RE.5.1.2.

For post-installed anchors having a reduced cross-sectional area anywhere along the anchor length, the effective cross-sectional area of the anchor should be provided by the manufacturer. For threaded rods and headed bolts, ANSI/ASME B1.1 defines $A_{se,V}$ as

$$A_{se,V} = \frac{\pi}{4}\left(d_a - \frac{0.9743}{n_t}\right)^2$$

where n_t is the number of threads per inch.

RE.6.2 *Concrete breakout strength of anchor in shear*

RE.6.2.1 The shear strength equations were developed from the CCD Method. They assume a breakout cone angle of approximately 35 degrees (refer to Fig. RE.4.2(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 of A_{Vc}/A_{Vco} in Eq. (E-30) and (E-31), and $\psi_{ec,V}$ in Eq. (E-31). For anchors far from the edge, E.6.2 usually will not govern. For these cases, E.6.1 and E.6.3 often govern.

Figure RE.6.2.1a shows A_{Vco} and the development of Eq. (E-32). A_{Vco} 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 RE.6.2.1b shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. A_{Vc} approximates the full surface area of the breakout cone for the particular arrangement of anchors. Because A_{Vc} is the total projected

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(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.

Factors $\psi_{ec,V}$, $\psi_{ed,V}$, $\psi_{c,V}$, and $\psi_{h,V}$ are defined in E.6.2.5, E.6.2.6, E.6.2.7, and E.6.2.8, respectively. V_b is the basic concrete breakout strength value for a single anchor. A_{Vc} 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 A_{Vc} 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_{a1} shall be taken as the distance from the edge to this axis. A_{Vc} shall not exceed nA_{Vco}, where n is the number of anchors in the group.

A_{Vco} is the projected area for a single anchor in a deep member with a distance from edges equal or greater than $1.5c_{a1}$ in the direction perpendicular to the shear force. It shall be permitted to evaluate A_{Vco} as the base of a half-pyramid with a side length parallel to the edge of $3c_{a1}$ and a depth of $1.5c_{a1}$

$$A_{Vco} = 4.5(c_{a1})^2 \tag{E-32}$$

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_{a1} 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.

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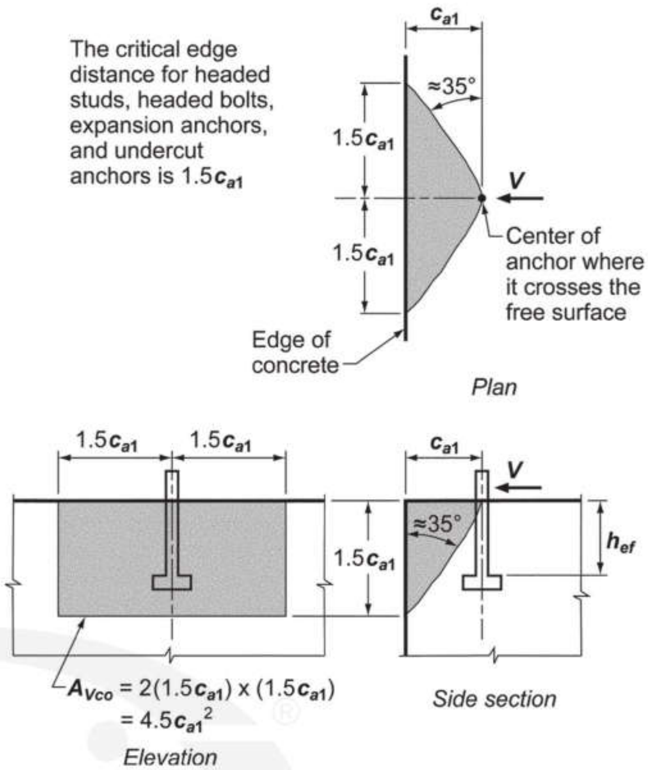

Fig. RE.6.2.1a—Calculation of $\mathbf{A_{Vco}}$.

area for a group of anchors, and A_{Vco} is the area for a single anchor, there is no need to include the number of anchors in the equation.

As shown in the examples in Fig. RE.6.2.1b of two-anchor groups loaded in shear, when using Eq. (E-31) for cases where the anchor spacing s is greater than the edge distance to the near edge anchor $c_{a1,1}$, both assumptions for load distribution illustrated in Cases 1 and 2 should be considered. This is because the anchors nearest to the free edge could fail first or the entire group could fail as a unit with the failure surface originating from the anchors farthest from the edge. For Case 1, the steel shear strength is provided by both anchors. For Case 2, the steel shear strength is provided entirely by the anchor farthest from the edge. No contribution of the anchor near the edge is then considered. In addition, checking the near-edge anchor for concrete breakout under service loads is advisable to preclude undesirable cracking at service conditions. If the anchor spacing s is less than the edge distance to the near-edge anchor, then the failure surfaces may merge (Eligehausen et al. 2006a) and Case 3 of Fig. RE.6.2.1(b) may be taken as a conservative approach.

If the anchors are welded to a common plate (regardless of anchor spacing s), when the anchor nearest the front edge begins to form a failure cone, shear load is transferred to the stiffer and stronger rear anchor. For this reason, only Case 2 need be considered, which is consistent with Section 6.5.5 of the *PCI Design Handbook* (PCI 2010). For determination of steel shear strength, it is conservative to consider only the

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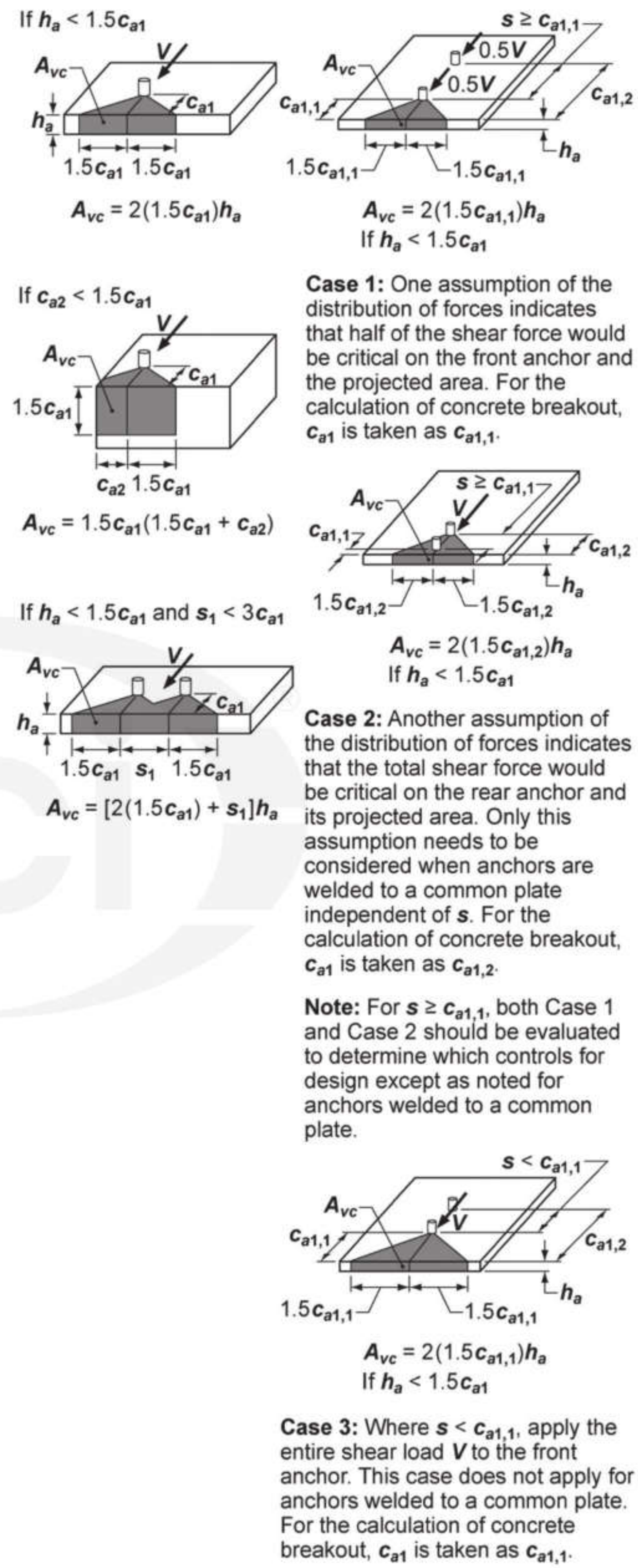

Fig. RE.6.2.1b—Calculation of $\mathbf{A_{vc}}$ *for single anchors and groups of anchors.*

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anchor farthest from the edge. However, for anchors having a ratio of $s/c_{a,1,1}$ less than 0.6, both the front and rear anchors may be assumed to resist the shear (Anderson and Meinheit 2007). For ratios of $s/c_{a,1,1}$ greater than 1, it is advisable to check concrete breakout of the near-edge anchor to preclude undesirable cracking at service conditions.

Further discussion of design for multiple anchors is given in Primavera et al. (1997).

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 Eligehausen et al. (2006a).

The detailed provisions of E.6.2.1(a) apply to the case of shear force directed toward an edge. When the shear force is directed away from the edge, the strength will usually be governed by E.6.1 or E.6.3.

The case of shear force parallel to an edge is shown in Fig. RE.6.2.1c. The maximum shear force that can be applied parallel to the edge, $V_{\parallel}$, as governed by concrete breakout, is twice the maximum shear force that can be applied perpendicular to the edge, $V_{\perp}$. 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 (refer to Fig. RE.6.2.1d), the provisions for shear force applied perpendicular to the edge should be checked in addition to the provisions for shear force applied parallel to the edge.

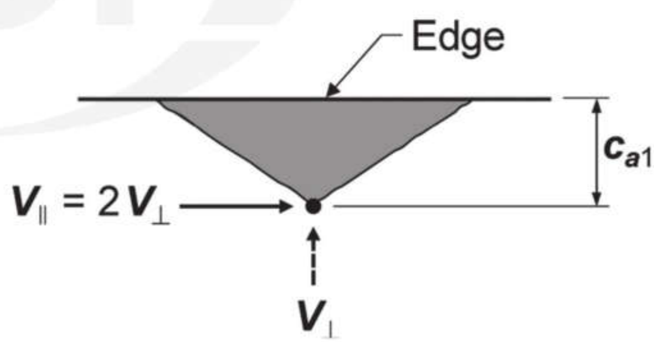

Fig. RE.6.2.1c—Shear force parallel to an edge.

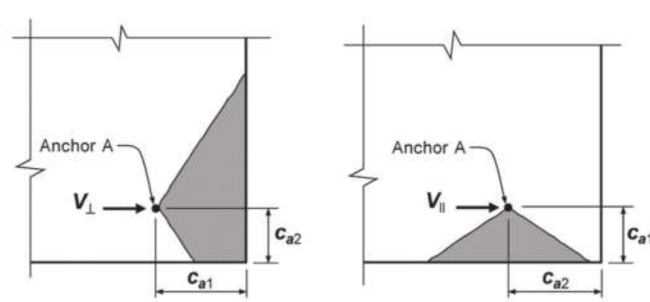

Fig. RE.6.2.1d—Shear force near a corner.

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E.6.2.2 The basic concrete breakout strength in shear of a single anchor in cracked concrete, V_b, shall be the smaller of (a) and (b):

(a)

$$V_b = \left(7\left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda_a\sqrt{f_c'}(c_{a1})^{1.5} \quad \text{(E-33)}$$

where ℓ_e is the load-bearing length of the anchor for shear:

$\ell_e = h_{ef}$ for anchors with a constant stiffness over the full length of embedded section, such as headed studs and post-installed anchors with one tubular shell over full length of the embedment depth,

$\ell_e = 2d_a$ for torque-controlled expansion anchors with a distance sleeve separated from expansion sleeve,

and

$\ell_e \leq 8d_a$ in all cases.

(b)

$$V_b = 9\lambda_a\sqrt{f_c'}\,(c_{a1})^{1.5} \quad \text{(E-34)}$$

E.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. and half of the anchor diameter, the basic concrete breakout strength in shear of a single anchor in cracked concrete, V_b, shall be the smaller of Eq. (E-34) and Eq. (E-35)

$$V_b = \left(8\left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda_a\sqrt{f_c'}(c_{a1})^{1.5} \quad \text{(E-35)}$$

where ℓ_e is defined in E.6.2.2

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) anchor spacing s is not less than 2.5 in.

(c) reinforcement is provided at the corners if $c_{a2} \leq 1.5h_{ef}$.

E.6.2.4 Where anchors are located in narrow sections of limited thickness such that both edge distances c_{a2} and thickness h_a are less than $1.5c_{a1}$, the value of c_{a1} used for the calculation of A_{Vc} in accordance with E.6.2.1 as well as in in Eq. (E-32) through (E-39) shall not exceed the largest of:

(a) $c_{a2}/1.5$, where c_{a2} is the largest edge distance

(b) $h_a/1.5$

(c) $s/3$, where s is the maximum spacing perpendicular to direction of shear, between anchors within a group

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RE.6.2.2 Like the concrete breakout tensile strength, the concrete breakout shear strength does not increase with the failure surface, which is proportional to $(c_{a1})^2$. Instead, the strength increases proportionally to $(c_{a1})^{1.5}$ due to size effect. The strength is also influenced by the anchor stiffness and the anchor diameter (Fuchs et al. 1995; Eligehausen and Balogh 1995; Eligehausen et al. 1987, 1988; Eligehausen et al. 2006b). The influence of anchor stiffness and diameter is not apparent in large-diameter anchors (Lee et al. 2010), resulting in a limitation on the shear breakout strength provided by Eq. (E-34).

The constant, 7, in the shear strength equation was determined from test data reported in Fuchs et al. (1995) at the 5 percent fractile adjusted for cracking.

RE.6.2.3 For the case of cast-in headed bolts continuously welded to an attachment, test data (Shaikh and Lee 1985) show that somewhat higher shear strength 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 but the upper limit of Eq. (E-34) is imposed because tests on large-diameter anchors welded to steel attachments are not available to justify any higher value than Eq. (E-34). The design of supplementary reinforcement is discussed in CEB (1997), Elighausen et al. (1987, 1988), and Eligehausen and Fuchs (1988).

RE.6.2.4 For the case of anchors located in narrow sections of limited thickness where the edge distances perpendicular to the direction of load and the member thickness are less than $1.5c_{a1}$, the shear breakout strength computed by the basic CCD Method is overly conservative. These cases were studied for the κ Method (Eligehausen and Fuchs 1988) and the problem was pointed out by Lutz (1995). Similar to the approach used for concrete breakout strength in tension in E.5.2.3, the concrete breakout strength in shear for this case is more accurately evaluated if the value of c_{a1} used in Eq. (E-30) to (E-39) and in the calculation of A_{vc} is limited to the maximum of two-thirds of the larger of the two edge distances perpendicular to the direction of shear, two-thirds of the member thickness, and one-third of the maximum spacing between anchors within the group, measured perpendicular to the direction of shear. The limit on c_{a1} of at least one-third of the maximum spacing between anchors within

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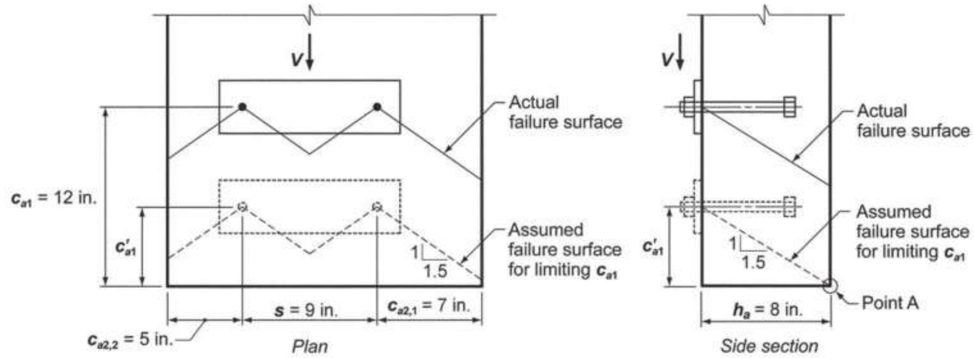

1. The actual c_{a1} = 12 in.

2. The two edge distances c_{a2} as well as h_a are all less than $1.5c_{a1}$.

3. The limiting value of c_{a1} (shown as c'_{a1} in the figure) to be used for the calculation of A_{Vc} and in Eq. (E-30) to (E-39) is determined as the largest of the following:

$(c_{a2,max})/1.5 = (7)/1.5 = 4.67$ in.

$(h_a)/1.5 = (8)/1.5 = 5.33$ in. (controls)

$s/3 = 1/3(9) = 3$ in.

4. For this case, A_{Vc}, A_{Vco}, $\psi_{ed,V}$, and $\psi_{h,V}$ are determined as follows:

$A_{Vc} = (5 + 9 + 7)(1.5 \times 5.33) = 168$ in.^{2}

$A_{Vco} = 4.5(5.33)^2 = 128$ in.^{2}

$\psi_{ed,V} = 0.7 + 0.3(5)/5.33 = 0.98$

$\psi_{h,V} = 1.0$ because $c_{a1} = (h_a)/1.5$. Point A shows the intersection of the assumed failure surface with the concrete surface that establishes the limiting value of c_{a1}.

Fig. RE.6.2.4—Example of shear where anchors are located in narrow members of limited thickness.

the group prevents the use of a calculated strength based on individual breakout prisms for a group anchor configuration.

This approach is illustrated in Fig. RE.6.2.4. In this example, the limiting value of c_{a1} is denoted as c_{a1}' and is used for the calculation of A_{vc}, A_{Vco}, $\psi_{ed,V}$, and $\psi_{h,V}$ as well as for V_b (not shown). The requirement of E.6.2.4 may be visualized by moving the actual concrete breakout surface originating at the actual c_a1 toward the surface of the concrete in the direction of the applied shear load. The value of c_{a1} used for the calculation of A_{Vc} and in Eq. (E-30) to (E-39) is determined when either: (a) an outer boundary of the failure surface first intersects the concrete surface; or (b) the intersection of the breakout surface between anchors within the group first intersects the concrete surface. For the example shown in Fig. RE.6.2.4, Point "A" shows the intersection of the assumed failure surface for limiting c_{a1} with the concrete surface.

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E.6.2.5 The modification factor for anchor groups loaded eccentrically in shear, $\psi_{ec,V}$, shall be computed as

$$\psi_{ec,V} = \frac{1}{\left(1+\frac{2e_V'}{3c_{a1}}\right)} \quad \text{(E-36)}$$

but $\psi_{ec,V}$ shall not be taken greater than 1.0.

If the loading on an anchor group is such that only some anchors are loaded in shear in the same direction, only those anchors that are loaded in shear in the same direction shall be considered when determining the eccentricity of e_V' for use in Eq. (E-36) and for the calculation of V_{cbg} according to Eq. (E-31).

E.6.2.6 The modification factor for edge effect for a single anchor or group of anchors loaded in shear, $\psi_{ed,V}$, shall be computed as follows using the smaller value of c_{a2}.

If $c_{a2} \geq 1.5c_{a1}$, then

$$\psi_{ed,V} = 1.0 \quad \text{(E-37)}$$

If $c_{a2} < 1.5c_{a1}$, then

$$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} \quad \text{(E-38)}$$

E.6.2.7 For anchors located in a region of a concrete member where analysis indicates no cracking at service loads, the following modification factor shall be permitted

$$\psi_{c,V} = 1.4$$

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:

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RE.6.2.5 This section provides a modification factor for an eccentric shear force toward an edge on a group of anchors. If the shear force 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 RE.6.2.5 defines the term e_V' for calculating the $\psi_{ec,V}$ modification factor that accounts for the fact that more shear is applied to one anchor than others, tending to split the concrete near an edge.

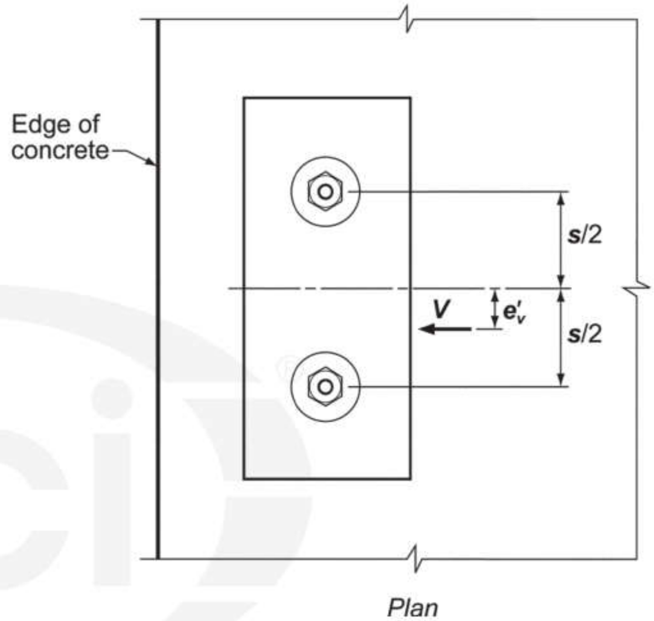

Fig. RE.6.2.5—Definition of $\mathbf{e}_V'$ *for a group of anchors.*

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(a) $\psi_{c,V}$ = **1.0** for anchors in cracked concrete without supplementary reinforcement or with edge reinforcement smaller than a No. 4 bar

(b) $\psi_{c,V}$ = **1.2** for anchors in cracked concrete with reinforcement of a No. 4 bar or greater between the anchor and the edge

(c) $\psi_{c,V}$ = **1.4** for anchors in cracked concrete with reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at not more than 4 in.

E.6.2.8 The modification factor for anchors located in a concrete member where $h_a < 1.5c_{a1}$, $\psi_{h,V}$ shall be computed as

$$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \quad \text{(E-39)}$$

but $\psi_{h,V}$ shall not be taken less than 1.0.

E.6.2.9 Where anchor reinforcement is either developed in accordance with Chapter 12 on both sides of the breakout surface, or encloses the anchor and is developed beyond the breakout surface, the design strength of the anchor reinforcement shall be permitted to be used instead of the concrete breakout strength in determining ϕV_n. A strength reduction factor of 0.75 shall be used in the design of the anchor reinforcement.

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RE.6.2.8 For anchors located in a concrete member where $h_a < 1.5c_{a1}$, tests (CEB 1997; Eligehausen et al. 2006b) have shown that the concrete breakout strength in shear is not directly proportional to the member thickness h_a. The factor $\psi_{h,V}$ accounts for this effect.

RE.6.2.9 For conditions where the factored shear force exceeds the concrete breakout strength of the anchor(s) in shear, or where the breakout strength is not evaluated, the nominal strength can be that of anchor reinforcement properly anchored as shown in Fig. RE.6.2.9a and RE.6.2.9b. To ensure yielding of the anchor reinforcement, the enclosing anchor reinforcement in Fig. RE.6.2.9a should be in contact with the anchor and placed as close as practicable to the concrete surface. The research (Eligehausen et al. 2006b) on which the provisions for enclosing reinforcement (refer to Fig. RE.6.2.9(a)) are based was limited to anchor reinforcement with maximum diameter similar to a No. 5 bar. The larger bend radii associated with larger bar diameters may significantly reduce the effectiveness of the anchor reinforcement and, therefore, anchor reinforcement with a diameter larger than No. 6 is not recommended.

The reinforcement could also consist of stirrups and ties (as well as hairpins) enclosing the edge reinforcement embedded in the breakout cone and placed as close to the anchors as practicable (refer to Fig. RE.6.2.9b). Only reinforcement spaced less than the lesser of $0.5c_{a1}$ and $0.3c_{a2}$ from the anchor centerline should be included as anchor reinforcement. In this case, the anchor reinforcement must be developed on both sides of the breakout surface. For equilibrium reasons, an edge reinforcement must be present. The research on which these provisions are based was limited to anchor reinforcement with maximum diameter similar to a No. 6 bar.

Because the anchor reinforcement is placed below where the shear is applied (refer to Fig. RE.6.2.9b), the force in the anchor reinforcement will be larger than the shear force. In sizing the anchor reinforcement, use of a 0.75 strength reduction factor ϕ is recommended as used for shear and for strut-and-tie models. If the alternate load factors of Appendix D are used, the corresponding strength reduction factor of 0.85 for shear and strut-and-tie models should be used. As a practical matter, the use of anchor reinforcement is generally limited to cast-in-place anchors.

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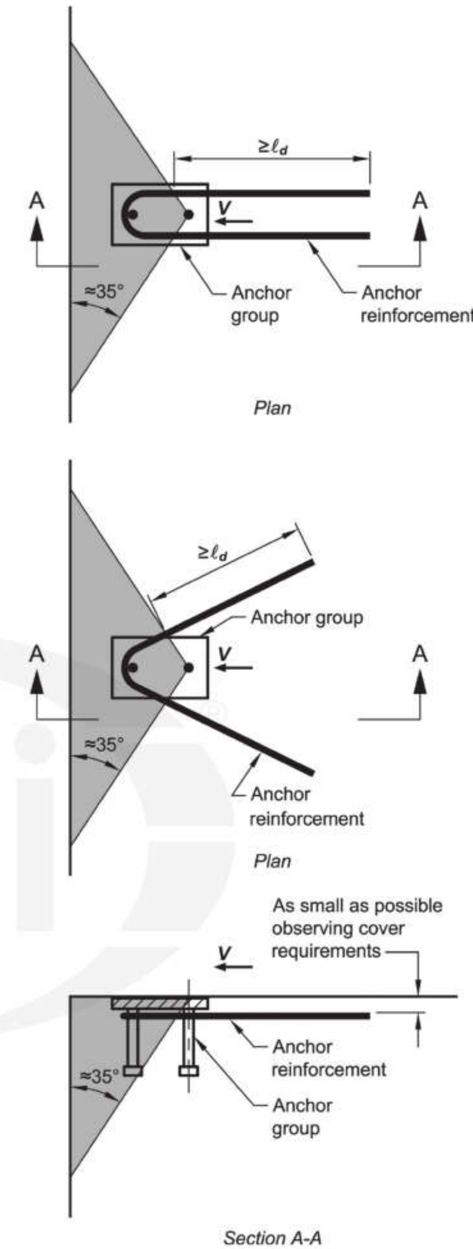

Fig. RE.6.2.9a—Hairpin anchor reinforcement for shear.

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E.6.3 *Concrete pryout strength of anchor in shear*

E.6.3.1 The nominal pryout strength—V_{cp} for a single anchor or V_{cpg} for a group of anchors—shall not exceed:

(a) For a single anchor

$$V_{cp} = k_{cp}N_{cp} \quad \text{(E-40)}$$

For cast-in, expansion, and undercut anchors, N_{cp} shall be taken as N_{cb} determined from Eq. (E-3), and for adhesive anchors, N_{cp} shall be the lesser of N_a determined from Eq. (E-18) and N_{cb} determined from Eq. (E-3).

(b) For a group of anchors

$$V_{cpg} = k_{cp}N_{cpg} \quad \text{(E-41)}$$

For cast-in, expansion, and undercut anchors, N_{cpg} shall be taken as N_{cbg} determined from Eq. (E-4), and for adhe-

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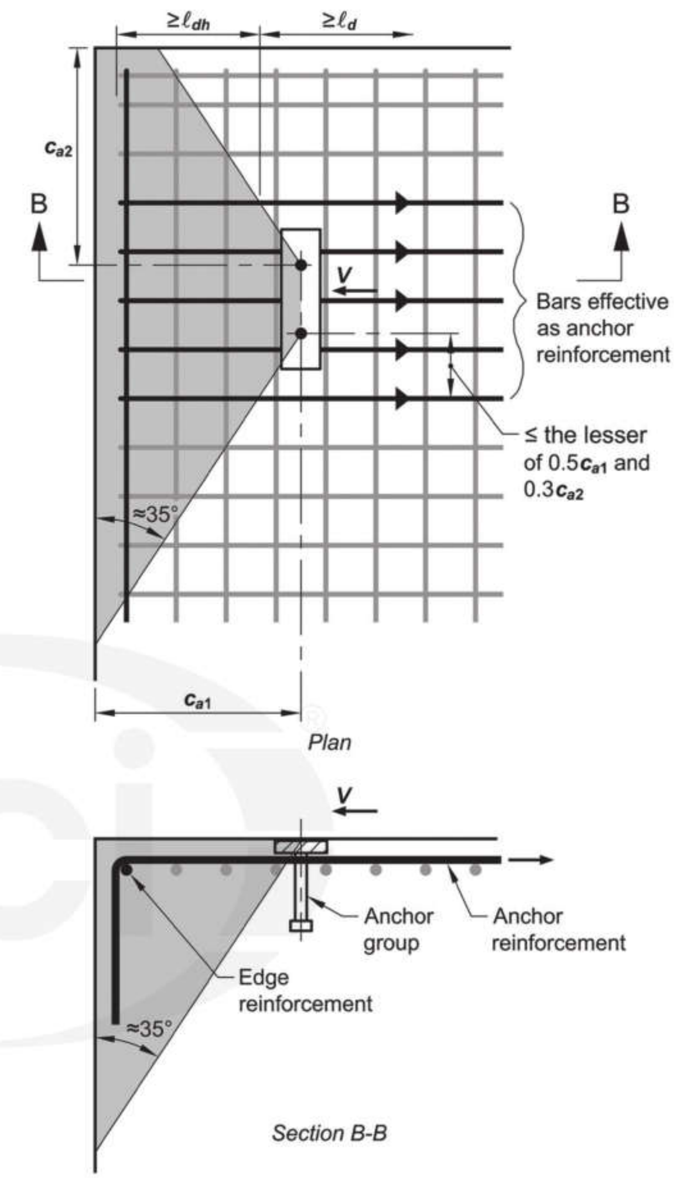

Fig. RE.6.2.9b—Edge reinforcement and anchor reinforcement for shear.

RE.6.3 *Concrete pryout strength of anchor in shear*

RE.6.3.1 Fuchs et al. (1995) 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. Because it is possible that the bond strength of adhesive anchors could be less than the concrete breakout strength, it is necessary to consider both E.5.2.1 and E.5.5.1 for determination of the pryout strength.

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sive anchors, N_{cpg} shall be the lesser of N_{ag} determined from Eq. (E-19) and N_{cbg} determined from Eq. (E-4).

In Eq. (E-40) and (E-41), k_{cp} = **1.0** for h_{ef} < **2.5 in.**, and k_{cp} = **2.0** for $h_{ef} \geq$ **2.5 in.**

E.7—Interaction of tensile and shear forces

Unless determined in accordance with E.4.1.3, anchors or groups of anchors that are subjected to both shear and axial loads shall be designed to satisfy the requirements of E.7.1 through E.7.3. The values of ϕN_n and ϕV_n shall be the required strengths as determined from E.4.1.1 or from E.3.3.

E.7.1 If $V_{ua}/(\phi V_n) \leq 0.2$ for the governing strength in shear, then full strength in tension shall be permitted: $\phi N_n \geq N_{ua}$.

E.7.2 If $N_{ua}/(\phi N_n) \leq 0.2$ for the governing strength in tension, then full strength in shear shall be permitted: $\phi V_n \geq V_{ua}$.

E.7.3 If $V_{ua}/(\phi V_n) > 0.2$ for the governing strength in shear and $N_{ua}/(\phi N_n) > 0.2$ for the governing strength in tension, then

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \tag{E-42}$$

COMMENTARY

RE.7—Interaction of tensile and shear forces

The shear-tension interaction expression has traditionally been expressed as

$$\left(\frac{N_{ua}}{N_n}\right)^{\varsigma} + \left(\frac{V_{ua}}{V_n}\right)^{\varsigma} \leq 1.0$$

where ς varies from 1 to 2. The current trilinear recommendation is a simplification of the expression where ς = **5/3** (Fig. RE.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 E.4.1.3.

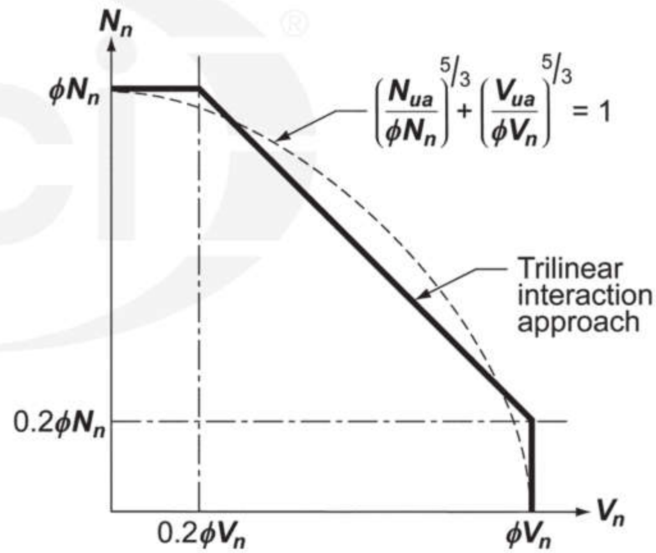

Fig. RE.7—Shear and tensile load interaction equation.

E.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 E.8.1 through E.8.6 unless supplementary reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with ACI 355.2 or ACI 355.4 shall be permitted.

RE.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 and ACI 355.4. In some cases, however, specific products are not known in

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E.8.1 Unless determined in accordance with E.8.4, minimum center-to-center spacing of anchors shall be $4d_a$ for cast-in anchors that will not be torqued, and $6d_a$ for torqued cast-in anchors and post-installed anchors.

E.8.2 Unless determined in accordance with E.8.4, minimum edge distances for cast-in anchors that will not be torqued shall be based on specified cover requirements for reinforcement in 12.7. For cast-in anchors that will be torqued, the minimum edge distances shall be $6d_a$.

E.8.3 Unless determined in accordance with E.8.4, minimum edge distances for post-installed anchors shall be based on the greater of specified cover requirements for reinforcement in 12.7, or minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2 or ACI 355.4, and shall not be less than twice the maximum aggregate size. In the absence of product-specific ACI 355.2 or ACI 355.4 test information, the minimum edge distance shall not be less than:

Adhesive anchors: $6d_a$
Undercut anchors: $6d_a$
Torque-controlled anchors: $8d_a$
Displacement-controlled anchors: $10d_a$

E.8.4 For anchors where installation does not produce a splitting force and that will not be torqued, if the edge distance or spacing is less than those specified in E.8.1 to E.8.3, calculations shall be performed by substituting for d_a a smaller value d_a' that meets the requirements of E.8.1 to E.8.3. Calculated forces applied to the anchor shall be limited to the values corresponding to an anchor having a diameter of d_a'.

E.8.5 Unless determined from tests in accordance with ACI 355.2, the value of h_{ef} for an expansion or undercut post-installed anchor shall not exceed the greater of two-thirds of the member thickness h_a and the member thickness minus 4 in.

E.8.6 Unless determined from tension tests in accordance with ACI 355.2 or ACI 355.4, the critical edge distance c_{ac} shall not be taken less than:

Adhesive anchors: $2h_{ef}$

COMMENTARY

the design stage. Approximate values are provided for use in design.

RE.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 E.5.4, in addition to the normal concrete cover requirements, it may be advantageous to use larger cover to increase the side-face blowout strength.

RE.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.

RE.8.4 In some cases, it may be desirable to use a larger-diameter anchor than the requirements on E.8.1 to E.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_a'.

RE.8.5 Splitting failures are caused by the load transfer between the bolt and the concrete. The limitations on the value of h_{ef} do not apply to cast-in and adhesive anchors because the splitting forces associated with these anchor types are less than for expansion and undercut anchors.

For all post-installed anchors, the maximum embedment depth for a given member thickness should be limited as required to avoid back-face blowout on the opposite side of the concrete member during hole drilling and anchor setting. This is dependent on many variables such as the anchor type, drilling method, drilling technique, type and size of drilling equipment, presence of reinforcement, and strength and condition of the concrete.

RE.8.6 The critical edge distance c_{ac} is determined by the corner test in ACI 355.2 or ACI 355.4, and is only applicable to designs for uncracked concrete. To permit the design of these types of anchors when product-specific informa-

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Undercut anchors: $\mathbf{2.5}h_{ef}$
Torque-controlled expansion anchors: $\mathbf{4}h_{ef}$
Displacement-controlled expansion anchors: $\mathbf{4}h_{ef}$

E.8.7 Contract documents shall specify use of anchors with a minimum edge distance as assumed in design.

E.9—Installation and inspection of anchors

E.9.1 Anchors shall be installed by qualified personnel in accordance with the contract documents. The contract documents shall require installation of post-installed anchors in accordance with the Manufacturer's Printed Installation Instructions (MPII). Installation of adhesive anchors shall be performed by personnel trained to install adhesive anchors.

E.9.2 Installation of anchors shall be inspected in accordance with 1.3 and the general building code. Adhesive anchors shall be subject to the following additional requirements:

E.9.2.1 For adhesive anchors, the contract documents shall specify proof loading where required in accordance with ACI 355.4. The contract documents shall also specify all parameters associated with the characteristic bond stress used for the design according to E.5.5 including minimum

COMMENTARY

tion is not available, conservative default values for c_{ac} are provided. Research has indicated that the corner-test requirements are not met with $c_{a,min} = 1.5h_{ef}$ for many expansion anchors and some undercut anchors because installation of these types of anchors introduces splitting tensile stresses in the concrete that are increased during load application, potentially resulting in a premature splitting failure. Similarly, adhesive anchors that meet the maximum embedment depth requirement of E.8.5 may not fulfill the corner test requirements with $c_{a,min} = c_{Na}$ due to the additional flexural stresses induced in the member by the anchor.

RE.9—Installation and inspection of anchors

RE.9.1 Many anchor performance characteristics depend on proper installation of the anchor. Installation of adhesive anchors should be performed by personnel qualified for the adhesive anchor system and installation procedures being used. Construction personnel can establish qualifications by becoming certified through certification programs. For cast-in anchors, care must be taken that the anchors are securely positioned in the formwork and oriented in accordance with the contract documents. Furthermore, it should be ensured that the concrete around the anchors is properly consolidated. Inspection is particularly important for post-installed anchors to make certain that the Manufacturer's Printed Installation Instructions (MPII) are followed. For adhesive anchors, continuous monitoring of installations by qualified inspectors is recommended to ensure required installation procedures are followed. Post-installed anchor strength and deformation capacity are assessed by acceptance testing under ACI 355.2 or ACI 355.4. These tests are carried out assuming installation in accordance with the MPII. 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 accounted for 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 or ACI 355.4 acceptance testing results could occur if anchor components are altered, or if anchor installation criteria or procedures vary from those specified in the MPII.

RE.9.2.1 Due to the sensitivity of bond strength to installation, on-site quality control is important for adhesive anchors. Where appropriate, a proof loading program should be specified in the contract documents. For adhesive anchors, the contract documents must also provide all parameters

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age of concrete; concrete temperature range; moisture condition of concrete at time of installation; type of lightweight concrete, if applicable; and requirements for hole drilling and preparation.

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relevant to the characteristic bond stress used in the design. These parameters may include, but are not limited to:

1. Acceptable anchor installation environment (dry or saturated concrete; concrete temperature range)
2. Acceptable drilling methods
3. Required hole cleaning procedures
4. Anchor type and size range (threaded rod or reinforcing bar)

Hole cleaning is intended to ensure that drilling debris and dust do not impair bond. Depending on the on-site conditions, hole cleaning may involve operations to remove drilling debris from the hole with vacuum or compressed air, mechanical brushing of the hole wall to remove surface dust, and a final step to evacuate any remaining dust or debris, usually with compressed air. Where wet core drilling is used, holes may be flushed with water and then dried with compressed air. If anchors are installed in locations where the concrete is saturated (for example, outdoor locations exposed to rainfall), the resulting drilling mud must be removed by other means. In all cases, the procedures used should be clearly described by the manufacturer in printed installation instructions accompanying the product. These printed installation instructions, which also describe the limits on concrete temperature and the presence of water during installation as well as the procedures necessary for void-free adhesive injection and adhesive cure requirements, constitute an integral part of the adhesive anchor system and are part of the assessment performed in accordance with ACI 355.4.

E.9.2.2 Installation of adhesive anchors horizontally or upwardly inclined to support sustained tension loads shall be performed by personnel certified by an applicable certification program. Certification shall include written and performance tests in accordance with the ACI/CRSI Adhesive Anchor Installer Certification program, or equivalent.

RE.9.2.2 The sensitivity of adhesive anchors to installation orientation combined with sustained tension loading warrants installer certification. Certification may also be appropriate for other safety-related applications. Certification is established through an independent assessment such as the ACI/CRSI Adhesive Anchor Installation Certification Program, or similar program with equivalent requirements. In addition, installers should obtain instruction through product-specific training offered by manufacturers of qualified adhesive anchor systems.

E.9.2.3 The acceptability of certification other than the ACI/CRSI Adhesive Anchor Installer Certification shall be the responsibility of the licensed design professional.

RE.9.2.3 For the purposes of satisfying E.9.2.3, an equivalent certified installer program should test the adhesive anchor installer's knowledge and skill by an objectively fair and unbiased administration and grading of a written and performance exam. Programs should reflect the knowledge and skill required to install available commercial anchor systems. The effectiveness of a written exam should be verified through statistical analysis of the questions and answers. An equivalent program should provide a responsive and accurate mechanism to verify credentials, which are renewed on a periodic basis.

E.9.2.4 Adhesive anchors installed in horizontal or upwardly inclined orientations to resist sustained tension loads shall be continuously inspected during installation

RE.9.2.4 The IBC (2009) requires special inspection of all post-installed anchors. The installation of adhesive anchors in horizontal or upwardly inclined orientations poses special

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by an inspector specially approved for that purpose by the building official. The special inspector shall furnish a report to the licensed design professional and building official that the work covered by the report has been performed and that the materials used and the installation procedures used conform with the approved contract documents and the Manufacturer's Printed Installation Instructions (MPII).

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challenges to the installer and requires particular attention to execution quality as well as an enhanced level of oversight. It is expected that these anchor installations will be inspected by a certified special inspector who is continuously present when and where the installations are being performed.

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APPENDIX F — STEEL 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 (A416)	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 (A416)	3/8 (0.375)	0.085	0.290
	7/16 (0.438)	0.115	0.390
	1/2 (0.500)	0.153	0.520
	(0.520)	0.167	0.568
	(0.563)	0.192	0.651
	(0.600)	0.217	0.740
	(0.700)	0.294	1.000
Prestressing wire (A421)	0.192	0.029	0.098
	0.196	0.030	0.100
	0.250	0.049	0.170
	0.276	0.060	0.200
Prestressing wire (A821)	0.162	0.021	0.070
	0.177	0.025	0.084
	0.192	0.029	0.098
	0.207	0.034	0.110
	0.225	0.040	0.140
	0.235	0.043	0.150
	0.250	0.049	0.170
Prestressing bars, plain, Grade 150 (A722)	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, Grade 150 (A722)	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
	1-3/4	2.58	9.10
	2-1/2	5.16	18.20

*Availability of some tendon sizes should be investigated in advance.

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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	D3	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	D2	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

*Reference "Manual of Standard Practice—Structural Welded Wire Reinforcement," Wire Reinforcement Institute, Fairfax, VA, 9th Edition, Dec. 2016, 52 pp.

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