Gratitude

In appreciation and gratitude

to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence

& Aviation and Inspector General

For their continuous support and gracious consideration,

the Saudi Building Code National Committee (SBCNC)

is honored to present the first issue of

the Saudi Building Code (SBC).

PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11th June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Requirements for Concrete Structures (SBC 304) were developed based on ICC code in addition to American Concrete Institute (ACI) materials. ACI grants permission to the SBCNC to include ACI materials in the SBC, and ACI is not responsible for any modifications or changes that SBCNC has made to accommodate local conditions.

The development process of SBC 304 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made on ACI, such as Durability Requirements, the simplified methods for the design of two-way slab system of Appendix C, expanding some topics such as Hot Weather, taking into considerations the properties of local material such as the Saudi steel and the engineering level for those involved in the building sector.

As a follow-up to the *Saudi Building Code*, SBCNC offers a companion document, the *Saudi Building Code Concrete Structural Requirements Commentary* (SBC 304C). The basic appeal of the Commentary is thus: it provides in a small package thorough coverage of many issues likely to be dealt with when using the *Saudi Building Code Steel Structural Requirements* (SBC 304) and then supplements that coverage with technical background. Reference lists, information sources and bibliographies are also included.

Strenuous effort has been made to keep the vast quantity of material accessible and its method of presentation useful. With a comprehensive yet concise summary of each section, the Commentary provides a convenient reference for regulations applicable to the construction of buildings and structures. In the chapters that follow, discussions focus on the full meaning and implications of the *Concrete Structural Requirements* (SBC 304) text. Guidelines suggest the most effective method of application, and the consequences of not adhering to the SBC 304 text. Illustrations are provided to aid understanding; they do not necessarily illustrate the only methods of achieving *code* compliance.

The format of the Commentary includes the section, table and figure which is applicable to the same section in the SBC 304C. The numbers of the section, table and figure in the commentary begin with the letter R. The Commentary reflects the most up-to-date text of the 2007 *Saudi Building Code concrete structural requirements* (SBC 304C). American Concrete Institute (ACI) grants permission to the SBCNC to include all or portions of ACI codes and standards in the SBC, and ACI is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Readers should note that the Commentary (SBC 304C) is to be used in conjunction with the *Saudi Building Code concrete structural requirements (SBC 304)* and not as a substitute for the code. **The Commentary is advisory only**; the code official alone possesses the authority and responsibility for interpreting the code.

Comments and recommendations are encouraged, for through your input, it can improve future editions.

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TABLE OF CONTENTS

CHAPTER 1 GENERAL REQUIREMENTS
R1.1 Scope
R1.2 Construction documents
R1.3 Inspection
R1.4 Approval of special systems of design or construction

CHAPTER 2 DEFINITIONS

CHAPTER 3 MATERIALS
R3.0 Notation
R3.1 Tests of materials
R3.2 Cements
R3.3 Aggregates
R3.4 Water
R3.5 Steel reinforcement
R3.6 Admixtures
R3.8 Referenced standards

CHAPTER 4 DURABILITY REQUIREMENTS
R4.2 Freezing and thawing exposures
R4.3 Sulfate exposures
R4.4 Corrosion protection of reinforcement
R4.5 Sulphate plus chloride exposures
R4.6 Sabkha exposures
R4.7 Salt weathering

CHAPTER 5 CONCRETE QUALITY, MIXING, AND PLACING
R5.1 General
R5.2 Selection of concrete proportions
R5.3 Proportioning on the basis of field experience or trial mixtures, or both
R5.4 Proportioning without field experience or trial mixtures
R5.6 Evaluation and acceptance of concrete
R5.7 Preparation of equipment and place of deposit
R5.8 Mixing
R5.9 Conveying
R5.10 Placing
R5.11 Curing
R5.13 Hot weather requirement

CHAPTER 6 FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION JOINTS
R6.1 Design of formwork
R6.2 Removal of forms, shores, and reshoring
R6.3 Conduits and pipes embedded in concrete
R6.4 Construction joints

CHAPTER 7 DETAILS OF REINFORCEMENT
R7.1 Standard hooks
R7.2 Minimum bend diameters
R7.3 Bending
R7.4 Surface conditions of reinforcement
R7.5 Placing reinforcement
R7.6 Spacing limits for reinforcement
R7.7 Concrete protection for reinforcement
R7.8 Special reinforcement details for columns
R7.9 Connections
R7.10 Lateral reinforcement for compression members
R7.11 Lateral reinforcement for flexural members
R7.12 Shrinkage and temperature reinforcement
R7.13 Requirements for structural integrity

CHAPTER 8 ANALYSIS AND DESIGN GENERAL CONSIDERATIONS
R8.0 Notation
R8.1 Design methods
R8.2 Loading
R8.3 Methods of analysis
R8.4 Redistribution of negative moments in continuous flexural members
R8.5 Modulus of elasticity
R8.6 Stiffness
R8.7 Span length
R8.8 Columns
R8.9 Arrangement of live load
R8.10 T-beam construction
R8.11 Joist construction
R8.12 Separate floor finish

CHAPTER 9 STRENGTH AND SERVICEABILITY REQUIREMENTS
R9.0 Notation
R9.1 General
R9.2 Required strength
R9.3 Design strength
R9.4 Design strength for reinforcement
R9.5 Control of deflections

CHAPTER 10 FLEXURE AND AXIAL LOADS
R10.0 Notation
R10.2 Design assumptions
R10.3 General principles and requirements
R10.4 Distance between lateral supports of flexural members
R10.5 Minimum reinforcement of flexural members
R10.6 Distribution of flexural reinforcement in beams and one way slabs
R10.7 Deep beams
R10.8 Design dimensions for compression members
R10.9 Limits for reinforcement of compression members
R10.10 Slenderness effects in compression members
R10.11 Magnified moments-General
R10.12 Magnified moments-Nonsway frames
R10.13 Magnified moments-Sway frames

R10.15 Transmission of column loads through floor system
R10.16 Composite compression members
R10.17 Bearing strength

CHAPTER 11 SHEAR AND TORSION
R11.0 Notation
R11.1 Shear strength
R11.2 Lightweight concrete
R11.3 Shear strength provided by concrete for non prestressed members
R11.4 Shear strength provided by concrete for prestressed members
R11.5 Shear strength provided by shear reinforcement
R11.6 Design for torsion

CHAPTER 12 DEVELOPMENT AND SPLICES OF REINFORCEMENT
R12.1 Development of reinforcement-General
R12.2 Development of deformed bars and deformed wire in tension
R12.3 Development of deformed bars and deformed wire in compression
R12.4 Development of bundled bars
R12.5 Development of standard hooks in tension
R12.6 Mechanical anchorage
R12.7 Development of welded deformed wire fabric in tension
R12.8 Development of welded plain wire fabric in tension
R12.9 Development of prestressing strand
R12.10 Development of flexural reinforcement-General
R12.11 Development of positive moment reinforcement
R12.12 Development of negative moment reinforcement
R12.13 Development of web reinforcement
R12.14 Splices of reinforcement-General
R12.15 Splices of deformed bars and deformed wire intension
R12.16 Splices of deformed bars in compression
R12.17 Special splice requirements for columns
R12.18 Splices of welded deformed wire fabric in tension
R12.19 Splices of welded plain wire fabric in tension

CHAPTER 13 TWO-WAY SLAB SYSTEMS
R13.0 Notation
R13.1 Scope
R13.2 Definitions
R13.3 Slab reinforcement
R13.4 Openings in slab systems
R13.5 Design procedures
R13.6 Direct design method
R13.7 Equivalent frame method

CHAPTER 14 WALLS
R14.0 Notation
R14.1 Scope
R14.2 General
R14.3 Minimum reinforcement
R14.5 Empirical design method
R14.8 Alternative design of slender walls

CHAPTER 15 FOOTINGS
R15.0 Notation
R15.1 Scope
R15.2 Loads and reactions
R15.5 Shear in footings
R15.8 Transfer of force at base of column, wall, or reinforced pedestal
R15.10 Combined footings and mats

CHAPTER 16 PRECAST CONCRETE
R16.0 Notation
R16.1 Scope
R16.2 General
R16.3 Distribution of forces among members
R16.4 Member design
R16.5 Structural integrity
R16.6 Connection and bearing design
R16.7 Items embedded after concrete placement
R16.9 Handling
R16.10 Strength evaluation of precast construction

CHAPTER 17 COMPOSITE CONCRETE FLEXURAL MEMBERS
R17.0 Notation
R17.1 Scope
R17.2 General
R17.3 Shoring
R17.5 Horizontal shear strength
R17.6 Ties for horizontal shear

CHAPTER 18 PRESTRESSED CONCRETE
R18.0 Notation
R18.1 Scope
R18.2 General
R18.3 Design assumptions
R18.4 Serviceability requirements-Flexural members
R18.5 Permissible stresses in prestressing steel
R18.6 Loss of prestress
R18.7 Flexural strength
R18.8 Limits for reinforcement of flexural members
R18.9 Minimum bonded reinforcement
R18.10 Statically indeterminate structures
R18.11 Compression members-Combined flexure and axial loads
R18.12 Slab systems
R18.13 Post-tensioned tendon anchorage zones
R18.14 Design of anchorage zones for monostrand or single 16 mm diameter bar tendons
R18.15 Design of anchorage zones for multistrand tendons
R18.16 Corrosion protection for unbonded tendons
R18.17 Post-tensioning ducts
R18.18 Grout for bonded tendons
R18.20 Application and measurement of prestressing force
R18.21 Post-tensioning anchorages and couplers
R18.22 External post-tensioning

CHAPTER 19 SHELLS AND FOLDED PLATE MEMBERS
R19.0 Notation
R19.1 Scope and definitions
R19.2 Analysis and design
R19.4 Shell reinforcement
R19.5 Construction

CHAPTER 20 STRENGTH EVALUATION OF EXISTING STRUCTURES
R20.0 Notation
R20.1 Strength evaluation-General
R20.2 Determination of required dimensions and material properties
R20.3 Load test procedure
R20.4 Loading criteria
R20.5 Acceptance criteria
R20.6 Provision for lower load rating

CHAPTER 21 SPECIAL PROVISIONS FOR SEISMIC DESIGN
R21.0 Notation
R21.1 Definitions
R21.2 General requirements
R21.3 Flexural members of special moment frames
R21.4 Special moment frame members subjected to bending and axial load
R21.5 Joints of special moment frames
R21.6 Special moment frames constructed using precast concrete
R21.7 Special reinforced concrete structural walls and coupling beams
R21.9 Special diaphragms and trusses
R21.10 Foundations
R21.11 Frame members not proportioned to resist forces induced by earthquake motions
R21.12 Requirements for intermediate moment frames
R21.13 Intermediate precast structural walls

APPENDICES

APPENDIX A STRUT-AND-TIE MODELS
A.0 Notation
A.1 Definitions
A.2 Strut-and-tie model design procedure
A.3 Strength of struts
A.4 Strength of ties
A.5 Strength of nodal zones

APPENDIX B ALTERNATIVE PROVISIONS FOR REINFORCED AND PRESTRESSED CONCRETE FLEXURAL AND COMPRESSION MEMBERS

APPENDIX D ANCHORING TO CONCRETE
D.0 Notation
D.1 Definitions
D.2 Scope
D.3 General requirements
D.4 General requirements for strength of anchors

D.5 Design requirements for tensile loading
D.6 Design requirements for shear loading
D.7 Interaction of tensile and shear forces
D.8 Required edge distances, spacings, and thicknesses to preclude splitting failure
D.9 Installation of anchors

APPENDIX F STEEL REINFORCEMENT INFORMATION

APPENDIX G DESIGN AIDS

COMMENTARY REFERENCES

CHAPTER 1
GENERAL REQUIREMENTS

SECTION R1.1
SCOPE

R1.1.1 The Saudi Building Code Commentary for Concrete Structures referred to as SBC 304C, provides comments on minimum requirements set forth in SBC 304. Hence, SBC 304C is not intended to be used independent of SBC 304, but rather as a companion to provide background information on code provisions.

The term "structural concrete" is used to refer to all concrete used for structural purposes. This covers the spectrum of structural applications of concrete from 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 the code apply to prestressed concrete except for those that are stated to apply specifically to nonprestressed concrete.

Chapter 21 of the code contains special provisions for design and detailing of earthquake resistant structures. See R1.1.8.

Appendix A of the SBC 304 contains provisions for the design of regions near geometrical discontinuities, or abrupt changes in loadings.

Appendix B of the SBC 304 contains provisions for reinforcement limits based on $0.65\rho_b$ determination of the strength reduction factor ϕ, and moment redistribution. The provisions are applicable to reinforced and prestressed concrete members. Designs made using the provisions of Appendix B are equally acceptable as those based on the body of the code, provided the provisions of Appendix B are used in their entirety.

Appendix C of the SBC 304 contains simplified coefficient methods for the design of two-way slabs.

Appendix D of the SBC 304 contains provisions for anchoring to concrete.

R1.1.4 Some special structures involve unique design and construction problems that are not covered by the code. However, many code provisions, such as the concrete quality and design principles, are applicable for these structures. Detailed recommendations for design and construction of some special structures are given in Refs. 1.1 to 1.5.

R1.1.5 The design and installation of piling fully embedded in the ground is regulated by the SBC. For portions of piling in air or water, or in soil not capable of providing adequate lateral restraint throughout the piling length to prevent buckling, the design provisions of this code govern where applicable. Further details about design and construction of piles may be found in Refs. 1.6 to 1.8.

R1.1.7 **Concrete on steel form deck.** In steel framed structures, it is common practice to cast concrete floor slabs on stay-in-place steel form deck. In all cases, the deck serves as the form and may, in some cases, serve an additional structural function.

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

R1.1.8 Special provisions for earthquake resistance

R1.1.8.1 For structures located in regions of low seismic risk, or for structures assigned to low seismic performance or design categories, no special design or detailing is required; the general requirements of the main body of the code apply for proportioning and detailing of reinforced concrete structures. It is expected that concrete structures proportioned by the main body of the code will provide a level of toughness adequate for low earthquake intensity.

R1.1.8.2 Intermediate and special concrete moment frames and shear walls proportioned to resist seismic effects require special reinforcement details. The special proportioning and detailing requirements of Chapter 21 are intended to provide a monolithic reinforced concrete or precast concrete structure with adequate "toughness" to respond inelastically under severe earthquake motions. See also R21.2.1.

SECTION R1.2
CONSTRUCTION DOCUMENTS

R1.2.1 SBC 304 lists some of the more important items of information that should be included in the design drawings, details, or specifications. The SBC does not imply an all inclusive list, and additional items may be required by the building official.

R1.2.2 Documented computer output is acceptable in lieu of manual calculations. When a computer program has been used by the designer, 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, app lied loads, and span lengths. The related output data should include member designation and the shears, moments, and reactions at key points in the span. For column design, it is desirable to include moment magnification factors in the output where applicable.

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

R1.2.3 Building official is the term used to identify the person charged with administration and enforcement of the provisions of the building code. However, such terms as building commissioner or building inspector are variations of the title, and the term building official as used in SBC is intended to include those variations as well as others that are used in the same sense.

SECTION R1.3
INSPECTION

R1.3.1 By inspection, the code does not mean that the inspector should supervise the construction. Rather it means that the one employed for inspection should visit the project with the frequency necessary to observe the various stages of work and ascertain that it is being done in compliance with contract documents and code requirements. The frequency should be at least enough to provide general knowledge of each operation, whether this be several times a day or once in several days.

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

Recommended procedures foe organization and conduct of concrete inspection are given in Ref. 1.9. Detailed methods of inspecting concrete construction are given in Ref. 1.10.

SECTION R1.4
APPROVAL OF SPECIAL SYSTEMS OF DESIGN OR CONSTRUCTION

New methods of design, new materials, and new uses of materials should 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 the SBC.

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

CHAPTER 2
DEFINITIONS

R2.1 For consistent application of the code, it is necessary that terms be defined where they have particular meanings in the code. The definitions given are for use in application of this code only and do not always correspond to ordinary usage. A glossary of most used terms relating to cement manufacturing, concrete design and construction, and research in concrete is contained in Reference 2.1.

Anchorage device - Most anchorage devices for post-tensioning are standard manufactured devices available from commercial sources. In some cases, designers or constructors develop “special” details or assemblages that combine various wedges and wedge plates for anchoring prestressing steel with specialty end plates or diaphragms.

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

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

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 needs not be vertical but may have any orientation in space. Bearing walls, columns, and pedestals qualify as compression members under this definition.

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

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

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

In this code, compressive strength of concrete is based on 150 × 300 mm cylindrical specimens (ASTM C39). Cubic specimens may be used for evaluating concrete compressive strength subjected to the provision of Section R. 5.1.2.

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

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

Loads - A number of 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 SBC 301. 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.

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.

Sheathing - Typically, sheathing is a continuous, seamless, high-density polyethylene material extruded directly on the coated pre-stressing steel.

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

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

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

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

The basic requirement for strength design may be expressed as follows:
Design strength $\geq$ Required strength

$$\phi P_n \geq P_u$$
$$\phi M_n \geq M_u$$
$$\phi V_n \geq V_u$$

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

CHAPTER 3
MATERIALS

SECTION R3.0
NOTATION

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

SECTION R3.1
TESTS OF MATERIALS

R3.1.3 The record of tests of materials and of concrete should be retained for at least 5 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.

SECTION R3.2
CEMENTS

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

SECTION R3.3
AGGREGATES

R3.3.1 Aggregates conforming to the ASTM specifications are not always economically available and, 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. Satisfactory performance in the past, however, does not guarantee good performance under other conditions and in other localities. Whenever possible, aggregates conforming to the designated specifications should be used.

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

SECTION R3.4
WATER

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

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

R.3.4.3 Nonpotable water for mixing or curing concrete should be substantially free from contamination particularly that raise the chloride and sulfate content of concrete. Water containing less than 2000 ppm of total dissolved solid can generally be used satisfactory for making concrete.

SECTION R3.5
STEEL REINFORCEMENT

R3.5.1 Materials permitted for use as reinforcement are specified. Other metal elements, such as inserts, anchor bolts, or plain bars for dowels at isolation or contraction joints, are not normally considered to be reinforcement under the provisions of SBC 304.

R3.5.2 Welding of reinforcing bars is not recommended. However, when needed, the procedure and provisions given in AWS D1.4 or equivalent may be used. See 3.8.2.

R3.5.3 Deformed reinforcement

R3.5.3.2 ASTM A 615M includes provisions for Grade 520 bars in sizes with Dia 20 mm and larger.

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

The 0.35 strain requirement is not applied to reinforcing bars having yield strengths of 420 MPa or less. For steels having strengths of 300 MPa, the assumption of an elasto-plastic stress-strain curve is well justified by extensive test data. For higher strength steels, up to 420 MPa, the stress-strain curve may or may not be elasto-plastic as assumed in 10.2.4, depending on the properties of the steel and the manufacturing process. However, when the stress-strain curve is not elasto-plastic, there is limited experimental evidence to suggest that the actual steel stress at ultimate strength may not be enough less than the specified yield strength to warrant the additional effort of testing to the more restrictive criterion applicable to steels having f_y greater than 420 MPa. In such cases, the ϕ-factor can be expected to account for the strength deficiency.

R3.5.3.5 Welded plain wire fabric should be made of wire conforming to "Specification for Steel Wire, Plain, for Concrete Reinforcement" (ASTM A 82). ASTM A 82 has a minimum yield strength of 482 MPa. The SBC has assigned a yield strength value of 420 MPa, but makes provision for the use of higher yield strengths provided the stress corresponds to a strain of 0.35 percent.

R3.5.3.6 Welded deformed wire fabric should be made of wire conforming to "Specification for Steel Wire, Deformed, for Concrete Reinforcement" (ASTM A

496). ASTM A 496 has a minimum yield strength of 482 MPa. The SBC 304 has assigned a yield strength value of 420 MPa, but makes provision for the use of higher yield strengths provided the stress corresponds to a strain of 0.35 percent.

R3.5.3.7 Galvanized reinforcing bars (A 767M), epoxy-coated reinforcing bars (A 775M) and epoxy-coated prefabricated reinforcing bars (A 934M) are used, especially for conditions where corrosion resistance of reinforcement is of particular concern. They have typically been used in parking decks, bridge decks, and other highly corrosive environments.

R3.5.4 **Plain reinforcement**. Plain bars and plain wire are permitted only for spiral reinforcement (either as lateral reinforcement for compression members, for torsion members, or for confining reinforcement for splices).

R3.5.5 **Prestressing steel**

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

SECTION R3.6 ADMIXTURE

R3.6.3 Admixtures containing any chloride, other than impurities from admixture ingredients, should not be used in prestressed concrete or in concrete with aluminum embedments. Concentrations of chloride ion may produce corrosion of embedded aluminum (e.g., conduit), especially if the aluminum is in contact with embedded steel and the concrete is in a humid environment. Serious corrosion of galvanized steel sheet and galvanized steel stay-in-place forms occurs, especially in humid environments or where drying is inhibited by the thickness of the concrete or coatings or impermeable coverings. See 4.4.1 for specific limits on chloride ion concentration in concrete.

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

R3.6.8 The use of admixtures in concrete containing ASTM C 845 expansive cements has reduced levels of expansion or increased shrinkage values. See Ref. 3.2.

SECTION 3.8 REFERENCED STANDARDS

The ASTM standard specifications listed are the latest editions at the time these SBC 304 provisions were adopted. Since these specifications are revised frequently, generally in minor details only, the user of the SBC 304 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.

Standard specifications or other material to be legally adopted by reference into SBC should 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 standard documents referenced in the SBC 304 are listed in section 3.8, with the title and complete serial designation. In other sections of the SBC 304, the designations do not include the date so that all may be kept up-to-date by simply revising section 3.8.

CHAPTER 4
DURABILITY REQUIREMENTS

This chapter emphasizes the importance of considering durability requirements before the designer selects f'_c and concrete cover over the reinforcing steel. Concrete exposed to sulfate-bearing soils or groundwater, seawater, or for preventing corrosion of reinforcing steel or salt weathering should be designed for maximum water-cementitious materials ratio, minimum cementitious materials content and appropriate type of cement. Maximum water-cementitious materials ratios of 0.40 to 0.50 that may be required for concretes exposed to sulfate-bearing soils or groundwaters, or for preventing corrosion of reinforcement will typically be equivalent to requiring an f'_c of about 35 to 28 MPa, respectively. Generally, the required average concrete strength, f'_{cr}, will be 5 to 8 MPa higher than the specified compressive strength, f'_c.

Since it is difficult to accurately determine the water-cementitious materials ratio of concrete during production, the f'_c specified should be reasonably consistent with the water-cementitious materials ratio selected for durability. Selection of an f'_c that is consistent with the water-cementitious materials ratio selected for durability will help ensure that the required water-cementitious materials ratio is actually obtained in the field. Because the usual emphasis on inspection is for strength, test results substantially higher than the specified strength may lead to a lack of concern for quality and production of concrete that exceeds the maximum water-cementitious materials ratio. Thus, an f'_c of less than 20 MPa and a maximum water-cementitious materials ratio of 0.45 should not be specified for structures exposed to aggressive environments.

The SBC 304 does not include provisions for especially severe exposures, such as acids or high temperatures, and is not concerned with aesthetic considerations, such as surface finishes. These items are beyond the scope of the SBC 304 and should be covered specifically in the project specifications. Concrete ingredients and proportions are to be selected to meet the minimum requirements stated in the SBC 304 and the additional requirements of the contract documents.

SECTION R4.2
FREEZING AND THAWING EXPOSURE

Freeze-thaw conditions are rarely observed in the climatic conditions of the Kingdom.

SECTION R4.3
SULFATE EXPOSURE

R4.3.1 Concrete exposed to injurious concentrations of sulfates from soil or groundwater should be made with sulfate-resisting cement. Table 4.3.1 lists the appropriate types of cement and the maximum water-cementitious materials ratios, minimum cementitious materials contents and minimum compressive strength for various exposure conditions. In selecting cement for sulfate resistance, the principal consideration is its tricalcium aluminate (C_3A) content. For moderate exposures, Type II cement is limited to a maximum C_3A content of 8.0 percent under ASTM

C 150. For severe exposures, Type V cement with a maximum C_3A content of 5 percent is specified. Type V cement may be used when Type II cement is not available.

ASTM C 1012[4.1] can be used to evaluate the sulfate resistance of concrete mixtures using combinations of cementitious materials.

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

TABLE 4.3.1 – REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-BEARING SOILS OR WATER

Sulfate exposure	Water soluble sulfate (SO_4) in soil, percent by weight	Sulfate (SO_4) in water, ppm	Cement type	Maximum water-cementitious materials ratio, by weight	Minimum cementitious materials content, kg/m^3	Minimum f_c' , MPa
Negligible	$0.00 \le SO_4 < 0.10$	$0 \le SO_4 < 150$	–	–	–	–
Moderate	$0.10 \le SO_4 < 0.20$	$150 \le SO_4 < 1500$	II	0.50	330	28
Severe+	$0.20 \le SO_4 \le 2.00$	$1500 \le SO_4 \le 10{,}000$	V	0.45	350	30
Very severe+	$SO_4 > 2.00$	$SO_4 > 10{,}000$	V plus pozzolan++	0.45	350	30

\+ If sulfate ions are associated with magnesium ions, supplementary protection, such as application of a barrier coating, is required.

++ Pozzolan that conforms to relevant ASTM standards or that is shown to improve the sulfate resistance by service records should only be used.

SECTION R4.4
CORROSION PROTECTION OF REINFORCEMENT

R4.4.1 Test procedures for determining chloride concentration in concrete should conform to those given in ASTM C 1218. An initial evaluation may be obtained by testing individual concrete ingredients for water-soluble chloride ion content. If the water-soluble chloride ion content, calculated on the basis of concrete proportions, exceeds those permitted in Table 4.4.1, it may be necessary to test samples of the hardened concrete for water-soluble chloride ion content. Some of the total chloride ions present in the ingredients will either be insoluble or will react with the cement during hydration and become insoluble under the test procedures described in ASTM C 1218. Additional information on the effect of chloride on the corrosion of reinforcement steel are given in Ref. 4.2 and 4.3.

When concrete is tested for water-soluble chloride ion content, the test should be made at an age of 28 to 42 days.

The limits in Table 4.4.1 are applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.

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

Type of member	Maximum water-soluble chloride ion (cl^-) in concrete, percent by weight of cement*
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

* Determined according to ASTM C 1218.

R4.4.2 When reinforced concrete structures are exposed to external sources of chlorides, the water-cementitious materials ratio, cementitious materials content, and specified compressive strength, f'_c, of Table 4.4.2 are the minimum requirements that are to be considered. Epoxy- or zinc-coated bars or slag meeting ASTM C 989 or fly ash meeting ASTM C 618 or silica fume meeting ASTM C 1240 with an appropriate high-range water reducer, ASTM C 494M, Types F and G, or ASTM C 1017M can provide additional protection [4.4]. When epoxy-coated steel bars are used, they should be according to ASTM A 775 specifications.

The requirements for minimum concrete cover over the reinforcing steel of 7.7 in conjunction with 7.7.5 should be considered.

The requirements for protection of concrete against carbonation are not provided as it is expected that the use of quality concrete and adequate cover over reinforcing steel, as specified in the SBC 304, will minimize this problem.

R4.4.5 In the coastal areas, such as in Jeddah, Yanbu, Dammam, Jizan, and others, the substructures are exposed to chloride- and sulfate-bearing soil and/or groundwater. In such situations, the requirements of 4.5 shall be considered.

TABLE 4.4.2 – REQUIREMENTS FOR CONCRETE EXPOSED TO CHLORIDE-BEARING SOIL AND WATER

Chloride exposure	Water soluble chloride (cl^-) in soil, percent by weight	Water soluble chloride (cl^-) in water, ppm	Cement type	Maximum water-cementitious materials ratio	Minimum cementitious materials content, kg/m^3	Minimum f'_c, MPa
Negligible	Upto 0.05	Up to 500	–	–	–	–
Moderate	0.05 to 0.1	500 to 2,000	–	0.50	330	28
Severe	0.1 to 0.5	2,000 to 10,000	I	0.45	350	30
Very severe	More than 0.5	More than 10,000	I + pozzolan$^+$	0.40	370	35

+Pozzolan that conforms to relevant standards shall only be used.

SECTION R4.5
SULFATE PLUS CHLORIDE EXPOSURES

R4.5.1 Structures exposed to environments containing both sulfate and chloride salts are prone to both sulfate attack and reinforcement corrosion. For such environments, the concrete mixture proportions should be selected for the severest of the exposure conditions, in terms of chloride or sulfate concentration, in Tables 4.3.1

or 4.4.2. In such situations, the lowest applicable maximum water-cementitious materials ratio and highest pertinent minimum cementitious materials content for the severest exposure conditions should be selected.

Since reinforcement corrosion is the major form of concrete deterioration, in a chloride-sulfate environment, as sulfate ions do not penetrate deeper into the concrete cover, it is suggested to use the cement type specified in Table 4.4.2, rather than that dictated by the severity of the exposure conditions.

SECTION R4.6
SABKHA EXPOSURES

R4.6.1 Sabkha is a saline flat land underlain by sand, silt or clay and is often encrusted with salt. These soils either border partially land-locked seas or cover a number of continental depressions. Both of these types are usually formed in hot, arid climates and are associated with shallow groundwater table. Sabkha terrains are typically located in Jubail, Rastanura, Abqaiq, Dammam, and Shaibah along the Arabian Gulf coast. They are prevalent in Jeddah, Jizan, Qunfudhah, Al-Lith, Rabigh, Yanbu in Western Province as well as in Wadi As-Sirhan, around Al-Qasim. and around Riyadh.

The salinity of sabkha soil is three to five times that of seawater from the same vicinity. The high chloride concentration in these soils accelerates concrete deterioration due to reinforcement corrosion. In such situations, in addition to utilizing quality concrete, the structural components need to be protected by appropriate measures, such as tanking or epoxy-based coating.

SECTION R4.7
SALT WEATHERING

R4.7.1 Concrete exposed to splash in a marine environment and soil with shallow groundwater table or water from irrigation is susceptible to deterioration due to salt weathering in the hot and arid environment of the Kingdom. In addition to utilizing quality concrete, it may be necessary to provide additional protective measures, such as the application of an appropriate barrier coating.

In marine structures, the protection should be provided in the splash zone. Tanking or application of a barrier coating in portions exposed to soil is necessary for the substructures.

CHAPTER 5
CONCRETE QUALITY, MIXING AND PLACING

The requirements for proportioning concrete mixtures are based on the philosophy that concrete should provide both adequate durability (Chapter 4) and strength. The criteria for acceptance of concrete are based on the philosophy that the SBC 304 is intended primarily to protect the safety of the public. Chapter 5 describes procedures by which concrete of adequate strength can be obtained, and provides procedures for checking the quality of the concrete during and after its placement in the work.

Chapter 5 also prescribes minimum criteria for mixing and placing concrete.
The provisions of 5.2, 5.3, and 5.4, together with Chapter 4, establish required mixture proportions. The basis for determining the adequacy of concrete strength is in 5.6.

SECTION R5.1
GENERAL

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

R5.1.2 Cubic specimens (150 × 150 × 150) in accordance with SASO 79 may be used in evaluating the compressive strength, using the following correction factor: $f'_c = k(\ f_{cubic}\ )$ where $k = 0.8$.

R5.1.4 Sections 9.5.2.3 (modulus of rupture), 11.2 (concrete shear strength) and 12.2.4 (development of reinforcement) require modification in the design criteria for the use of lightweight aggregate concrete. Two alternative modification procedures are provided. One alternative is based on laboratory tests to determine the relationship between splitting tensile strength f_{ct} and specified compressive strength f'_c for the lightweight concrete. For a lightweight aggregate from a given source, it is intended that appropriate values of f_{ct} be obtained in advance of design.

R5.1.5 Tests for splitting tensile strength of concrete (as required by 5.1.4) are not intended for control of, or acceptance of the strength of concrete in the field. Indirect control will be maintained through the normal compressive strength test requirements provided by 5.6.

SECTION R5.2
SELECTION OF CONCRETE PROPORTIONS

Recommendations for selecting proportions for concrete are given in detail in Reference 5.1. (Provides two methods for selecting and adjusting proportions for normalweight concrete: the estimated weight and absolute volume methods.

Example calculations are shown for both methods. Proportioning of heavyweight concrete by the absolute volume method is presented in an appendix.)

Recommendations for lightweight concrete are given in Reference 5.2. (Provides a method of proportioning and adjusting structural grade concrete containing lightweight aggregates.)

R5.2.1 The selected water-cementitious materials ratio should be low enough, or in the case of lightweight concrete the compressive strength high enough to satisfy both the strength criteria (see 5.3 or 5.4) and the special exposure requirements (Chapter 4). The SBC 304 does not include provisions for especially severe exposures, such as acids or high temperatures, and is not concerned with aesthetic considerations such as surface finishes. These items are beyond the scope of the SBC 304 and should be covered specifically in the project specifications. Concrete ingredients and proportions are to be selected to meet the minimum requirements stated in the SBC 304 and the additional requirements of the contract documents.

R5.2.3 The SBC 304 emphasizes the use of field experience or laboratory trial mixtures (see 5.3) as the preferred method for selecting concrete mixture proportions.

SECTION R5.3
PROPORTIONING 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. The second is the determination of the required average strength. The third is the selection of mixture proportions required to produce that average strength, either by conventional trial mixture procedures or by a suitable experience record. Fig. R5.3 is a flow chart outlining the mixture selection and documentation procedure.

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

R5.3.1 **Standard deviation.** When a concrete production facility has a suitable record of 30 consecutive tests of similar materials and conditions expected, the standard deviation is calculated from those results in accordance with the following formula:

$$s = \left[\frac{\sum \left(x_i - \bar{x} \right)^2}{(n-1)} \right]^{1/2}$$

where:

s = standard deviation, MPa

x_i = individual strength tests as defined in 5.6.2.4

$\bar{x}$ = average of n strength test results

n = number of consecutive strength tests

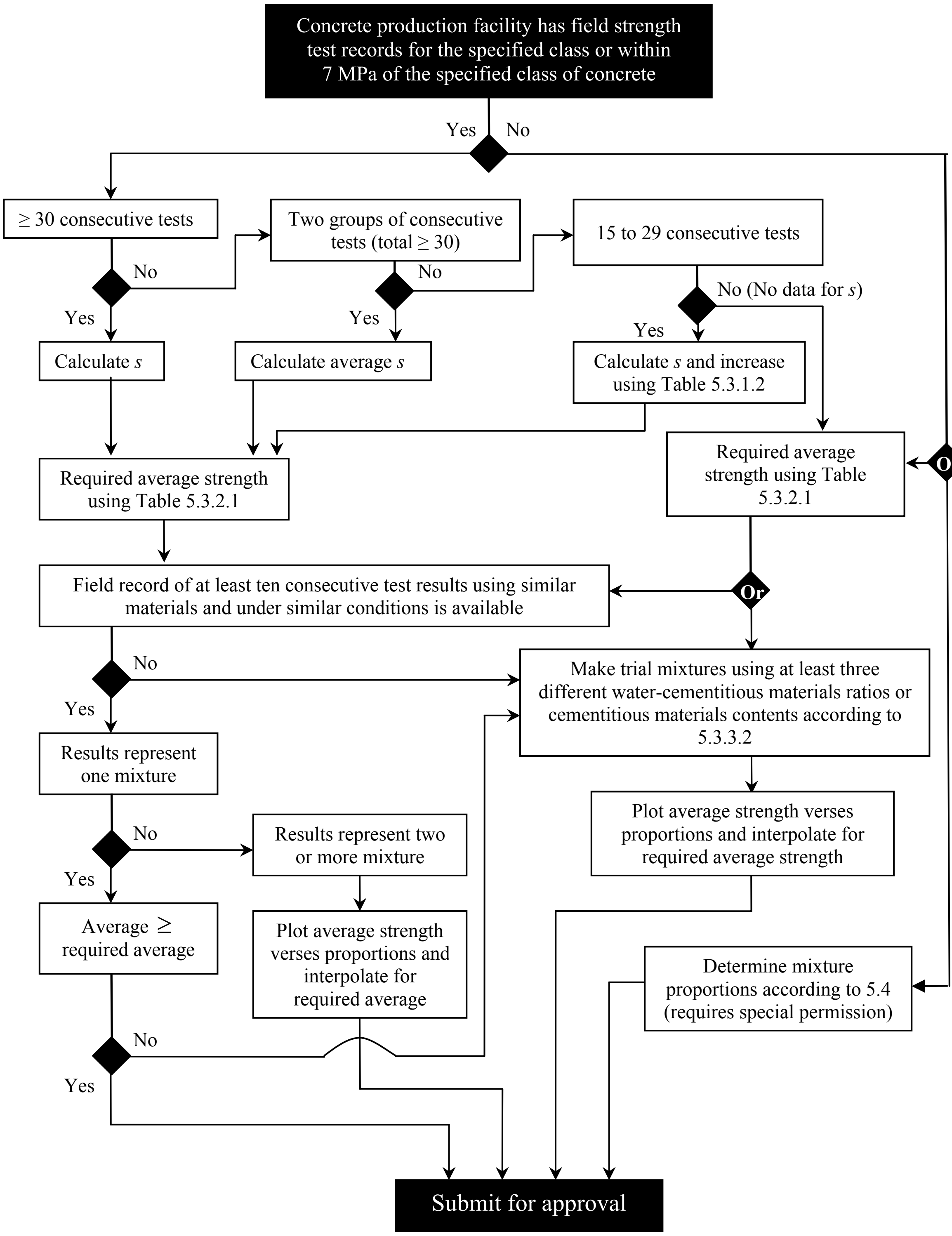


Fig. R5.3 - Flow chart for selection and documentation of concrete proportions

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

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

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

where:

$\bar{s}$ = statistical average standard deviation where two test records are used to estimate the standard deviation

s_1, s_2 = standard deviations calculated from two test records, 1 and 2, respectively

n_1, n_2 = number of tests in each test record, respectively

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

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

Concrete for background tests to determine standard deviation is considered to be "similar" to that required if made with the same general types of ingredients under no more restrictive conditions of control over material quality and production methods than on the proposed work, and if its specified strength does not deviate more than 7 MPa from the f_c' required [see 5.3.1.1(b)]. A change in the type of concrete or a major increase in the strength level may increase the standard deviation. Such a situation might occur with a change in type of aggregate (i.e., from natural aggregate to lightweight aggregate or vice versa) or a change from non-air-entrained concrete to air-entrained concrete. Also, there may be an increase in standard deviation when the average strength level is raised by a significant amount, although the increment of increase in standard deviation should be somewhat less than directly proportional to the strength increase. When there is reasonable doubt, any estimated standard deviation used to calculate the required average strength should always be on the conservative (high) side.

Note that the SBC 304 uses the standard deviation in mega Pascal instead of the coefficient of variation in percent. The latter is equal to the former expressed as a percent of the average strength.

Even when the average strength and standard deviation are of the levels assumed, there will be occasional tests that fail to meet the acceptance criteria prescribed in 5.6.3.3 (perhaps 1 test in 100).

TABLE 5.3.1.2-MODIFICATION FACTOR FOR STANDARD DEVIATION WHEN LESS THAN 30 TESTS ARE AVAILABLE

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

* Interpolate for intermediate numbers of tests.

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

R5.3.2 Required average strength

R5.3.2.1 Once the standard deviation has been determined, the required average compressive strength is obtained from the larger value computed from Eq. (5-1) and (5-2) for f'_c of 35 MPa or less, or the larger value computed from Eq. (5-1) and (5-3) for f'_c over 35 MPa. 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 3.5 MPa below the specified compressive strength f'_c. Equation (5-3) is based on the same 1-in-100 probability that an individual test may be less than 0.90 f'_c. These equations assume that the standard deviation used is equal to the population value appropriate for an infinite or very large number of tests. For this reason, use of standard deviations estimated from records of 100 or more tests is desirable. When 30 tests are available, the probability of failure will likely be somewhat greater than 1-in-100. The additional refinements required to achieve the 1-in-100 probability are not considered necessary, because of the uncertainty inherent in assuming that conditions operating when the test record was accumulated will be similar to conditions when the concrete will be produced.

R5.3.2.2 Recent evaluation of production control of ready mix concrete in the Kingdom has shown that values of standard deviation provided by ready mix concrete plants under-estimate the true standard deviation substantially.[5.3,5.4] Therefore, it is prudent to use the required average strength based on Table 5.3.2.2 until the true standard deviation can be established by qualified independent laboratory.

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

Specified compressive strength, f'_c, MPa	Required average compressive strength, f'_{cr}, MPa
20 to 35	$f'_c + 8.5$
Over 35	$1.10 f'_c + 5.0$

R.5.3.3 **Documentation of average strength**. Once the required average strength f'_{cr}, is known, the next step is to select mixture proportions that will produce an average strength at least as great as the required average strength, and also meet special exposure requirements of Chapter 4. The documentation may consist of a strength test record, several strength test records, or suitable laboratory or field trial mixtures. 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 the required average strength, different proportions may be necessary or desirable. In such instances, the average from a record of as few as 10 tests may be used, or the proportions may be established by interpolation between the strengths and proportions of two such records of consecutive tests. All test records for establishing proportions necessary to produce the average strength are to meet the requirements of 5.3.3.1 for "similar materials and conditions."

For strengths over 35 MPa where the average strength documentation is based on laboratory trial mixtures, it may be appropriate to increase f'_{cr}, calculated in Table 5.3.2.2 to allow for a reduction in strength from laboratory trials to actual concrete production.

SECTION R5.4
PROPORTIONING WITHOUT FIELD EXPERIENCE OR TRIAL MIXTURES

R5.4.1 When no prior experience (5.3.3.1) or trial mixture data (5.3.3.2) meeting the requirements of these sections is available, other experience may be used only when special permission is given. Because combinations of different ingredients may vary considerably in strength level, this procedure is not permitted for f'_c greater than 35 MPa and the required average strength should exceed f'_c by 8.5 MPa. The purpose of this provision is to allow work to continue when there is an unexpected interruption in concrete supply and there is not sufficient time for tests and evaluation or in small structures where the cost of trial mixture data is not justified.

SECTION R5.6
EVALUATION AND ACCEPTANCE OF CONCRETE

Once the mixture proportions have been selected and the job started, the criteria for evaluation and acceptance of the concrete can be obtained from 5.6.
An effort has been made in the SBC 304 to provide a clear-cut basis for judging the acceptability of the concrete, as well as to indicate a course of action to be followed when the results of strength tests are not satisfactory.

R5.6.1 Laboratory and field technicians can establish qualifications by becoming certified through certification programs. Field technicians in charge of sampling concrete; testing for slump, unit weight, yield, air content, and temperature; and making and curing test specimens should be certified in accordance with the requirements of ASTM C 1077,[5.5] or an equivalent program. Concrete testing laboratory personnel should be certified in accordance with the requirements of ASTM C 1077 or an equivalent program.

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

R5.6.2 **Frequency of testing**

R5.6.2.1 The following three criteria establish the required minimum sampling frequency for each class of concrete:

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

(b) Once for each 120 m^3 of each class placed each day, nor less than

(c) Once for each 500 m^2 of slab or wall surface area placed each day.

In calculating surface area, only one side of the slab or wall should be considered. Criteria (c) will require more frequent sampling than once for each 120 m^3 placed if the average wall or slab thickness is less than 250 mm.

R5.6.2.2 Samples for strength tests are to be taken on a strictly random basis if they are to measure properly the acceptability of the concrete. To be representative, the choice of times of sampling, or the batches of concrete to be sampled, are to be made on the basis of chance alone, within the period of placement. Batches should not be sampled on the basis of appearance, convenience, or other possibly biased criteria, because the statistical analyses will lose their validity. Not more than one test (average of two cylinders made from a sample, 5.6.2.4) should be taken from a single batch, and water may not be added to the concrete after the sample is taken.

ASTM D 3665[5.6] describes procedures for random selection of the batches to be tested.

R5.6.3 Laboratory-cured specimens

R5.6.3.3 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 SBC 304, regardless of design method used. The concrete strength is considered to be satisfactory as long as averages of any three consecutive strength tests remain above the specified f_c' and no individual strength test falls below the specified f_c' by more than 3.5 MPa if f_c' is 35 MPa or less, or falls below f_c' by more than 10 percent if f_c' is over 35 MPa. 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.

R5.6.3.4 When concrete fails to meet either of the strength requirements of 5.6.3.3, steps should be taken to, increase the average of the concrete test results. If sufficient concrete has been produced to accumulate at least 15 tests, these should be used to establish a new target average strength as described in 5.3.

If fewer than 15 tests have been made on the class of concrete in question, the new target strength 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 accompanied by evidence that the average strength level will be improved.

R5.6.4 Field-cured specimens

R5.6.4.1 Strength tests of cylinders cured under field conditions may be required to check the adequacy of curing and protection of concrete in the structure.

R5.6.4.4 Positive guidance is provided in the SBC 304 concerning the interpretation of tests of field-cured cylinders. Research has shown that cylinders protected and cured to simulate good field practice should test not less than about 85 percent of standard laboratory moist-cured cylinders. This percentage has been set as a rational basis for judging the adequacy of field curing. The comparison is made between the actual measured strengths of companion job-cured and laboratory-cured cylinders, not between job-cured cylinders and the specified value of f_c'. However, results for the job-cured cylinders are considered satisfactory if the job-cured cylinders exceed the specified f_c' by more than 3.5 MPa, even though they fail to reach 85 percent of the strength of companion laboratory-cured cylinders.

R5.6.5 **Investigation of low-strength test results.** Instructions are provided concerning the procedure to be followed when strength tests have failed to meet the specified acceptance criteria. For obvious reasons, these instructions cannot be dogmatic. The building official 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 in place, such as by probe penetration, impact hammer, ultrasonic pulse velocity or pull out may be useful in determining whether or not a portion of the structure actually contains low-strength concrete. Such tests are of value primarily for comparisons within the same job rather than as quantitative measures of strength. For cores, if required, conservatively safe acceptance criteria are provided that should ensure structural adequacy for virtually any type of construction.[5.7-5.10] Lower strength may, of course, be tolerated under many circumstances, but this again becomes a matter of judgment on the part of the building official and design engineer. When the core tests fail to provide assurance of structural adequacy, it may be practical, particularly in the case of floor or roof systems, for the building official to require a load test (Chapter 20). Short of load tests, if time and conditions permit, an effort may be made to improve the strength of the concrete in place by supplemental wet curing. Effectiveness of such a treatment should 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.[5.11] The restriction on the commencement of core testing provides a minimum time for the moisture gradient to dissipate.

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

The SBC 304, as stated, concerns itself with assuring structural safety, and the instructions in 5.6 are aimed at that objective. It is not the function of the SBC 304 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'.

SECTION R5.7
PREPARATION OF EQUIPMENT AND PLACE OF DEPOSIT

Recommendations for mixing, handling and transporting, and placing concrete are given in Reference 5.12 (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 should be removed. Reinforcement should be thoroughly cleaned of dirt, loose rust, mill scale, or other coatings. Water should be removed from the forms.

SECTION R5.8
MIXING

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

SECTION R5.9
CONVEYING

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

The SBC 304 requires the equipment for handling and transporting concrete to be capable of supplying concrete to the place of deposit continuously and reliably under all conditions and for all methods of placement. The provisions of 5.9 apply to all placement methods, including pumps, belt conveyors, pneumatic systems, wheelbarrows, buggies, crane buckets, and tremies.

Serious loss in strength can result when concrete is pumped through pipe made of aluminum or aluminum alloy 5.13 Hydrogen gas generated by the reaction between the cement alkalies and the aluminum eroded from the interior of the pipe surface has been shown to cause strength reduction as much as 50 percent. Hence, equipment made of aluminum or aluminum alloys should not be used for pump lines, tremies, or chutes other than short chutes such as those used to convey concrete from a truck mixer.

SECTION R5.10
PLACING

Re-handling concrete can cause segregation of the materials. Hence the SBC 304 cautions against this practice. Re-tempering of partially set concrete with the addition of water should not be permitted, unless authorized. This does not preclude the practice (recognized in ASTM C 94) of adding water to mixed concrete to bring it up to the specified slump range so long as prescribed limits on the maximum mixing time and water-cementitious materials ratio are not violated.

Recommendations for consolidation of concrete are given in detail in Ref. 5.14 (Presents current information on the mechanism of consolidation and gives recommendations on equipment characteristics and procedures for various classes of concrete).

SECTION R5.11
CURING

Recommendations for curing concrete are given in detail in Ref. 5.15 (Presents basic principles of proper curing and describes the various methods, procedures, and materials for curing of concrete.)

R5.11.3 **Accelerated curing.** The provisions of this section apply whenever an accelerated curing method is used, whether for precast or cast-in-place elements. The compressive strength of steam-cured concrete is not as high as that of similar concrete continuously cured under moist conditions at moderate temperatures. Also the modulus of elasticity E_c, of steam-cured specimens may vary from that of specimens moist-cured at normal temperatures. When steam curing is used, it is advisable to base the concrete mixture proportions on steam-cured test cylinders.

Accelerated curing procedures require careful attention to obtain uniform and satisfactory results. Preventing moisture loss during the curing is essential.

R5.11.4 In addition to requiring a minimum curing temperature and time for normal- and high-early-strength concrete, the SBC 304 provides a specific criterion in 5.6.4 for judging the adequacy of field curing. At the test age for which the strength is specified (usually 28 days), field-cured cylinders should produce strength not less than 85 percent of that of the standard, laboratory-cured cylinders. For a reasonably valid comparison to be made, field-cured cylinders and companion laboratory-cured cylinders should come from the same sample. Field-cured cylinders should be cured under conditions identical to those of the structure. If the structure is protected from the elements, the cylinder should be protected.

Cylinders related to members not directly exposed to weather should be cured adjacent to those members and provided with the same degree of protection and method of curing. The field cylinders should not be treated more favorably than the elements they represent. (See 5.6.4 for additional information.) If the field-cured cylinders do not provide satisfactory strength by this comparison, measures should be taken to improve the curing. If the tests indicate a possible serious deficiency in strength of concrete in the structure, core tests may be required, with or without supplemental wet curing, to check the structural adequacy, as provided in 5.6.5.

SECTION R5.13
HOT WEATHER REQUIREMENTS

R5.13.1 Hot weather is any combination of high ambient temperature, high concrete temperature; low relative humidity; wind speed; and solar radiation that tends to impair the quality of fresh or hardened concrete.

Potential problems for concrete in the freshly mixed state are likely to include: (a) increased water demand, (b) increased rate of slump loss, (c) increased rate of setting, and (d) increased tendency for plastic shrinkage cracking.
Potential deficiencies to concrete in the hardened state may include: (a) decreased long-term strength, (b) increased tendency for drying shrinkage and differential thermal cracking, and (c) decreased durability.
A detailed description of the hot weather factors and their effect on concrete properties is given in Ref. 5.16.

R5.13.2 Local Experience has shown that when reasonable precautions are employed by the batching plants, concrete with a temperature of less than 35 °C can be delivered to jobsite [5.3].

Some of the precautions that may be employed to control concrete temperature include: (*i*) shading the aggregate stockpiles and bins, (*ii*) sprinkling water to cool the coarse aggregates, (*iii*) cooling the mixing water, (*iv*) using ice as part of the mixing water, and (*v*) painting trucks, silos and other related equipments, with white or light color. The contribution of aggregates, cement and water to the temperature of the freshly mixed concrete is related to their temperature, specific heat, and quantity of each material.

Tanks and pipelines carrying mixing water should be buried, insulated, shaded, or painted white or with light color to keep water at the lowest possible temperature. Silos and bins should be painted with heat-reflective paint to minimize absorption of heat. The surface of truck mixer should be painted white to minimize solar heat gain.

R5.13.3 Chemical admixtures have been found to be beneficial in offsetting some of the undesirable characteristics of concrete placed during periods of high ambient temperatures. The benefits may include lower mixing water, extended period of use, and strengths comparable to, or higher than, those of concrete without admixtures, placed at lower temperatures [5.16].

Admixtures meeting the requirements of ASTM C 494, Type D, have both water-reducing and set-retarding properties and are widely used under hot weather conditions. They can be included in concrete in varying proportions and in combination with other admixtures[5.16].

Some high-range, water-reducing and retarding admixtures (ASTM C 494, Type G) and plasticizing retarding admixtures (ASTM C 1017, Type II), often referred to as superplasticizers, can provide significant benefits under hot weather conditions. In particular, improved handling characteristic of concrete permits rapid placement and consolidation, and the period between mixing and placing can thus be reduced [5.16].

R5.13.4 Transporting and placing concrete should be done as quickly as practicable. Delays contribute to loss of slump and an increase in the concrete temperature. Sufficient labor and equipment must be available at the jobsite to handle and place concrete immediately upon delivery.

Water-reducing and retarding admixtures formulated for extended slump retention should be considered if longer delivery periods are anticipated.
Concrete placement may be scheduled at times other than during daylight hours. Night-time production and placement require good planning and good lighting.

R5.13.5 High-range water-reducing admixtures or superplasticizers may be utilized to regain the desired workability.

R5.13.6 Plastic shrinkage cracking is usually associated with hot-weather concreting. However, it can occur at ambient conditions that produce rapid evaporation of moisture from the concrete surface. These cracks can occur on the surface of freshly placed concrete while it is being finished or shortly thereafter, if the rate of water evaporation is more than the rate of bleeding. These cracks that appear mostly on horizontal surfaces can be substantially eliminated if preventive measures are taken.

Precautions to avoid plastic shrinkage cracking may include: erecting wind breakers and sun shades, fog spraying of form and reinforcement, dampening the sub-grade and forms, or placing concrete at the lowest practicable temperature, and time. After the completion of placing and finishing operations, concrete should be protected from high temperature, direct sun light, low humidity, and drying winds. When the rate of evaporation exceeds 1 kg/m^2 per hour, precautionary measures are essential. Pozzolanic cement concrete is particularly prone to plastic shrinkage. Therefore, protection from premature drying is essential for pozzolanic cement concrete even at low evaporation rates.

The probability of plastic shrinkage cracks to occur may be increased if the time of setting of concrete is delayed due to the use of an excessive dosage of retarding admixture.

R5.13.7 Curing is more critical under hot weather conditions. Early curing is essential when pozzolan cement concrete is utilized.

Of the different curing procedures, moist-curing is the best. It can be provided by ponding, covering with clean sand kept continuously wet or continuous sprinkling of water. A more practical method of moist-curing is that of covering the pre-wetted concrete with an impervious sheeting or application of absorptive mats or fabric kept continuously wet with a soaker hose or similar means [5.16].

R5.13.8 The specimens should be representative of the concrete as delivered. High temperature, low relative humidity, and drying winds are particularly detrimental to the fresh concrete used for making tests and molding specimens. Leaving the specimens of fresh concrete exposed to sun, wind, or dry air will invalidate the test results.

Particular attention should be given to the protection and curing of strength test specimens used as a basis for acceptance of concrete. Due to their small size in relation to most parts of the structure, test specimens are influenced more readily by changes in ambient temperatures. Extra effort is needed in hot weather to maintain strength test specimens at a temperature of 16 to 27°C and to prevent moisture loss during the initial curing period [5.16].

CHAPTER 6
FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION JOINTS

SECTION 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 Reference 6.1. (Provides recommendations for design, construction, and materials for formwork, forms for special structures, and formwork for special methods of construction. Directed primarily to contractors, the suggested criteria will aid engineers in preparing job specifications for the contractors.)

***Formwork for Concrete*[6.2] -** A how-to-do-it handbook for contractors, engineers, and architects following the guidelines established in Ref. 6.2. Planning, building, and using formwork are discussed, including tables, diagrams, and formulas for form design loads.

SECTION R6.2
REMOVAL OF FORMS, SHORES, AND RESHORING

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

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

(a) Tests of cast-in-place cylinders in accordance with "Standard Test Method for Compressive Strength of Concrete Cylinders Cast-in-Place in Cylindrical Molds" (ASTM C 873[6.4]). (This method is limited to use in slabs where the depth of concrete is from 125 to 300 mm);

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

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

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

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

Where the structure is adequately supported on shores, the side forms of beams, girders, columns, walls, and similar vertical forms may generally be removed after 12 h 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 10°C. The 12-h cumulative curing time is based on regular cements and ordinary conditions; the use of special cements or unusual conditions may require adjustment of the given limits. For example, concrete made with Type II or V (ASTM C 150) or ASTM C 595 cements, concrete containing retarding admixtures, and concrete to which ice was added during mixing (to lower the temperature of fresh concrete) may not have sufficient strength in 12 h and should be investigated before removal of formwork.

The removal of formwork for multistory construction should be a part of a planned procedure considering the temporary support of the whole structure as well as that of each individual member. Such a procedure should be worked out 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 multistory construction, the strength of the concrete during the various stages of construction should be substantiated by field-cured test specimens or other approved methods.

SECTION R6.3
CONDUITS AND PIPES EMBEDDED IN CONCRETE

R6.3.1 Conduits, pipes, and sleeves not harmful to concrete can be embedded within the concrete, but the work should be done in such a manner that the structure will not be endangered. Empirical rules are given in 6.3 for safe installations under common conditions; for other than common conditions, special designs should be made. The contractor should not be permitted to install conduits, pipes, ducts, or sleeves that are not shown on the plans or not approved by the engineer.

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

R6.3.2 The SBC prohibits the use of aluminum in structural concrete unless it is effectively coated or covered. Aluminum reacts with concrete and, in the presence of chloride-ions, may also react electrolytically with steel, causing cracking and/or spatting of the concrete. Aluminum electrical conduits present a special problem since stray electric current accelerates the adverse reaction.

SECTION R6.4
CONSTRUCTION JOINTS

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

R6.4.2 The use of neat cement on vertical joints is not permitted since it is rarely practical and can be detrimental where deep forms and steel congestion prevent proper access. Often wet blasting and other procedures are more appropriate. Because the SBC sets only minimum standards, the engineer may have to specify special procedures if conditions warrant. The degree to which mortar batches are needed at the start of concrete placement depends on concrete proportions, congestion of steel, vibrator access, and other factors.

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

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

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

CHAPTER 7
DETAILS OF REINFORCEMENT

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 the ACI Detailing Manual (Reference 7.1).

All provisions in the SBC 304 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 meter as the ASTM designated bar, wire, or strand sizes. Cross-sectional area of reinforcement is based on nominal dimensions.

SECTION R7.1
STANDARD HOOKS

R7.1.3 Standard stirrup and tie hooks are limited to Dia 25 mm bars and smaller, and the 90-deg hook with $6d_b$ extension is further limited to Dia 16 mm bars and smaller, in both cases as the result of research showing that larger bar sizes with 90-deg hooks and $6d_b$ extensions tend to pop out under high load.

SECTION R7.2
MINIMUM BEND DIAMETERS

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

R7.2.2 The minimum $4d_b$ bend for the bar sizes commonly used for stirrups and ties is based on accepted industry practice. Use of a stirrup bar size not greater than Dia 16 mm for either the 90-deg or 135-deg standard stirrup hook will permit multiple bending on standard stirrup bending equipment.

R7.2.3 Welded wire fabric, of plain or deformed wire, can be used for stirrups and ties. The wire at welded inter-sections does not have the same uniform ductility and bend-ability as in areas which were not heated. These effects of the welding temperature are usually dissipated in a distance of approximately four wire diameters. Minimum bend diameters permitted are in most cases the same as those required in the ASTM bend tests for wire material (ASTM A 82 and A 496).

SECTION R7.3
BENDING

R7.3.1 The engineer may be the design engineer or the engineer employed by the owner to perform inspection. For unusual bends with inside diameters less than ASTM bend test requirements, special fabrication may be required.

R7.3.2 Construction conditions may make it necessary to bend bars that have been embedded in concrete. Such field bending should not be done without authorization of the engineer. The engineer should determine whether the bars should be bent cold or if heating should be used. Bends should be gradual and should be straightened as required.

Tests[7.2, 7.3] have shown that A 615M Grade 300 and Grade 420 reinforcing bars can be cold bent and straightened up to 90 deg at or near the minimum diameter specified in 7.2. If cracking or breakage is encountered, heating to a maximum temperature of 800°C 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 150 mm 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 300°C.

SECTION R7.4
SURFACE CONDITIONS OF REINFORCEMENT

R7.4.3 Guidance for evaluating the degree of rusting on strand is given in Reference 7.4.

SECTION R7.5
PLACING REINFORCEMENT

R7.5.1 Reinforcement, including tendons, and post-tensioning ducts should be adequately supported in the forms to prevent displacement by concrete placement or workers. Beam stirrups should be supported on the bottom form of the beam by positive supports such as continuous longitudinal beam bolsters. If only the longitudinal beam bottom reinforcement is supported, construction traffic can dislodge the stirrups as well as any prestressing tendons tied to the stirrups.

R7.5.2 Generally accepted practice, as reflected in Ref. 7.5 has established tolerances on total depth (formwork or finish) and fabrication of truss bent reinforcing bars and closed ties, stirrups, and spirals. The engineer should specify more restrictive tolerances than those permitted by the SBC 304 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 SBC 304 may be desirable for prestressed concrete to achieve camber control within limits acceptable to the designer or owner. In such cases, the engineer should specify the necessary tolerances. Recommendations are given in Ref. 7.6.

R7.5.2.1 The SBC 304 specifies a tolerance on depth d, an essential component of strength of the member. Because reinforcing steel is placed with respect to edges of members and formwork surfaces, the depth d is not always conveniently measured in the field. Engineers should specify tolerances for bar placement, cover, and member size. See ACI 117.[7.5]

R7.5.4 "Tack" welding (welding crossing bars) can seriously weaken a bar at the point welded by creating a metallurgical notch effect. This operation can be performed safely only when the material welded and welding operations are under continuous competent control, as in the manufacture of welded wire fabric.

SECTION R7.6
SPACING LIMITS FOR REINFORCEMENT

Since the development length is a function of the bar spacing, it may be desirable to use larger than minimum bar spacing in some cases. The minimum limits are 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.

R7.6.6 **Bundled bars.** Bond research[7.7] 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 Dia 32 mm not be bundled in beams or girders is a practical limit for application to building size members. Conformance to the crack control requirements of 10.6 will effectively preclude bundling of bars larger than Dia 32 mm as tensile reinforcement. The SBC phrasing "bundled in contact to act as a unit," is intended to preclude bundling more than two bars in the same plane. Typical bundle shapes are triangular, square, or L-shaped patterns for three- or four-bar bundles. As a practical caution, bundles more than one bar deep in the plane of bending should not be hooked or bent as a unit. Where end hooks are required, it is preferable to stagger the individual bar hooks within a bundle.

R7.6.7 Tendons and ducts

R7.6.7.1 The allowed decreased spacing in this section for transfer strengths of 28 MPa or greater is based on Reference 7.8, 7.9.

R7.6.7.2 When ducts for prestressing steel in a beam are arranged closely together vertically, provision should be made to prevent the prestressing steel from breaking through the duct when tensioned. Horizontal disposition of ducts should allow proper placement of concrete. A clear spacing of one and one-third times the size of the coarse aggregate, but not less than 25 mm, 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.

SECTION R7.7
CONCRETE PROTECTION FOR REINFORCEMENT

Concrete cover as protection of reinforcement against weather and other effects is measured from the concrete surface to the outermost surface of the steel to which the cover requirement applies. Where minimum cover is prescribed for a class of structural member, it is measured to the outer edge of stirrups, ties, or spirals if transverse reinforcement encloses main bars; to the outermost layer of bars if more than one layer is used without stirrups or ties; or to the metal end fitting or duct on post-tensioned prestressing steel.

The condition "concrete surfaces exposed to earth or weather" refers to direct exposure to moisture changes and not just to temperature changes. Slab or thin shell soffits are not usually considered directly exposed unless subject to alternate wetting and drying, including that due to condensation conditions or direct leakage from exposed top surface, run off, or similar effects.

Alternative methods of protecting the reinforcement from weather may be provided if they are equivalent to the additional concrete cover required by the SBC 304. 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 lengths given in Chapter 12 are now a function of the bar cover. As a result, it may be desirable to use larger than minimum cover in some cases.

R7.7.3 **Precast concrete (manufactured under plant control conditions).** The lesser cover thicknesses for precast construction reflect the greater convenience of control for proportioning, placing, and curing inherent in precasting. The term "manufactured under plant control conditions" does not specifically imply that precast members should be manufactured in a plant. Structural elements precast at the job site will also qualify under this section if the control of form dimensions, placing of reinforcement, quality control of concrete, and curing procedure are equal to that normally expected in a plant.

Concrete cover to 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.

R7.7.5 **Corrosive environments.** Where concrete will be exposed to external sources of chlorides in service, such as brackish water, seawater, or spray from these sources, concrete should be proportioned to satisfy the special exposure requirements of Chapter 4. These include maximum water-cementitious materials ratio, minimum strength for normal weight and lightweight concrete, maximum chloride ion in concrete, and cement type. Additionally, for corrosion protection, a minimum concrete cover for reinforcement of 50 mm for walls and slabs and 60 mm for other members is recommended. For precast concrete manufactured under plant control conditions, a minimum cover of 40 and 50 mm, respectively, is recommended.

R7.7.5.1 Corrosive environments are defined in Chapter 4 of this SBC 304. Additional information on corrosion in parking structures is given in Ref. 7.10, "Design of Parking Structures," pp. 21-26.

R7.7.6 **Future extension.** Exposed reinforcements, inserts and plates, etc, intended for bonding with future extensions, should be adequately protected against corrosion and any other corrosive ambient attack. The protection measures can include but may not be limited to the following:

a) for accessible roofs, the column rebars should be bent inside the roof screeds in such a manner that they can be re-exposed and lapped for future extensions, as and when necessary.

b) for non-accessible roofs, the column rebars can be covered inside low-height stub columns of lean concrete, in such manner so that they can be re-exposed and lapped for future extensions, as and when necessary.

c) for all other elements like beams, slab projections, etc. the rebars should desirably be protected in lean concrete, in such manner so that they can be re-exposed and lapped for future extensions, as and when necessary.

SECTION R7.8
SPECIAL REINFORCEMENT DETAILS FOR COLUMNS

R7.8.2 **Steel cores**. The 50 percent limit on transfer of compressive load by end bearing on ends of structural steel cores is intended to provide some tensile capacity at such splices (up to 50 percent), since the remainder of the total compressive stress in the steel core are to be transmitted by dowels, splice plates, welds, etc. This provision should ensure that splices in composite compression members meet essentially the same tensile capacity as required for conventionally reinforced concrete compression members.

SECTION R7.9
CONNECTIONS

Confinement is essential at connections to ensure that the flexural capacity of the members can be developed without deterioration of the joint under repeated loadings.[7.11,7.12]

SECTION R7.10
LATERAL REINFORCEMENT FOR COMPRESSION MEMBERS

R7.10.3 Precast columns with cover less than 40 mm, prestressed columns without longitudinal bars, columns of concrete with small size coarse aggregate, wall-like columns, and other special cases may require special designs for lateral reinforcement. Plain or deformed wire, WD 5.5, or larger, may be used for ties or spirals. If such special columns are considered as spiral columns for load strength in design, the ratio of spiral reinforcement ρ_s is to conform to 10.9.3.

R7.10.4 **Spirals.** For practical considerations in cast-in-place construction, the minimum diameter of spiral reinforcement is 10 mm. This is the smallest size that can be used in a column with 40 mm or more cover and having concrete strengths of 20 MPa or more if the minimum clear spacing for placing concrete is to be maintained.

Standard spiral sizes are 10, 14, and 16 mm diameter for hot rolled or cold drawn material, plain or deformed.

The SBC 304 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 or drop panel. If beams or brackets enclose all sides of the column but are of different depths, the ties should extend from the spiral to the level of the horizontal reinforcement of the shallowest beam or bracket framing into the column. These additional ties are to enclose the longitudinal column reinforcement and the portion of bars from beams bent into the column for anchorage. See also 7.9.

Spirals should be held firmly in place, at proper pitch and alignment, to prevent displacement during concrete placement. When spacers are used, the following may be used for guidance: For spiral bar or wire smaller than 16 mm diameter, a minimum of two spacers should be used for spirals less than 0.5 m in diameter, three pacers for spirals 0.5 to 0.75 m in diameter, and four spacers for spirals greater than 0.75 m in diameter. For spiral bar or wire 16 mm diameter or larger, a minimum of three spacers should be used for spirals 600 mm or less in diameter, and four spacers for spirals greater than 600 mm in diameter. The project specifications or subcontract agreements should be clearly written to cover the supply of spacers or field tying of the spiral reinforcement.

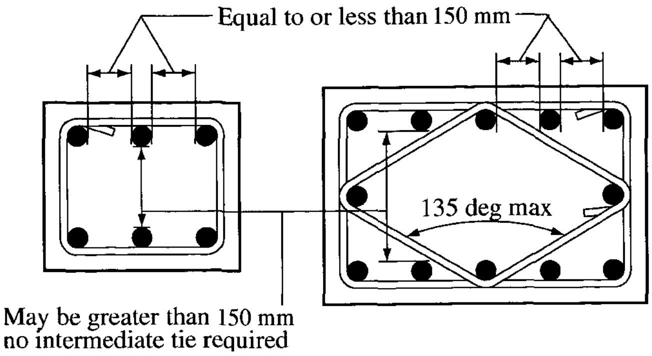


Fig. R7.10.5 - Sketch to clarify measurements between laterally supported column bars.

R7.10.5 **Ties.** All longitudinal bars in compression should be enclosed within lateral 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 (see also 7.10.4.3).

Since spliced bars and bundled bars were not included in the tests of Reference 7.13, is prudent to provide a set of ties at each end of lap spliced bars, above and below end-bearing splices, and at minimum spacings immediately below sloping regions of offset bent bars.

Standard tie hooks are intended for use with deformed bars only, and should be staggered where possible. See also 7.9.

Continuously wound bars or wires can be used as ties provided their pitch and area are at least equivalent to the area and spacing of separate ties. Anchorage at the end of a continuously wound bar or wire should be by a standard hook as for separate bars or by one additional turn of the tie pattern. A circular continuously wound bar or wire is considered a spiral if it conforms to 7.10.4, otherwise it is considered a tie.

R7.10.5.5 Ties may be terminated only when elements frame into all four sides of square and rectangular columns; for round or polygonal columns, such elements frame into the column from four directions

R7.10.5.6 Confinement of anchor bolts that are placed in the top of columns or pedestal 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

SECTION R7.11
LATERAL REINFORCEMENT FOR FLEXURAL MEMBERS

R7.11.1 Compression reinforcement in beams and girders should be enclosed to prevent buckling; similar requirements for such enclosure have remained essentially unchanged through several editions of the SBC 304, except for minor clarification.

SECTION R7.12
SHRINKAGE AND TEMPERATURE REINFORCEMENT

R7.12.1 Shrinkage and temperature reinforcement is required at right angles to the principal reinforcement to minimize cracking and to tie the structure together to ensure its acting as assumed in the design. The provisions of this section are intended for structural slabs only; they are not intended for soil supported slabs on grade.

R7.12.1.2 The area of shrinkage and temperature reinforcement required by 7.12 has been satisfactory where shrinkage and temperature movements are permitted to occur. For cases where structural walls or large columns provide significant restraint to shrinkage and temperature movements, it may be necessary to increase the amount of reinforcement normal to the flexural reinforcement in 7.12.1.2 (see Reference 7.14). Top and bottom reinforcement are both effective in controlling cracks. Control strips during the construction period, which permit initial shrinkage to occur without causing an increase in stresses, are also effective in reducing cracks caused by restraint.

R7.12.2 The amounts specified given for deformed bars and welded wire fabric are empirical but have been used satisfactorily for many years. Splices and end anchorages of shrinkage and temperature reinforcement are to be designed for the full specified yield strength in accordance with 12.1, 12.15, 12.18, and 12.19.

R7.12.3 Prestressed reinforcement requirements have been selected to provide an effective force on the slab approximately equal to the yield strength force for nonprestressed shrinkage and temperature reinforcement. This amount of prestressing, 1.0 MPa on the gross concrete area, has been successfully used on a large number of projects. When the spacing of tendons used for shrinkage and temperature reinforcement exceeds 1.4 m, additional bonded reinforcement is required at slab edges where the prestressing forces are applied in order 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. Application of the provisions of 7.12.3 to monolithic cast in-place post-tensioned beam and slab construction is illustrated in Fig. R7.12.3.

Tendons used for shrinkage and temperature reinforcement should be positioned vertically in the slab as close as practicable to the center 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 designer should evaluate the effects of slab shortening to ensure proper action. In most cases, the low level of prestressing recommended should not cause difficulties in a properly detailed structure. Special attention may be required where thermal effects become significant.

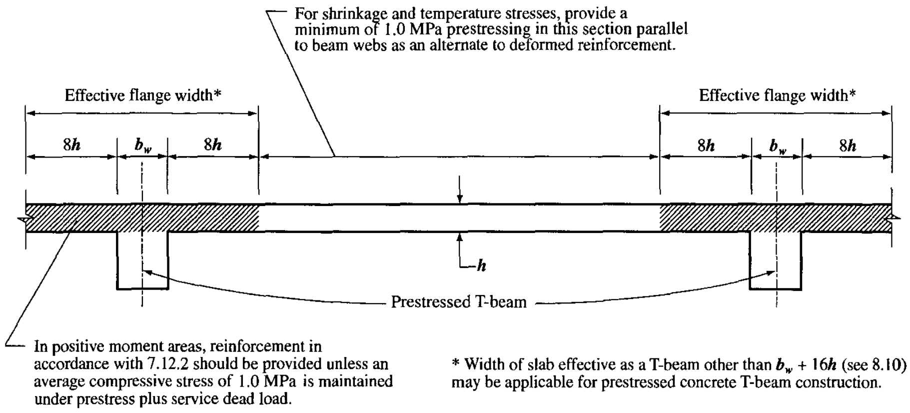


Fig. R7.12.3 - Prestressing used for shrinkage and temperature.

SECTION R7.13
REQUIREMENTS FOR STRUCTURAL INTEGRITY

Experience has shown that the overall integrity of a structure can be substantially enhanced by minor changes in detailing of reinforcement. It is the intent of this section of the SBC 304 to improve the redundancy and ductility in structures so that in the event of damage to a major supporting element or an abnormal loading event, the resulting damage may be confined to a relatively small area and the structure will have a better chance to maintain overall stability.

R7.13.2 With damage to a support, top reinforcement hat is continuous over the support, but not confined by stirtrups, will tend to tear out of the concrete and will not provide the catenary action needed to bridge the damaged support. By making a portion of the bottom reinforcement continuous, catenary action can be provided.

Requiring continuous top and bottom reinforcement in perimeter or spandrel beams provides a continuous tie around the structure. It is not the intent to require a tensile tie of continuous reinforcement of constant size around the entire perimeter of a structure, but simply to require that one half of the top flexural reinforcement required to extend past the point of inflection by 12.12.3 be further extended and spliced at or near midspan. Similarly, the bottom reinforcement required to extend into the support by 12.11.1 should be made continuous or spliced with bottom reinforcement from the adjacent span. If the depth of a continuous beam changes at a support, the bottom reinforcement in the deeper member should be terminated with a standard hook and bottom reinforcement in the shallower member should be extended into and fully developed in the deeper member.

R7.13.3 The SBC 304 requires tension ties for precast concrete buildings of all heights. Details should provide connections to resist applied loads. Connection details that rely solely on friction caused by gravity forces are not permitted.

Connection details should be arranged so as to minimize the potential for cracking due to restrained creep, shrinkage and temperature movements. For information on connections and detailing requirements, see Reference 7.15.

Ref. 7.16 recommends minimum tie requirements for precast concrete bearing wall buildings.

CHAPTER 8
ANALYSIS AND DESIGN GENERAL CONSIDERATIONS

SECTION R8.0
NOTATION

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

The definition of net tensile strain in 2.1 excludes strains due to effective prestress, creep, shrinkage, and temperature.

SECTION 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 (design strength).

R8.1.2 Designs in accordance with Appendix B are equally acceptable, provided the provisions of Appendix B are used in their entirety.

R8.1.3 The SBC 304 includes specific provisions for anchoring to concrete in Appendix D of SBC 304.

SECTION R8.2
LOADING

The provisions in the SBC 304 are for all types of loads as specified in SBC 301.
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.

R8.2.3 Any reinforced concrete wall that is monolithic with other structural elements is considered to be an "integral part." Partition walls may or may not be integral structural parts. If partition walls may be removed, the primary lateral load resisting system should provide all of the required resistance without contribution of the removable partition. However, the effects of all partition walls attached to the structure should be considered in the analysis of the structure because they may lead to increased design forces in some or all elements. Special provisions for seismic design are given in Chapter 21.

R8.2.4 Information is accumulating on the magnitudes of these various effects, especially the effects of column creep and shrinkage in tall structures,[8.1] and on procedures for including the forces resulting from these effects in design.

SECTION R8.3
METHOD OF ANALYSIS

R8.3.1 Factored loads are service loads multiplied by appropriate load factors. For the strength design method, elastic analysis is used to obtain moments, shears, and reactions.

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.

These moment coefficients were derived by elastic analysis assuming rigid supports and considering alternative placement of live load to yield maximum negative or positive moments at the critical sections. For one-way slab systems, a supporting beam or girder may be considered rigid if it is supported by columns or walls and has a total depth not less than about three times the solid slab thickness or three times the equivalent joist-slab thickness.

R8.3.4 The strut-and-tie model in Appendix A 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 flexure 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.

SECTION R8.4
REDISTRIBUTION OF NEGATIVE MOMENTS IN CONTINUOUS FLEXURAL MEMBERS

Moment redistribution is dependent on adequate ductility in plastic hinge regions. These plastic hinge regions develop at points of maximum moment and cause a shift in the elastic moment diagram. The usual result is a reduction in the values of negative moments in the plastic hinge region and an increase in the values of positive moments from those computed by elastic analysis. Because negative moments are determined for one loading arrangement and positive moments for another, each section has a reserve capacity that is not fully utilized for any one loading condition. The plastic hinges permit the utilization of the full capacity of more cross sections of a flexural member at ultimate loads.

Using conservative values of limiting concrete strains and lengths of plastic hinges derived from extensive tests, flexural members with small rotation capacity were analyzed for moment redistribution up to 20 percent, depending on the reinforcement ratio. The results were found to be conservative. Studies (reference 8.2 and 8.3) 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 SBC 304 is available if the members satisfy the SBC 304 requirements.

Moment redistribution may not be used for slab systems designed by the Direct Design Method (see 13.6.1.7).

The SBC 304 specifies the permissible redistribution percentage in terms of the net tensile strain ε_t. Reference 8.4 compares moment redistribution percentage in terms of reinforcement indices and the net tensile strain ε_t.

SECTION R8.5
MODULUS OF ELASTICITY

R8.5.1 Studies leading to the expression for modulus of elasticity of concrete in 8.5.1 are summarized in Reference 8.5 where, E_c, was defined as the slope of the line drawn from a stress of zero to a compressive stress of $0.45 f_c^{'}$. The modulus for concrete is sensitive to the modulus of the aggregate and may differ from the specified value. Measured values range typically from 120 to 80 percent of the specified value. Methods for determining Young's modulus for concrete are described in Reference 8.6.

SECTION R8.6
STIFFNESS

R8.6.1 Ideally, the member stiffnesses *EI* and *GJ* should reflect the degree of cracking and inelastic action that has occurred along each member before yielding. However, the complexities involved in selecting different stiffnesses for all members of a frame would make frame analyses inefficient in design offices. Simpler assumptions are required to define flexural and torsional stiffnesses. For braced frames, relative values of stiffness are important. Two usual assumptions are to use gross *EI* values for all members or, to use half the gross *EI* of the beam stem for beams and the gross *EI* for the columns. For frames that are free to sway, a realistic estimate of *EI is* desirable and should be used if second-order analyses are carried out. Guidance for the choice of *EI* for this case is given in R10.11.1.

Two conditions determine whether it is necessary to consider torsional stiffness in the analysis of a given structure: (1) the relative magnitude of the torsional and flexural stiffnesses, and (2) whether torsion is required for equilibrium of the structure (equilibrium torsion) or is due to members twisting to maintain deformation compatibility (compatibility torsion). In the case of compatibility torsion, the torsional stiffness may be neglected. For cases involving equilibrium torsion, torsional stiffness should be considered.

R8.6.2 Stiffness and fixed-end moment coefficients for haunched members may be obtained from Reference 8.7.

SECTION R8.7
SPAN LENGTH

Beam moments calculated at support centers may be reduced to the moments at support faces for design of beams. Reference 8.8 provides an acceptable method of reducing moments at support centers to those at support faces.

SECTION R8.8
COLUMNS

Section 8.8 has been developed with the intent of making certain that the most demanding combinations of axial load and moments be identified for design.
Section 8.8.4 has been included to make certain that moments in columns are recognized in the design if the girders have been proportioned using 8.3.3. The moment in 8.8.4 refers to the difference between the moments in a given vertical plane, exerted at column centerline by members framing into that column.

SECTION R8.9
ARRANGEMENT OF LIVE LOAD

For determining column, wall, and beam moments and shears caused by gravity loads, the SBC 304 permits the use of a model limited to the beams in the level considered and the columns above and below that level. Far ends of columns are to be considered as fixed for the purpose of analysis under gravity loads. This assumption does not apply to lateral load analysis. 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 engineer is expected to establish the most demanding sets of design forces 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 10.11, 10.12, and 10.13.

SECTION R8.10
T- BEAM CONSTRUCTION

Special provisions related to T-beams and other flanged members are stated in 11.6.1 with regard to torsion.

SECTION R8.11
JOIST CONSTRUCTION

The size and spacing limitations for concrete joist construction meeting the limitations of 8.11.1 through 8.11.3 are based on practice.

R8.11.3 A limit on the maximum spacing of ribs is required because of the special provisions permitting higher shear strengths and less concrete protection for the reinforcement for these relatively small, repetitive members.

R8.11.8 The increase in shear strength permitted by 8.11.8 is justified on the basis of:

(1) satisfactory performance of joist construction with higher shear strengths and,

(2) redistribution of local overloads to adjacent joists.

SECTION R8.12
SEPARATE FLOOR FINISH

The SBC 304 does not specify an additional thickness for wearing surfaces subjected to unusual conditions of wear. The need for added thickness for unusual wear is left to the discretion of the designer.

A floor finish may be considered for strength purposes only if it is cast monolithically with the slab. Permission is given to include a separate finish in the structural thickness if composite action is provided for in accordance with Chapter 17.

All floor finishes may be considered for nonstructural purposes such as cover for reinforcement, fire protection, etc. Provisions should be made, however, to ensure that the finish will not spall off, thus causing decreased cover. Furthermore, development of reinforcement considerations require minimum monolithic concrete cover according to 7.7.

CHAPTER 9
STRENGTH AND SERVICEABILITY REQUIREMENTS

SECTION R9.0
NOTATION

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

The definition of net tensile strain in 2.1 excludes strains due to effective prestress, creep, shrinkage, and temperature.

SECTION R 9.1
GENERAL

Chapter 9 defines the basic strength and serviceability conditions for proportioning structural 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.

SECTION 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 Saudi building SBC 304 for loading 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 SBC 304 gives load factors for specific combinations of loads. In assigning factors to combinations of loading, some consideration is given to the probability of simultaneous occurrence. While most of the usual combinations of loadings are included, the designer should not assume that all cases are covered.

Due regard is to be given to sign in determining U for combinations of loadings, as one type of loading may produce effects of opposite sense to that produced by another type. The load combinations with $0.9D$ are specifically included for the case where a higher dead load reduces the effects of other loads. The loading case may also be critical for tension-controlled column sections. In such a case, a reduction in axial load and an increase in moment may result in a critical load combination.

Consideration should be given to various combinations of loading to determine the most critical design condition. This is particularly true when strength is dependent on more than one load effect, such as strength for combined flexure and axial load or shear strength in members with axial load.

If special circumstances require greater reliance on the strength of particular members than encountered in usual practice, some reduction in the stipulated strength reduction factors ϕ or increase in the stipulated load factors U may be appropriate for such members.

R9.2.2 If the live load is applied rapidly, as may be the case for parking structures, loading docks, warehouse floors, elevator shafts, etc., impact effects should be considered. In all equations, substitute (L + impact) for L when impact should be considered.

R9.2.3 The designer should consider the effects of differential settlement, creep, shrinkage, temperature, and shrinkage-compensating concrete. The term realistic assessment is used to indicate that the most probable values rather than the upper bound values of the variables should be used.

R9.2.4 The load factor of 1.2 applied to the maximum tendon jacking force results in a design load of about 113 percent of the specified prestressing steel yield strength but 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.

SECTION 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 the SBC 304 multiplied by a strength reduction factor ϕ, which is always less than one.

The purposes of the strength reduction factor ϕ are (1) to allow for the probability of understrength members due to variations in material strengths and dimensions, (2) to allow for inaccuracies in the design equations, (3) to reflect the degree of ductility and required reliability of the member under the load effects being considered, and (4) to reflect the importance of the member in the structure.[9.1,9.2]

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 The magnitude of the ϕ factor is 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 since 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. The concept of net tensile strain ε_t is discussed in R10.3.3.

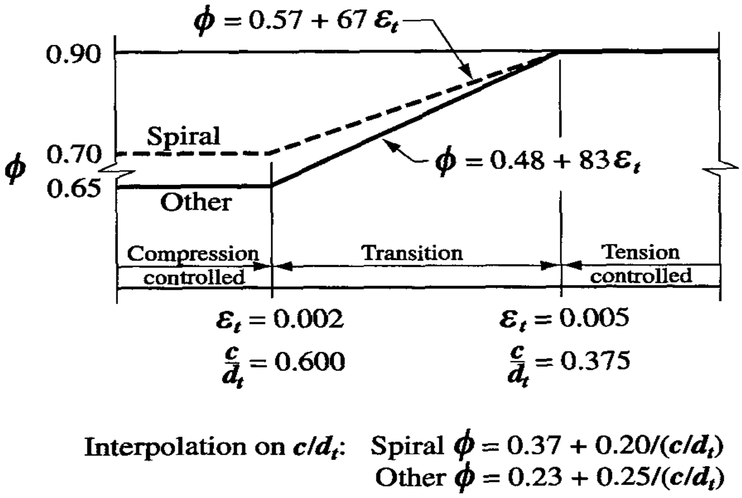


Fig. R9.3.2 - Variation of ϕ with net tensile ε_t and c/d_t for Grade 420 reinforcement and for prestressing steel.

Since the compressive strain in the concrete at nominal strength is assumed in 10.2.3 to be 0.003, the net tensile strain limits for compression-controlled members may also be stated in terms of the ratio c/d_t, where c is the depth of the neutral axis at nominal strength, and d_t is the distance from the extreme compression fiber to the extreme tension steel. The c/d_t limits for compression-controlled and tension-controlled sections are 0.6 and 0.375, respectively. The 0.6 limit applies to sections reinforced with Grade 420 steel and to prestressed sections. Fig. R9.3.2 also gives equations for ϕ as a function of c/d_t.

The net tensile strain limit for tension-controlled sections may also be stated in terms of the ρ/ρ_b. The net tensile strain limit of 0.005 corresponds to a ρ/ρ_b ratio of 0.63 for rectangular sections with Grade 420 reinforcement.

R9.3.2.5 The ϕ factor of 0.85 reflects the wide scatter of results of experimental anchorage zone studies. Since 18.13.4.2, limits the nominal compressive strength of unconfined concrete: in the general zone to $0.7\lambda f'_{ci}$ the effective design strength for unconfined concrete is $0.85 \times 0.7\lambda f'_{ci} = 0.6\lambda f'_{ci}$.

R9.3.2.6 The ϕ factor used in strut-and-tie models is taken equal to the ϕ factor for shear. The value of 4, for strut and-tie models is applied to struts, ties, and bearing areas in such models.

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.

R9.3.4 Strength reduction factors in 9.3.4 are intended to compensate for uncertainties in estimation of strength of structural members in buildings. They are based primarily on experience with constant or steadily increasing applied load. For construction in regions of high seismic risk, some of the strength reduction factors have been modified in 9.3.4 to account for the effects of displacement reversals into the nonlinear range of response on strength.

Section 9.3.4(a) refers to brittle members such as low-rise walls, portions of walls between openings, or diaphragms that are impractical to reinforce to raise their nominal shear strength above nominal flexural strength for the pertinent loading conditions.

Short structural walls were the primary vertical elements of the lateral-force-resisting system in many of the parking structures that sustained damage during the 1994 Northridge earthquake. Section 9.3.4(b) requires the shear strength reduction factor for diaphragms to be 0.60 if the shear strength reduction factor for the walls is 0.60.

R9.3.5 The strength reduction factor ϕ for structural plain concrete design is the same for all strength conditions. Since both flexural tension strength and shear strength for plain concrete depend on the tensile strength characteristics of the concrete, with no reserve strength or ductility possible due to the absence of reinforcement, equal strength reduction factors for both bending and shear are considered appropriate.

SECTION R9.4
DESIGN STRENGTH FOR REINFORCEMENT

In addition to the upper limit of 550 MPa for yield strength of non-prestressed reinforcement, there are limitations on yield strength in other sections of the SBC 304.

In 11.5.2, 11.6.3.4, and 11.7.6, the maximum f_y that may be used in design for shear and torsion reinforcement is 420 MPa, except that f_y up to 550 MPa may be used for shear reinforcement meeting the requirements of ASTM A 497.

In 19.3.2 and 21.2.5, the maximum specified f_y is 420 MPa in shells, folded plates, and structures governed by the special seismic provisions of Chapter 21.

The deflection provisions of 9.5 and the limitations on distribution of flexural reinforcement of 10.6 become increasingly critical as f_y increases.

SECTION 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. When 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.[9.3] 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 SBC 304 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 SBC 304.

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 sections of the SBC 304, and are limited to the values in Table 9.5(b).

TABLE 9.5(a)-MINIMUM THICKNESS OF NON-PRESTRESSED 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:

1) Span length ℓ is in mm.
2) Values given shall be used directly for members with normal weight concrete (w_c=2300 kg/m^3) and Grade 420 reinforcement For other conditions, the values shall be modified as follows:
   a) For structural lightweight concrete having unit weight in the range 1500-2000 kg/m³ , the values shall be multiplied by (1.65 - 0.0003w_c) but not less than 1.09, where w_c is the unit weight in kg/m^3 .
   b) For f_y other than 420 MPa, the values shall be multiplied by (0.4 + f_y/700).

R9.5.2 One-way construction (nonprestressed)

R9.5.2.1 The minimum thicknesses of Table 9.5(a) apply for nonprestressed beams and one-way slabs (see 9.5.2), and for composite members (see 9.5.5). 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 should be modified if other than normal weight concrete and Grade 420 reinforcement are used. The notes beneath the table are essential to its use for reinforced concrete members constructed with structural lightweight concrete or with reinforcement having a yield strength other than 420 MPa. If both of these conditions exist, the corrections in footnotes (a) and (b) should both be applied.

The modification for lightweight concrete in footnote (a) is based on studies of the results and discussions in Reference 9.14. No correction is given for concretes weighing between 1900 and 2300 kg/m^3 because the correction term would be close to unity in this range.

The modification for yield strength in footnote (b) is approximate but should yield conservative results for the type of members considered in the table, for typical reinforcement ratios, and for values of f_y between 300 and 550 MPa.

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 $E_c I_g$ 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 SBC 304 and developed in Reference 9.5 was selected as. being sufficiently accurate for use to control deflections.[9.6-9.8] 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 cases I_e will be less than I_g.

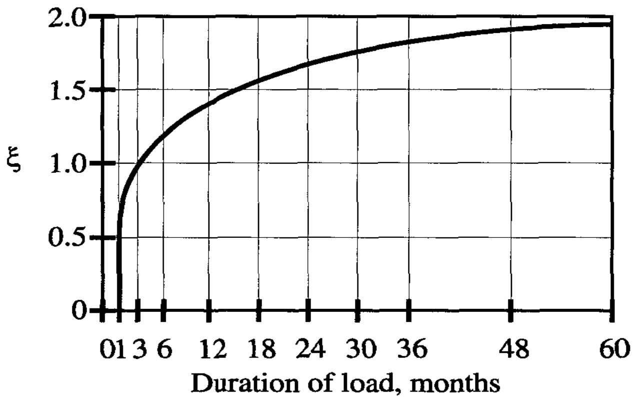


Fig. R9.5.2.5-Multipliers for long-term deflections

R9.5.2.4 For continuous members, the SBC 304 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 in References 9.3, 9.9 and 9.10.

R9.5.2.5 Shrinkage and creep due to sustained loads cause additional long-term deflections over and above those which 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, and magnitude of the sustained load. The expression given in this section is considered satisfactory for use with the SBC 304 procedures for the calculation of immediate deflections, and with the limits given in Table 9.5(b). 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.

Eq. (9-11) was developed in Reference 9.19. In Eq. (9-11) the multiplier on ζ accounts for the effect of compression reinforcement in reducing long-term deflections. $\zeta = 2.0$ 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 ζ for loading periods less than five years.

If it is desired to consider creep and shrinkage separately, approximate equations provided in References 9.5, 9.6, 9.11, and 9.12 may be used.

R9.5.2.6 It should be noted that the limitations given in this table relate only to supported or attached nonstructural elements. For those structures in which structural members are likely to be affected by deflection or deformation of members to which they are attached in such a manner as to affect adversely the strength of the structure, these deflections and the resulting forces should be considered explicitly in the analysis and design of the structures as required by 9.5.1. (See Reference 9.8.)

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.

R9.5.3 Two-way construction (nonprestressed)

R9.5.3.2 The minimum thicknesses in Table 9.5(c) are those that have been developed through the years. Slabs conforming to those limits have not resulted in systematic problems related to stiffness for short- and long-term loads. These limits apply to only the domain of previous experience in loads, environment, materials, boundary conditions, and spans.

R9.5.3.3 For panels having a ratio of long to short span greater than 2, the use of Eq. (9-12) and (9-13), 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.

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_g specified in 9.5.2.3 may be used.[9.16] However, other procedures and other values of the stiffness EI may be used if they result in predictions of deflection in reasonable agreement with the results of comprehensive tests.

Since 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 SBC 304 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.

TABLE 9.5(b) - MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to non-structural 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

Yield strength, f_y, MPa*	Without drop panels†			With drop panels†		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams‡		Without edge beams	With edge beams‡	
300	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{40}$	$\frac{\ell_n}{40}$
420	$\frac{\ell_n}{30}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$
520	$\frac{\ell_n}{28}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{34}$	$\frac{\ell_n}{34}$

* For values of reinforcement yield strength between the values given in the table, minimum thickness shall be determined by linear interpolation.

† Drop panel is defined in 13.3.7.1 and 13.3.7.2.

‡ Slabs with beams between columns along exterior edges. The value of α for the edge beam shall not be less than 0.8.

R9.5.4.2 Class C and Class T prestressed flexural members are defined in 18.3.3. Reference 9.13 gives information on deflection calculations using a bilinear moment-deflection relationship and using an effective moment of inertia. Reference 9.14 gives additional information on deflection of cracked prestressed concrete members.

Reference 9.15 shows that the I_e method can be used to compute deflections of Class T prestressed members loaded above the cracking load. For this case, the cracking moment should take into account the effect of prestress. A method for predicting the effect of nonprestressed tension steel in reducing creep camber is also given in Reference 9.15, with approximate forms given in References 9.8 and 9.16.

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 members shorten more with time than similar nonprestressed members due to the precompression in the slab or beam which causes axial creep. This creep together with concrete shrinkage results in 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 steel, 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 that is 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 increase tensile stresses in the prestressed member.

Any suitable method for calculating long-term deflections of prestressed members may be used, provided all effects are considered. Guidance may be found in References 9.8, 9.17, 9.18, and 9.19.

R9.5.5 **Composite construction.** A composite construction is defined as a type of construction using members produced by combining different (e.g., concrete and structural steel), members produced by combining cast-in-place and precast concrete, or cast-in-place concrete elements constructed in separate placements but so interconnected that the combined components act together as a single member and respond to loads as a unit.

Since 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 9.5.2 are based 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 are to 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.)

CHAPTER 10
FLEXURAL AND AXIAL LOADS

SECTION R10.0
NOTATION

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

d_t = distance from extreme compression fiber to extreme tension steel, mm

The definition of net tensile strain in 2.1 excludes strains due to effective prestress, creep, shrinkage, and temperature.

SECTION R10.2
DESIGN ASSUMPTIONS

R10.2.1 The strength of a member computed by the strength design method of the SBC 304 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 should 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.
Both the strains in reinforcement and in concrete are 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. However, the strain at which ultimate moments are developed 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 200,000 MPa (see 8.5.2).

R10.2.5 The tensile strength of concrete in flexure (modulus of rupture) is a more variable property than the compressive strength and is about 10 to 15% 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 SBC 304 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 SBC 304 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 SBC 304 allows the use of an equivalent rectangular compressive stress distribution (stress block) to replace the more exact concrete stress distribution. In the equivalent rectangular stress block, an average stress of $0.85 f_c^{'}$ is used with a rectangle of depth $a = \beta_1 c$. The β_1 of 0.85 for concrete with $f_c^{'} \leq$ 30 MPa and 0.05 less for each 7 MPa of $f_c^{'}$ in excess of 30 was determined experimentally.

Research data from tests with high strength concretes[10.1,10.2] supported the equivalent rectangular stress block for concrete strengths exceeding 55 MPa, with a β_1 equal to 0.65.

The equivalent rectangular stress distribution does not represent the actual stress distribution in the compression zone at ultimate, but does provide essentially the same results as those obtained in tests.[10. 3]

SECTION R10.3
GENERAL PRINCIPLES AND REQUIREMENTS

R10.3.1 For members subject to flexure or combined flexure and axial load, derivations of design strength equations are given in Reference 10.3 for rectangular cross section as well as for other cross sections**.**

R10.3.2 A balanced strain condition exists at a cross section when the maximum strain at the extreme compression fiber just reaches 0.003 simultaneously with the first yield strain f_y / E_s in the tension reinforcement. The reinforcement ratio ρ_b which produces balanced conditions under flexure, depends on the shape of the cross section and the location of the reinforcement.

R10.3.3 The nominal flexural strength of a member is reached when the strain in the extreme compression fiber reaches the assumed strain limit 0.003. The net tensile strain ε_t is the tensile strain in the extreme tension steel at nominal strength, exclusive of strains due to prestress, creep, shrinkage, and temperature. The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength, shown in Fig. R10.3.3, using similar triangles.

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, whereas compression members are usually compression-controlled. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections. Section 9.3.2 specifies the appropriate strength reduction factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

Before the development of these provisions, the limiting tensile strain for flexural members was not stated, but was implicit in the maximum tension reinforcement ratio that was given as a fraction of ρ_b, which was dependent on the yield strength of the reinforcement. The net tensile strain limit of 0.005 for tension-controlled sections was chosen to be a single value that applies to all types of steel (prestressed and nonprestressed) permitted by this SBC 304.

Unless unusual amounts of ductility are required, the 0.005 limit will provide ductile behavior for most designs. One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames. Section 8.4 permits redistribution of negative moments. Since 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.

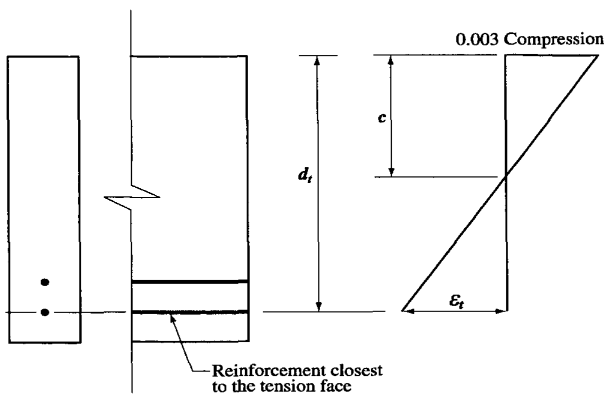


Fig. R10.3.3 - Strain distribution and net tensile strain.

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.

R10.3.5 The effect of this limitation is to restrict the reinforcement ratio in nonprestressed beams. The reinforcement limit of $0.75\rho_b$ results in a net tensile strain at nominal strength of 0.00376. The proposed limit of 0.005 is more conservative. This limitation does not apply to prestressed members.

R10.3.6 and R10.3.7 The specified minimum eccentricities are intended to serve as a means of reducing the axial load design strength of a section in pure compression to account for accidental eccentricities not considered in the analysis that may exist in a compression member, and to recognize that concrete strength may be less than $f_c^{'}$ under sustained high loads. The primary purpose of the minimum eccentricity requirement is to limit the maximum design axial load strength of a compression member. This is accomplished directly in 10.3.6 by limiting the design axial load strength of a section in pure compression to 85 or 80 percent of the nominal strength. These percentage values approximate the axial load strengths at e/h ratios of 0.05 and 0.10, for the spirally reinforced and tied members, respectively. The same axial load limitation applies to both cast-in-place and precast compression members. Design aids and computer programs based on the minimum eccentricity requirement are equally applicable.

For prestressed members, the design axial load strength in pure compression is computed by the strength design methods of Chapter 10, including the effect of the prestressing force.

Compression member end moments should be considered in the design of adjacent flexural members. In nonsway frames, the effects of magnifying the end moments need not be considered in the design of the adjacent beams. In sway frames, the magnified end moments should be considered in designing the flexural members, as required in 10.13.7.

Corner and other columns exposed to known moments about each axis simultaneously should be designed for biaxial bending and axial load. Satisfactory methods are available in References 10.4 and 10.5. The reciprocal load method (Reference 10.6) and the load contour method (Reference 10.7) are the methods used in those two hand-books. Research [10.8, 10.9] indicates that using the equivalent rectangular stress block provisions of 10.2.7 produces satisfactory strength estimates for doubly symmetric sections. A simple and somewhat conservative estimate of nominal strength P_{ni} can be obtained from the reciprocal load relationship.[10.6]

$$\frac{1}{P_{ni}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_o}$$

where:

P_{ni} = nominal axial load strength at given eccentricity along both axes

P_o = nominal axial load strength at zero eccentricity

P_{nx} = nominal axial load strength at given eccentricity along x-axis

P_{ny} = nominal axial load strength at given eccentricity along y-axis

This relationship is most suitable when values P_{nx} and P_{ny} are greater than the balanced axial force P_b for the particular axis.

SECTION R10.4
DISTANCE BETWEEN LATERAL SUPPORTS OF FLEXURAL MEMBERS

Tests[10.10,10.11] have shown that laterally unbraced reinforced concrete beams of any reasonable dimensions, even when very deep and narrow, will not fail prematurely by lateral buckling provided the beams are loaded without lateral eccentricity that causes 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 loading conditions.

SECTION 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.

R10.5.2 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 for slabs should be equal to the same amount as that required by 7.12 for shrinkage and temperature reinforcement.

Soil-supported slabs such as slabs on grade are not considered to be structural slabs in the context of this section, unless they transmit vertical loads from other parts of the structure to the soil. Reinforcement, if any, in soil-supported slabs should be proportioned with due consideration of all design forces. Mat foundations and other slabs that help support the structure vertically should meet the requirements of this section.

In reevaluating the overall treatment of 10.5, the maximum spacing for reinforcement in structural slabs (including footings) was reduced from the $4h$ for temperature and shrinkage reinforcement to the compromise value of $3h$, which is somewhat larger than the $2h$ limit of 13.3.2 for two-way slab systems.

SECTION R10.6 DISTRIBUTION OF FLEXURAL REINFORCEMENT IN BEAMS AND ONE-WAY SLABS

R10.6.1 Many structures designed by working stress methods and with low steel stress served their intended functions with very limited flexural cracking. When high strength reinforcing steels are used at high service load stresses, however, visible cracks should be expected, and steps should 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 300 MPa is used. Current good detailing practices will usually lead to adequate crack control even when reinforcement of 420 MPa yield is used.

Extensive laboratory work[10.12-10.14] involving deformed bars has confirmed that crack width at service loads is proportional to steel stress. The significant variables reflecting steel detailing were found to be thickness of concrete cover and the area of concrete in the zone of maximum tension surrounding each individual reinforcing bar.

Crack width is inherently subject to wide scatter even in careful laboratory work and is influenced by shrinkage and other time-dependent effects. The better crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

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 To control flexural cracks, the maximum bar spacing is specified in Reference 10.15, 10.16, and 10.17. For the usual case of beams with Grade 420 reinforcement and 50 mm clear cover to the main reinforcement, with f_s = 250 MPa, the maximum bar spacing is 300 mm

Crack widths in structures are highly variable. In previous SBC 304s, provisions were given for distribution of reinforcement that were based on empirical equations using a calculated maximum crack width of 0.4 mm. 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 (Reference 10.18 and 10.19) shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels. For this reason, no distinction between interior and exterior exposure is made.

R10.6.5 Although a number of studies have been conducted, clear experimental evidence is not available regarding the crack width beyond which a corrosion danger exists. Exposure tests indicate that concrete quality, adequate consolidation, and ample concrete cover may be of greater importance for corrosion protection than crack width at the concrete surface.

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 one-tenth limitation is to guard against too wide a spacing, with some additional reinforcement required to protect the outer portions of the flange.

R10.6.7 For relatively deep flexural members, some reinforcement should be placed near the vertical faces of the tension zone to control cracking in the web.[10.16] (See Fig. R10.6.7.) Without such auxiliary steel, the width of the cracks in the web may exceed the crack widths at the level of the flexural tension reinforcement.
Where the provisions for deep beams, walls, or precast panels require more steel, those provisions (along with their spacing requirements) will govern.

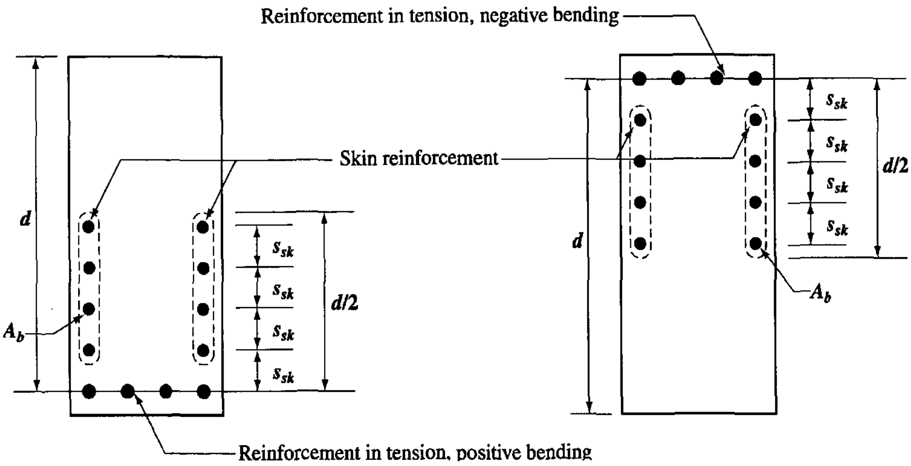


Fig. R10.6.7 - Skin reinforcement of beams and joists with d > 900mm

SECTION R10.7
DEEP BEAMS

Are based on D-region behavior (see Appendix A in SBC 304).
SBC 304 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 References 10.20, 10.21, and 10.22.

SECTION R10.8
DESIGN DIMENSIONS FOR COMPRESSION MEMBERS

The engineer should recognize the need for careful workmanship, as well as the increased significance of shrinkage stresses with small sections.

R10.8.2-
R10.8.4 For column design,[10.23] the SBC 304 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 should 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.

SECTION R10.9
LIMITS FOR REINFORCEMENT OF COMPRESSION MEMBERS

R10.9.1 This section prescribes the limits on the amount of longitudinal reinforcement for noncomposite compression members. If the use of high reinforcement ratios would involve practical difficulties in the placing of concrete, a lower percentage and hence a larger column, or higher strength concrete or reinforcement (see R9.4) should be considered. The percentage of reinforcement in columns should usually not exceed 4 percent if the column bars are required to be lap spliced.

Minimum reinforcement - Since the design methods for columns incorporate separate terms for the load carried by concrete and by reinforcement, it is necessary to specify some minimum amount of reinforcement to ensure that only reinforced concrete columns are designed by these procedures. Reinforcement is necessary to provide resistance to bending, which may exist whether or not computations show that bending exists, and to reduce the effects of creep and shrinkage of the concrete under sustained compressive stresses. Tests have shown that creep and shrinkage tend to transfer load from the concrete to the reinforcement, with a consequent increase in stress in the reinforcement, and that this increase is greater as the ratio of reinforcement decreases. Unless a lower limit is placed on this ratio, the stress in the reinforcement may increase to the yield level under sustained service loads. This phenomenon was emphasized in Reference 10.24 and minimum reinforcement ratios of 0.01 and 0.005 were recommended for spiral and tied columns, respectively. However, in this SBC 304, the minimum ratio is 0.01 for both types of laterally reinforced columns.

Maximum reinforcement - Extensive tests of column investigation (Reference 10.24) 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 Reference 10.24 for spiral and tied columns, respectively. In this SBC 304, it is required that bending be considered in the design of all columns, and the maximum ratio of 0.08 is applied to both types of columns. This limit can be considered a practical maximum for reinforcement in terms of economy and requirements for placing.

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.

R10.9.2 For compression members, a minimum of four longitudinal bars are required when bars are enclosed by rectangular or circular ties. For other shapes, one bar should be provided at each apex or corner and proper lateral reinforcement provided. For example, tied triangular columns require three longitudinal bars, one at each apex of the triangular ties. For bars enclosed by spirals, six bars are required.

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 spa11 off. The amount of spiral reinforcement required by Eq. (10-5) 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 Reference 10.24 and is adopted in this SBC 304. The derivation of Eq. (10-5) is given in Reference 10.24. Tests and experience show that columns containing the amount of spiral reinforcement required by this section exhibit considerable toughness and ductility.

SECTION R10.10
SLENDERNESS EFFECTS IN COMPRESSION MEMBERS

Provisions for slenderness effects in compression members and frames are discussed in Reference 10.25 to better recognize the use of second-order analyses and to improve the arrangement of the provisions dealing with sway (unbraced) and nonsway (braced) frames. The use of a refined nonlinear second-order analysis is permitted in 10.10.1. Sections 10.11, 10.12, and 10.13 present an approximate design method based on the moment magnifier method. For sway frames, the magnified sway moment $\delta_s M_s$ may be calculated using a second-order elastic analysis, by an approximation to such an analysis, or by the traditional sway moment magnifier.

R10.10.1 Two limits are placed on the use of the refined second-order analysis. First, 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. Second, the refined second-order analysis procedure should have been shown to predict ultimate loads within 15 percent of those reported in tests of indeterminate reinforced concrete structures. At the very least, the comparison should include tests of columns in planar nonsway frames, sway frames, and frames with varying column stiffnesses. To allow for variability in the actual member properties and in the analysis, the member properties used in analysis should be multiplied by a stiffness reduction factor ϕ_k less than one. For consistency with the second-order analysis in 10.13.4.1, the stiffness reduction factor ϕ_k can be taken as 0.80. The concept of a stiffness reduction factor ϕ_k is discussed in R10.12.3.

R10.10.2 As an alternate to the refined second-order analysis of 10.10.1, design may be based on elastic analyses and the moment magnifier approach.[10.26,10.27] For sway frames the magnified sway moments may be calculated using a second-order elastic analysis based on realistic stiffness values. See R10.13.4.1.

SECTION R10.11
MAGNIFIED MOMENTS - GENERAL

This section describes an approximate design procedure that uses the moment magnifier concept to account for slenderness effects. Moments computed using an ordinary first-order frame analysis are multiplied by a moment magnifier that is a function of the factored axial load P_u and the critical buckling load P_c for the column. Nonsway and sway frames are treated separately in 10.12 and 10.13. Provisions applicable to both nonsway and sway columns are given in 10.11. A first-order frame analysis is an elastic analysis that does not include the internal force effects resulting from deflections.

R10.11.1 The stiffnesses EI used in an elastic analysis used for strength design should represent the stiffnesses of the members immediately prior to failure. This is particularly true for a second-order analysis that should predict the lateral deflections at loads approaching ultimate. The EI 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.

The alternative values of E_c, I_g, and A_g given in 10.11.1 have been chosen from the results of frame tests and analyses and include an allowance for the variability of the computed deflections. The modulus of elasticity E_c is based on the specified concrete strength while the sway deflections are a function of the average concrete strength, which is higher. The moments of inertia were taken as 0.875 times those in Reference 10.28. These two effects result in an overestimation of the second-order deflections in the order of 20 to 25 percent, corresponding to an implicit stiffness reduction factor ϕ_k of 0.80 to 0.85 on the stability calculation. The concept of a stiffness reduction factor ϕ_k is discussed in R10.12.3

The moment of inertia of T-beams should be based on the effective flange width defined in 8.10. It is generally sufficiently accurate to take I_g of a T-beam as two times the I_g for the web, $2(b_w h^3/12)$.

If the factored moments and shears from an analysis based on the moment of inertia of a wall taken equal to $0.70I_g$ indicate that the wall will crack in flexure, based on the modulus of rupture, the analysis should be repeated with $I = 0.35I_g$ in those stories where cracking is predicted at factored loads.

The alternative values of the moments of inertia given in 10.11.1 were derived for nonprestressed members. For prestressed members, the moments of inertia may differ from the values in 10.11.1 depending on the amount, location, and type of the reinforcement and the degree of cracking prior to ultimate. The stiffness values for prestressed concrete members should include an allowance for the variability of the stiffnesses.

Sections 10.11 through 10.13 provide requirements for strength and assume frame analyses will be carried out using factored loads. Analyses of deflections, vibrations, and building periods are needed at various service (unfactored) load levels[10.29,10.30] to determine the serviceability of the structure and to estimate the wind forces in wind tunnel laboratories. The seismic base shear is also based on the service load periods of vibration. The magnified service loads and deflections by a second-order analysis should also be computed using service loads. The moments of inertia of the structural members in the service load analyses should, therefore, be representative of the degree of cracking at the various service load levels investigated. Unless a more accurate estimate of the degree of cracking at design service load level is available, it is satisfactory to use *1/0.70 = 1.43* times the moments of inertia given in 10.11.1 for service load analyses.

Item (d) in 10.11.1 refers to the unusual case of sustained lateral loads. Such a case might exist, for example, if there were permanent lateral loads resulting from unequal earth pressures on two sides of a building.

R10.11.4 The moment magnifier design method requires the designer to distinguish between nonsway frames, which are designed according to 10.12, and sway frames, which are designed according to 10.13. Frequently this can be done by inspection by comparing the total lateral stiffness of the columns in a story to that of the bracing elements. A compression member may be assumed nonsway by inspection if it is located in a story in which the bracing elements (shearwalls, shear trusses, or other types of lateral bracing) have such substantial lateral stiffness to resist the lateral deflections of the story that any resulting lateral deflection is not large enough to affect the column strength substantially. If not readily apparent by inspection, 10.11.4.1 and 10.11.4.2 give two possible ways of doing this. In 10.11.4.1, a story in a frame is said to be nonsway if the increase in the lateral load moments resulting from $P\Delta$ effects does not exceed 5 percent of the first-order moments.[10.28] Section 10.11.4.2 gives an alternative method of determining this based on the stability index for a story Q. In computing Q, $\sum P_u$ should correspond to the lateral loading case for which $\sum P_u$ is greatest. A frame may contain both nonsway and sway stories. This test would not be suitable if V_u 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 10.11.1, it is permissible to compute Q in Eq. (10-6) using 1.2 times the sum of the service gravity loads, the service load story shear, and 1.43 times the first-order service load story deflections.

R10.11.5 An upper limit is imposed on the slenderness ratio of columns designed by the moment magnifier method of 10.11 to 10.13. No similar limit is imposed if design is carried out according to 10.10.1. The limit of $k\ell_u / r = 100$ represents the upper range of actual tests of slender compression members in frames.

R10.11.6 When biaxial bending occurs in a compression member, the computed moments about each principal axes should be magnified. The magnification factors δ are computed considering the buckling load P_c about each axis separately based on the appropriate effective length kl_u and the stiffness EI. If the buckling capacities are different about the two axes, different magnification factors will result.

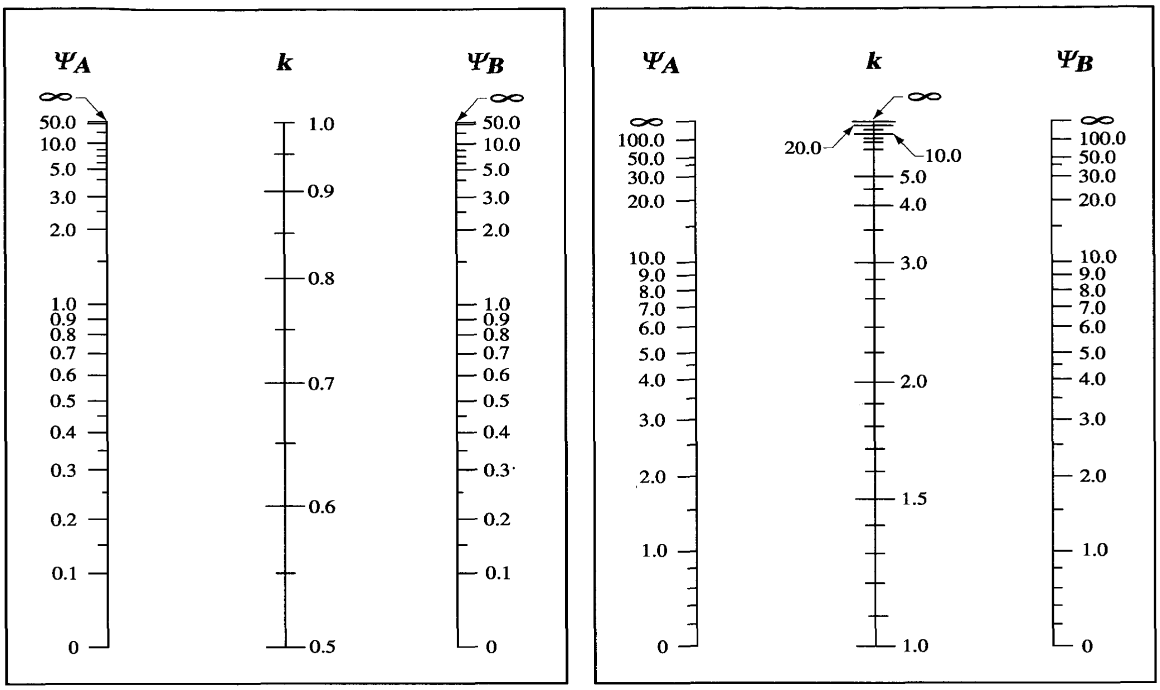
SECTION R10.12
MAGNIFIED MOMENTS – NONSWAY FRAMES

R10.12.1 The moment magnifier equations were derived for hinged end columns and should be modified to account for the effect of end restraints. This is done by using an effective length $k\ell_u$ in the computation of P_c.

The primary design aid to estimate the effective length factor k is the Jackson and Moreland Alignment Charts (Fig. R10.12.1), which allow a graphical determination of k for a column of constant cross section in a multibay frame.[10.31,10.32]

The effective length is a function of the relative stiffness at each end of the compression member. Studies have indicated that the effects of varying beam and column reinforcement percentages and beam cracking should be considered in determining the relative end stiffnesses. In determining ψ for use in evaluating the effective length factor k, the rigidity of the flexural members may be calculated on the basis of $0.35I_g$ for flexural members to account for the effect of cracking and reinforcement on relative stiffness, and $0.70I_g$ for compression members.

The simplified equations (A-E), listed below for computing the effective length factors for nonsway and sway members, may be used. Eq. (A), (B), and (E) are taken from Reference 10.33 and 10.34. Eq. (C) and (D) for sway members were developed in Reference 10.32.



(a) Nonsway Frames (b) Sway Frames

Ψ = ratio of Σ(EI/ℓc) of compression members to Σ(EI/ℓ) of flexural members in a plane at one end of a compression member

ℓ = span length of flexural member measured center to center of joints

Fig. R10.12.1 - Effective length factors, k.

For compression members in a nonsway frame, an upper bound to the effective length factor may be taken as the smaller of the following two expressions:

$$k = 0.7 + 0.05(\psi_A + \psi_B) \leq 1.0 \qquad \text{(A)}$$

$$k = 0.85 + 0.05\psi_{\min} \leq 1.0 \qquad \text{(B)}$$

where ψ_A and ψ_B are the values of ψ at the two ends of the column and $\psi_{\min}$ is the smaller of the two values.

For compression members in a sway frame, restrained at both ends, the effective length factor may be taken as:

For $\Psi_m < 2$

$$k = \frac{20 - \Psi_m}{20}\sqrt{1 + \Psi_m} \qquad \text{(C)}$$

For $\Psi_m \geq 2$

$$k = 0.9\sqrt{1 + \Psi_m} \qquad \text{(D)}$$

where ψ_m is the average of the ψ-values at the two ends of the compression member.

For compression members in a sway frame, hinged at one end, the effective length factor may be taken as:

$$k = 2.0 + 0.3\psi \qquad \text{(E)}$$

Where ψ is the value at the restrained end.

The use of the charts in Fig. R10.12.1, or the equations in this section, may be considered as satisfying the requirements of the SBC 304 to justify k less than 1.0.

R10.12.2 Eq. (10-7) is derived from Eq. (10-9) assuming that a 5 percent increase in moments due to slenderness is acceptable.[10.26] The derivation did not include ϕ in the calculation of the moment magnifier. As a first approximation, k may be taken equal to 1.0 in Eq. (10-7).

R10.12.3 Studies reported in Reference 10.35 indicate that the stiffness reduction factor ϕ_k, and the cross-sectional strength reduction ϕ-factors 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 factors in Eq. (10-9) and (10-18) are stiffness reduction factors. This is done to avoid confusion between a stiffness reduction factor ϕ_k in Eq. (10-9) and (10-18), and the cross-sectional strength reduction ϕ-factors.

In defining the critical load, the main problem is the choice of a stiffness EI that reasonably approximates the variations in stiffness due to cracking, creep, and the nonlinearity of the the concrete stress-strain curve. Eq. (10-11) was derived for small eccentricity ratios and high levels of axial load where the slenderness effects are most pronounced.

Creep due to sustained load will increase the lateral deflections of a column and hence the moment magnification. This is approximated for design by reducing the stiffness EI used to compute P_c and hence δ_{ns} by dividing EI by $(1 + \beta_d)$. Both the concrete and steel terms in Eq. (10-11) are divided by $(1+\beta_d)$. This reflects the premature yielding of steel in columns subjected to sustained load.

Either Eq. (10-11) or (10-12) may be used to compute EI. Eq. (10-12) is a simplified approximation to Eq. (10-11). It is less accurate than Eq. (10-11).[10.36] Eq. (10-12) may be simplified further by assuming β_d = 0.6. When this is done Eq. (10-12) becomes

$$EI = 0.25 E_c I_g \tag{F}$$

The term β_d is defined differently for nonsway and sway frames. See 10.0. For nonsway frames, β_d is the ratio of the maximum factored axial sustained load to the maximum factored axial load.

R10.12.3.1 The factor C_m is an equivalent moment correction factor. The derivation of the moment magnifier assumes that the maximum moment is at or near midheight of the column. If the maximum moment occurs at one end of the column, design should be based on an equivalent uniform moment C_m / M_2 that would lead to the same maximum moment when magnified.[10.26]

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. (10-8). In accordance with the last sentence of 10.12.3.1, C_m is to be taken as 1.0 for this case.

R10.12.3.2 In the SBC 304, 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 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. (10-13) 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.

SECTION R10.13
MAGNIFIED MOMENTS – SWAY FRAMES

The design procedure of sway frames for slenderness consists of three steps:

(1) The magnified sway moments $\delta_s M_s$ are computed. This should be done in one of three ways. First, a second-order elastic frame analysis may be used (10.13.4.1).

Second, an approximation to such analysis (10.13.4.2) may be used. The third option is to use the sway magnifier δ_s from previous editions of the SBC 304 (10.13.4.3);

(2) The magnified sway moments $\delta_s M_s$ are added to the unmagnified nonsway moment M_{ns} at each end of each column (10.13.3). The nonsway moments may be computed using a first-order elastic analysis;

(3) If the column is slender and heavily loaded, it is checked to see whether the moments at points between the ends of the column exceed those at the ends of the column. As specified in 10.13.5 this is done using the nonsway frame magnifier δ_{ns} with P_c computed assuming $k = 1.0$ or less.

R10.13.1 See R10.12.1.

R10.13.3 The analysis described in this section deals only with plane frames subjected to loads causing deflections in that plane. If torsional displacements are significant, a three-dimensional second-order analysis should be used.

R10.13.4 **Calculation of $\delta_s M_s$**

R10.13.4.1 A second-order analysis is a frame analysis that includes the internal force effects resulting from deflections. When a second-order elastic analysis is used to compute $\delta_s M_s$, the deflections should be representative of the stage immediately prior to the ultimate load. For this reason the reduced $E_c I_g$ values given in 10.11.1 should be used in the second-order analysis.

The term β_d is defined differently for nonsway and sway frames. See 10.0. Sway deflections due to short-term loads such as wind or earthquake are a function of the short-term stiffness of the columns following a period of sustained gravity load. For this case the definition of β_d in 10.0 gives β_d = 0. In the unusual case of a sway frame where the lateral loads are sustained, β_d will not be zero. This might occur if a building on a sloping site is subjected to earth pressure on one side but not on the other.

In a second-order analysis the axial loads in all columns that are not part of the lateral load resisting elements and depend on these elements for stability should be included.

The second-order analysis method is based on the values of E and I from 10.11.1. These lead to a 20 to 25 percent overestimation of the lateral deflections that corresponds to a stiffness reduction factor ϕ_k between 0.80 and 0.85 on the $P\Delta$ moments. No additional ϕ-factor is needed in the stability calculation. Once the moments are established, selection of the cross sections of the columns involves the strength reduction factors ϕ from 9.3.2.2.

R10.13.4.2 The iterative $P\Delta$ analysis for second-order moments can be represented by an infinite series. The solution of this series is given by Eq. (10-17).[10.28] Reference 10.37 shows that Eq. (10-17) closely predicts the second order moments in a sway frame until δ_s exceeds 1.5.

The $P\Delta$ moment diagrams for deflected columns are curved, with Δ related to the deflected shape of the columns. Eq. (10-17) and most commercially available second-order frame analyses have been derived assuming that the $P\Delta$ moments result from equal and opposite forces of $P\Delta / \ell_C$ applied at the bottom and top of the story. These forces give a straight line $P\Delta$ moment diagram. The curved $P\Delta$

moment diagrams lead to lateral displacements in the order of 15 percent larger than those from the straight line $P\Delta$ moment diagrams. This effect can be included in Eq. (10-17) by writing the denominator as $(1-1.15Q)$ rather than $(1-Q)$. The 1.15 factor has been left out of Eq. (10-17) to maintain consistency with available computer programs.

If deflections have been calculated using service loads, Q in Eq. (10-17) should be calculated in the manner explained in R10.11.4.

The Q factor analysis is based on deflections calculated using the values of E_c and I_g from 10.11.1, which include the equivalent of a stiffness reduction factor ϕ_k as explained in R10.13.4.1. As a result, no additional ϕ-factor is needed in the stability calculation. Once the moments are established using Eq. (10-17), selection of the cross sections of the columns involves the strength reduction factors ϕ from 9.3.2.2.

R10.13.4.3 To check the effects of story stability, δ_s is computed as an averaged value for the entire story based on use of $\sum P_u / \sum P_c$. This reflects the interaction of all sway resisting columns in the story in the $P\Delta$ effects since 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 will have ℓ_u / r greater than the value given in Eq. (10-19) and should be checked using 10.13.5.

If the lateral load deflections involve a significant torsional displacement, the moment magnification in the columns farthest from the center of twist may be underestimated by the moment magnifier procedure. In such cases, a three-dimensional second-order analysis should be considered.

The 0.75 in the denominator of Eq. (10-18) is a stiffness reduction factor ϕ_k as explained in R10.12.3.

In the calculation of EI, β_d will normally be zero for a sway frame because the lateral loads are generally of short duration. (See R10.13.4.1).

R10.13.5 The unmagnified nonsway moments at the ends of the columns are added to the magnified sway moments at the same points. Generally, one of the resulting end moments is the maximum moment in the column. However, for slender columns with high axial loads the point of maximum moment may be between the ends of the column so that the end moments are no longer the maximum moments. If ℓ_u / r is less than the value given by Eq. (10-19) the maximum moment at any point along the height of such a column will be less than 1.05 times the maximum end moment. When ℓ_u / r exceeds the value given by Eq. (10-19), the maximum moment will occur at a point between the ends of the column and will exceed the maximum end moment by more than 5 percent.[10.25] In such a case the maximum moment is calculated by magnifying the end moments using Eq. (10-8).

R10.13.6 The possibility of sidesway instability under gravity loads alone should be investigated. When using second-order analyses to compile $\delta_s M_s$ (10.13.4.1), the frame should be analyzed twice for the case of factored gravity loads plus a lateral load applied to the frame. This load may be the lateral load used in design or it may be a single lateral load applied to the top of the frame. The first analysis should be a first-order analysis, the second analysis should be a second-order analysis. The deflection from the second-order analysis should not exceed 2.5 times the deflection from the first-order analysis. If one story is much more flexible than the others, the deflection ratio should be computed in that story. The lateral load should be large enough to give deflections of a magnitude that can be compared accurately. In unsymmetrical frames that deflect laterally under gravity loads alone, the lateral load should act in the direction for which it will increase the lateral deflections.

When using 10.13.4.2 to compute $\delta_s M_s$ the value of Q evaluated using factored gravity loads should not exceed 0.60. This is equivalent to δ_s = 2.5. The values of V_u and Δ_o used to compute Q can result from assuming any real or arbitrary set of lateral loads provided that V_u and Δ_o are both from the same loading. If Q as computed in 10.11.4.2 is 0.2 or less, the stability check in 10.13.6 is satisfied.

When $\delta_s M_s$ is computed using Eq. (10-18), an upper limit of 2.5 is placed on δ_s. For higher δ_s values, the frame will be very susceptible to variations in EI and foundation rotations. If δ_s exceeds 2.5 the frame should be stiffened to reduce $\delta_s . \sum P_u$ shall include the axial load in all columns and walls including columns that are not part of the lateral load resisting system. The value δ_s = 2.5 is a very high magnifier. It has been chosen to offset the conservatism inherent in the moment magnifier procedure.

For nonsway frames, β_d is the ratio of the maximum factored axial sustained load to the maximum factored axial load.

R10.13.7 The strength of a sway frame is governed by the stability of the columns and by the degree of end restraint provided by the beams in the frame. If plastic hinges form in the restraining beam, the structure approaches a failure mechanism and its axial load capacity is drastically reduced. Section 10.13.7 provides that the designer make certain that the restraining flexural members have the capacity to resist the magnified column moments.

SECTION R10.15
TRANSMISSION OF COLUMN LOADS THROUGH FLOOR SYSTEM

The requirements of this section are based on a paper on the effect of floor concrete strength on column strength.[10.38] The provisions mean that when the column concrete strength does not exceed the floor concrete strength by more than 40 percent, no special precautions need be taken. For higher column concrete strengths, methods in 10.15.1 or 10.15.2 should be used for corner or edge columns. Methods in 10.15.1, 10.15.2, or 10.15.3 should be used for interior columns with adequate restraint on all four sides.

R10.15.1 Application of the concrete placement procedure described in 10.15.1 requires the placing of two different concrete mixtures in the floor system. The lower strength mixture should be placed while the higher strength concrete is still plastic and should be adequately vibrated to ensure the concretes are well integrated. This requires careful coordination of the concrete deliveries and the possible use of retarders. In some cases, additional inspection services will be required when this procedure is used. It is important that the higher strength concrete in the floor in the region of the column be placed before the lower strength concrete in the remainder of the floor to prevent accidental placing of the low strength concrete in the column area. It is the designer's responsibility to indicate on the drawings where the high and low strength concretes are to be placed.

Since 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. The amount of column concrete to be placed within the floor is expressed as a simple 600-mm extension from face of the column, to be directly evident to workers.

R10.15.3 Research[10.39] 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.

SECTION R10.16
COMPOSITE COMPRESSION MEMBERS

R10.16.1 Composite columns are defined without reference to classifications of combination, composite or concrete-filled pipe column. Reference to other metals used for reinforcement has been omitted because they are seldom used in concrete construction.

R10.16.2 The same rules used for computing the load-moment interaction strength for reinforced concrete sections can be applied to composite sections. Interaction charts for concrete-filled tubing would have a form identical to those of Reference 10.40 and Reference 10.32 but with γ slightly greater than 1.0.

R10.16.3 and
R10.16.4 Direct bearing or direct connection for transfer of forces between steel and concrete can be developed through lugs, plates, or reinforcing bars welded to the structural shape or tubing before the concrete is cast. Flexural compressive stress need not be considered a part of direct compression load to be developed by bearing. A concrete encasement around a structural steel shape may stiffen the shape, but it would not necessarily increase its strength.

R10.16.5 Eq. (10-20) is given because the rules of 10.11.2 for estimating the radius of gyration are overly conservative for concrete filled tubing and are not applicable for members with enclosed structural shapes.

In reinforced concrete columns subject to sustained loads, creep transfers some of the load from the concrete to the steel, increasing the steel stresses. In the case of lightly reinforced columns, this load transfer may cause the compression steel to yield prematurely, resulting in a loss in the effective EI. Accordingly, both the concrete and steel terms in Eq. (10-11) 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. (10-21) will be used so that only the EI of the concrete is reduced for sustained load effects.

R10.16.6 **Structural steel encased concrete core**. Steel encased concrete sections should have a metal wall thickness large enough to attain longitudinal yield stress before buckling outward.

R10.16.7 **Spiral reinforcement around structural steel core.** Concrete that is laterally confined by a spiral has increased load-carrying strength, and the size of the spiral required can be regulated on the basis of the strength of the concrete outside the spiral the same reasoning that applies for columns reinforced only with longitudinal bars. The radial pressure provided by the spiral ensures interaction between concrete, reinforcing bars, and steel core such that longitudinal bars will both stiffen and strengthen the cross section.

R10.16.8 **Tie reinforcement around structural steel core.** Concrete that is laterally confined by tie bars is likely to be rather thin along at least one face of a steel core section. Therefore, complete interaction between the core, the concrete, and any longitudinal reinforcement should not be assumed. Concrete will probably separate from smooth faces of the steel core. To maintain the concrete around the structural steel core, it is reasonable to require more lateral ties than needed for ordinary reinforced concrete columns. Because of probable separation at high strains between the steel core and the concrete, longitudinal bars will be ineffective in stiffening cross sections even though they would be useful in sustaining compression forces. The yield strength of the steel core should be limited to that which exists at strains below those that can be sustained without spalling of the concrete. It has been assumed that axially compressed concrete will not spall at strains less than 0.0018. The yield strength of 0.0018 x 200,000, or 360 MPa, represents an upper limit of the useful maximum steel stress.

SECTION R10.17
BEARING STRENGTH

R10.17.1 This section deals with bearing strength of concrete supports. The permissible bearing stress of $0.85 f_c^{'}$ is based on tests reported in Reference 10.41. (See also 15.8).

When the supporting area is wider than the loaded area on all sides, the surrounding concrete confines the bearing area, resulting in an increase in bearing strength. No minimum depth is given for a supporting member. The minimum depth of support will be controlled by the shear requirements of 11.12.

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. Fig. R10.17 illustrates the application of the frustum to find A_2. The frustum should not be confused with the path by which a load spreads out as it travels downward through the support.

Such a load path would have steeper sides. However, the frustum described has somewhat flat side slopes to ensure that there is concrete immediately surrounding the zone of high stress at the bearing. A_1 is the loaded area but not greater than the bearing plate or bearing cross-sectional area.

R10.17.2 Post-tensioning anchorages are usually laterally reinforced, in accordance with 18.13.

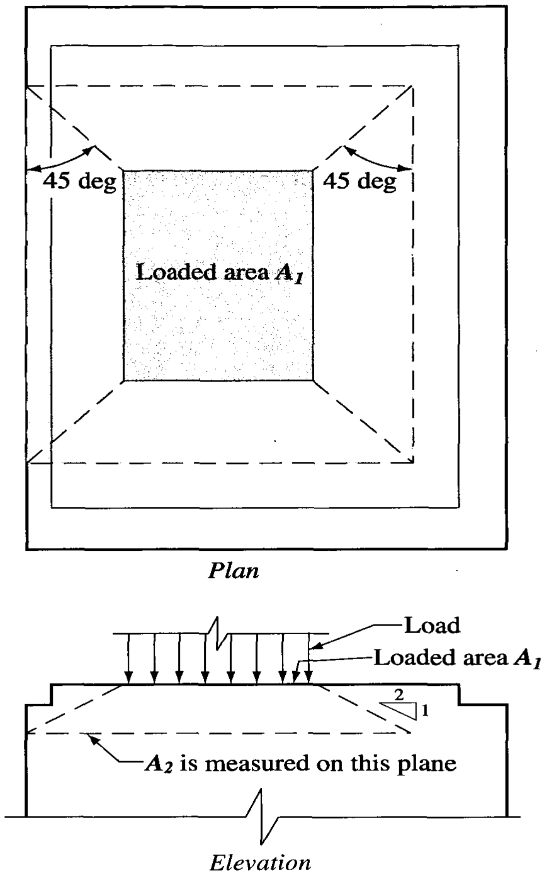


Fig. R10.17 - Application of frustum to find A_2 in stepped or sloped supports

CHAPTER 11
SHEAR AND TORSION

This chapter includes shear and torsion provisions for both nonprestressed and prestressed concrete members. The shear-friction concept (11.7) is particularly applicable to design of reinforcement details in precast structures. Special provisions are included for deep flexural members (11.8), brackets and corbels (11.9), and shear walls (11.10). Shear provisions for slabs and footings are given in 11.12.

SECTION R11.0
NOTATION

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

Tests[11.1] have indicated that the average shear stress over the full effective section also may be applicable for circular sections. Note the special definition of d for such sections.

Although the value of d may vary along the span of a prestressed beam, studies[11.2] showed that, for prestressed concrete members, d need not be taken less than $0.80h$. The beams considered had some straight tendons or reinforcing bars at the bottom of the section and had stirrups that enclosed that steel.

SECTION R11.1
SHEAR STRENGTH

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 strength 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 References 11.1, 11.2, and 11.3.

Appendix A 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 Reference 11.1 and in References 11.4 and 11.5.

R11.1.1.2 In a member of variable depth, the internal shear at any section is increased or decreased by the vertical component of the inclined flexural stresses. Computation methods are outlined in various textbooks and in Reference 11.6

R11.1.2 Because of a lack of test data and practical experience with concretes having compressive strengths greater than 70 MPa, a maximum value of 25/3 MPa is imposed 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 70 MPa, it is prudent to limit $\sqrt{f_c'}$ to 25/3 MPa for the calculation of shear strength.

R11.1.2.1 Based on the test results in References 11.7, 11.8, 11.9, 11.10, and 11.11, an increase in the minimum amount of transverse reinforcement is required for high-strength concrete. These tests indicated a reduction in the reserve shear strength as f_c' increased in beams reinforced with the specified minimum amount of transverse reinforcement, which is equivalent to an effective shear stress of 0.34 MPa.

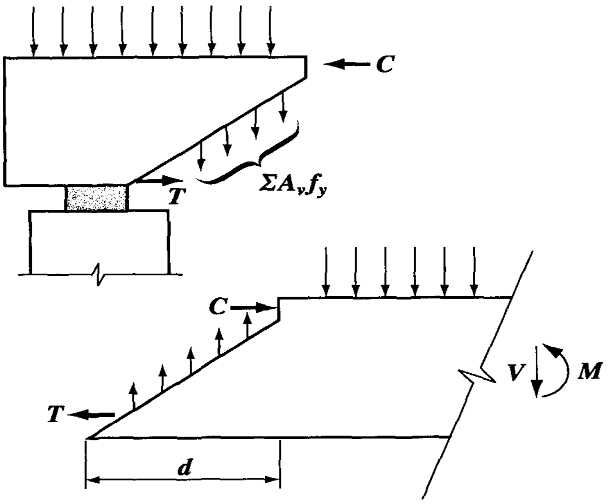


Fig. R11.1.3.1(a) - Free body diagrams of the end of a beam

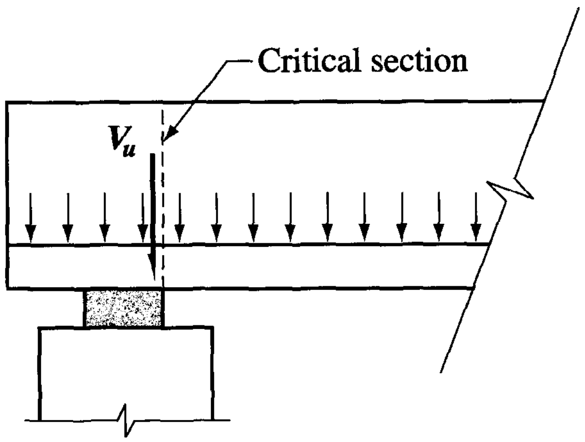


Fig. R11.1.3.1(b) - Location of critical section for shear in a member loaded near bottom

R11.1.3.1 The closest inclined crack to the support of the beam in Fig. R11.1.3.1(a) will extend upwards from the face of the support reaching the compression zone about d from the face of the support. If loads are applied to the top of this beam, the stirrups across this crack are stressed by loads acting on the lower freebody in Fig. R11.1.3.1(a). The loads applied to the beam between the face of the column and the point d away from the face are transferred directly to the support by compression in the web above the crack. Accordingly, the SBC 304 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.

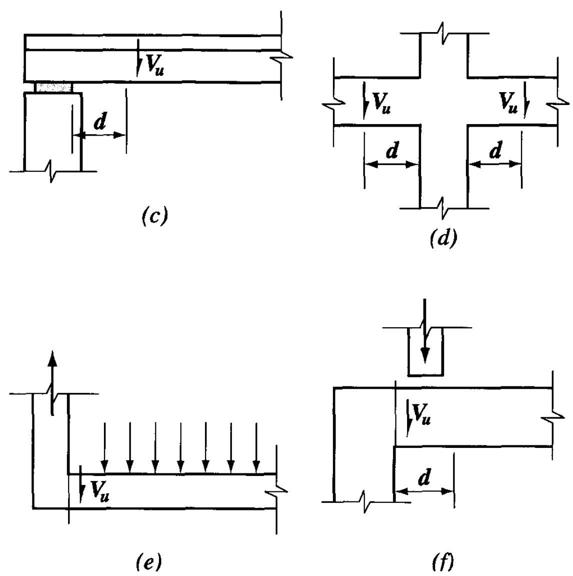


Fig. R11.1.3.1(c), (d), (e), (f) - Typical support conditions for locating factored shear force V_u

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. 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 d from the support differs radically from the shear at distance d. This commonly occurs in brackets and in beams where a concentrated load is located close to the support, as shown in Fig. R11.1.3.1(f) or in footings supported on piles. In this case the shear at the face of' the support should be used.

R11.1.3.2 Because d frequently varies in prestressed members, the location of the critical section has arbitrarily been taken as $h/2$ from the face of the support.

SECTION R11.2
LIGHTWEIGHT CONCRETE

Two alternative procedures are provided to modify the provi sions for shear and torsion when lightweight aggregate concrete is used. The lightweight concrete modification applies only to the terms containing $\sqrt{f_c^{'}}$ in the equations of Chapter 11 SBC 304.

R11.2.1.1 The first alternative bases the modification on laboratory tests to determine the relationship between splitting tensile strength f_{ct} and the compressive strength $f_c^{'}$ for the lightweight concrete being used. For normal weight concrete, the splitting tensile strength f_{ct} is approximately equal to, $\sqrt{f_c^{'}}$ /1.8.[11.10,11.11]

R11.2.1.2 The second alternative bases the modification on the assumption that the tensile strength of lightweight concrete is a fixed fraction of the tensile strength of normalweight concrete[11.12]. The multipliers are based on data from tests[11.13] on many types of structural lightweight aggregate concrete.

SECTION R11.3
SHEAR STRENGTH PROVIDED BY CONCRETE FOR NONPRESTRESSED MEMBERS

R11.3.1.1 See R11.3.2.1
R11.3.1.2 and
R11.3.1.3 See R11.3.2.2.

R11.3.2.1 Eq. (11-5) is the basic expression for shear strength of members without shear reinforcement.[11.3] Designers should recognize that the three variables in Eq. (11-5), $\sqrt{f_c^{'}}$ (as a measure of concrete tensile strength), ρ_w and $V_u d/M_u$, are known to affect shear strength, although some research data [11.1.1,1.14] indicate that Eq. (11-5) overestimates the influence of $f_c^{'}$ and underestimates the influence of ρ_w and $V_u d/M_u$. Further information[11.15] has indicated that shear strength decreases as the overall depth of the member increases.

The minimum value of M_u equal to $V_u d$ in Eq. (11-5) is to limit V_c near points of inflection.

For most designs, it is convenient to assume that the second term of Eq. (11-5) equals $0.17\sqrt{f_c^{'}}$ and use V_c equal to $(1/6)\sqrt{f_c^{'}}b_w d$ as permitted in 11.3.1.1.

R11.3.2.2 Eq. (11-6) and (11-7), for members subject to axial compression in addition to shear and flexure are derived in Reference 11.3. As N_u is increased, the value of

V_c computed from Eq. (11-5) and (11-6) will exceed the upper limit given by Eq. (11-7) before the value of M_m given by Eq. (11-6) becomes negative. The value of V_c obtained from Eq. (11-5) has no physical significance if a negative value of M_m is substituted. For this condition, 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. R11.3.2.2. The background for these equations is discussed and comparisons are made with test data in Reference 11.2.

Because of the complexity of Eq. (11-5) and (11-6), an alternative design provision, Eq. (11-4), is permitted.

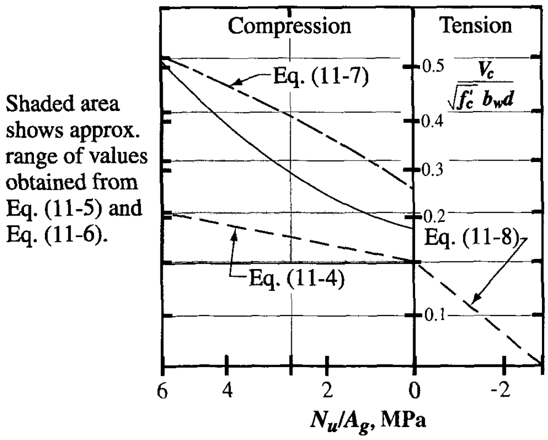


Fig. R11.3.2.2 - Comparison of shear strength equations for members subject to axial load

R11.3.2.3 Eq. (11-8) may be used to compute V_c for members subject to significant axial tension. Shear reinforcement may then be designed for $V_n - V_c$. The term significant is used to recognize that a designer must use judgment in deciding whether axial tension needs to be considered. Low levels of axial tension often occur due to volume changes, but are not important in structures with adequate expansion joints and minimum reinforcement. It may be desirable to design shear reinforcement to carry total shear if there is uncertainty about the magnitude of axial tension.

R11.3.3 Shear tests of members with circular sections indicate that the effective area can be taken as the gross area of the section or as an equivalent rectangular area.[11.1,11.16,11.17]

SECTION 11.4
SHEAR STRENGTH PROVIDED BY CONCRETE FOR PRESTRESSED MEMBERS

R11.4.1 Eq. (11-9) offers a simple means of computing V_c for prestressed concrete beams.[11.2] It may be applied to beams having prestressed reinforcement only, or to members reinforced with a combination of prestressed reinforcement and nonprestressed deformed bars. 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 $V_u d / M_u$ can be expressed as

$$\frac{V_u d}{M_u} = \frac{d(\ell - 2x)}{x(\ell - x)}$$

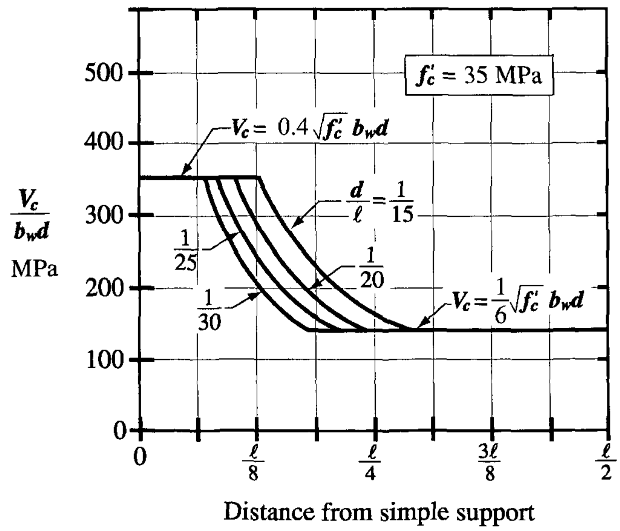


Fig. R11.4.1 - Application of Eq. (11-9) to uniformly loaded prestressed members

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 35 MPa, V_c from 11.4.1 varies as shown in Fig. R11.4.1. Design aids based on this equation are given in Reference 11.18.

R11.4.2 Two types of inclined cracking occur in concrete beams: web-shear cracking and flexure-shear cracking. These two types of inclined cracking are illustrated in Fig. R11.4.2.

Web-shear cracking begins from an interior point in a member when the principal tensile stresses exceed the tensile strength of the concrete. Flexure-shear cracking is initiated by flexural cracking. When flexural cracking occurs, the shear stresses in the concrete above the crack are increased. The flexure-shear crack develops when the combined shear and tensile stress exceeds the tensile strength of the concrete.

Eqn. (11-11) Eq. (11-10) and (11-12) may be used to determine the shear forces causing flexure-shear and web-shear cracking, respectively. The shear strength provided by the concrete V_c is assumed equal to the lesser of V_{ci} and V_{cw}. The derivations of Eq. (11-10) and (11-12) are summarized in Reference 11.19.

In deriving Eq. (11-10) it was assumed that V_{ci} is the sum of the shear required to cause a flexural crack at the point in question given by:

$$V = \frac{V_i M_{cr}}{M_{max}}$$

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_{cr} for substitution into Eq. (11-10), I and y_t, are the properties of the section resisting the externally applied loads.

For a composite member, where part of the dead load is resisted by only a part of the section, appropriate section properties should be used to compute f_d. The shear due to dead loads, V_d and that due to other loads V_i are separated in this case.

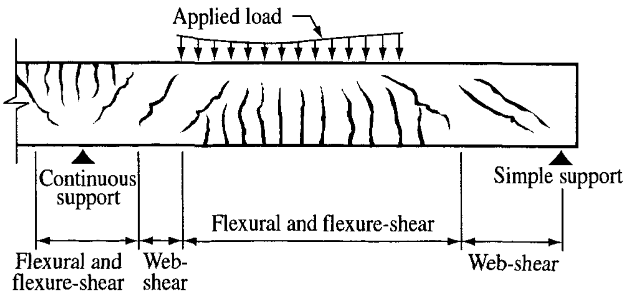


Fig. R11.4.2 - Types of cracking in concrete beams

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 (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:

$$V_{ci} = \frac{\sqrt{f_c'}}{20} b_w d + \frac{V_u M_{ct}}{M_u}$$

where:

$$M_{ct} = (I / y_t)\left(\sqrt{f_c'} + f_{pe}\right)$$

The symbol M_{ct} in the two preceding equations represents the total moment, including dead load, required to cause cracking at the extreme fiber in tension. This is not the same as M_{cr}, in SBC 304 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. Since 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 $\sqrt{f_c'}/2$, assumed to cause flexural cracking.

Eq. (11-12) is based on the assumption that web-shear cracking occurs due to the shear causing a principal tensile stress of approximately $(1/3)\sqrt{f_c'}$ at the centroidal axis of the cross section. V_p is calculated from the effective prestress force without load factors.

R11.4.3 and
R11.4.4 The effect of the reduced prestress near the ends of pretensioned beams on the shear strength should be taken into account. Section 11.4.3 relates to the shear strength at sections within the transfer length of prestressing steel when bonding of prestressing steel extends to the end of the member.

Section 11.4.4 relates to the shear strength at sections.within the length over which some of the prestressing steel is not bonded to the concrete, or within the transfer length of the prestressing steel for which bonding does not extend to the end of the beam.

SECTION R11.5
SHEAR STRENGTH PROVIDED BY SHEAR REINFORCEMENT

R11.5.2 Limiting the design yield strength of shear reinforcement to 420 MPa provides a control on diagonal crack width. Research[11.20,11.21,11.22] has indicated that the performance of higher strength steels as shear reinforcement has been satisfactory. In particular**,** full-scale beam tests described in Reference 11.21 indicated that the widths of inclined shear cracks at service load levels were less for beams reinforced with smaller diameter deformed welded wire fabric cages designed on the basis of a yield strength of 520 MPa than beams reinforced with deformed Grade 420 stirrups.

R11.5.3 It is essential that shear (and torsion) reinforcement be adequately anchored at both ends to be fully effective on either side of any potential inclined crack. This generally requires a hook or bend at the end of the reinforcement as provided by 12.13.

R11.5.5 **Minimum shear reinforcement**

R11.5.5.1 Shear reinforcement restrains the growth of inclined cracking. Ductility is increased and a warning of failure is provided. In an unreinforced web, the sudden formation of inclined cracking might lead directly to failure without warning. Such reinforcement is of great value if a member is subjected to an unexpected tensile force or an overload. Accordingly, a minimum area of shear reinforcement not less than that given by Eq. (11-13) or (11-14) is required wherever the total factored shear force V_u is greater than one-half the shear strength provided by concrete ϕV_c Slabs, footings and joists are excluded from the minimum shear reinforcement requirement because there is a possibility of load sharing between weak and strong areas. However, research results[11.23] have shown that deep, lightly reinforced one-way slabs, particularly if constructed with high-strength concrete, may fail at shear loads less than V_u, calculated from Eq. (11-3).

Even when the total factored shear strength V_u is less than one-half of the shear strength provided by the concrete ϕV_c the use of some web reinforcement is recommended in all: thin-web post-tensioned prestressed concrete members (joists, waffle slabs, beams, and T-beams) to reinforce against tensile forces in webs resulting from local deviations from the design tendon profile, and to provide a means of supporting the tendons in the design profile during construction. If sufficient support is not provided, lateral wobble and local deviations from the smooth parabolic tendon profile assumed in design may result during placement of the concrete. In such cases, the deviations in the tendons tend to straighten out when the tendons are stressed. This process may impose large tensile stresses in webs, and severe cracking may develop if no web reinforcement is provided. Unintended curvature of the tendons, and the resulting tensile stresses in webs, may be minimized by securely tying tendons to stirrups that are rigidly held in place by other elements of the reinforcing cage and held down in the forms. The maximum spacing of stirrups used for this purpose should not exceed the smaller of $1.5h$ or 1.2 m. When applicable, the shear reinforcement provisions of 11.5.4 and 11.5.5 will require closer stirrup spacings.

For repeated loading of flexural members, the possibility of inclined diagonal tension cracks forming at stresses appreciably smaller than under static loading should be taken into account in the design. In these instances, it would be prudent to use at least the minimum shear reinforcement expressed by Eq. (11-13) or (11-14), even though tests or calculations based on static loads show that shear reinforcement is not required.

R11.5.5.2 When a member is tested to demonstrate that its shear and flexural strengths are adequate, the actual member dimensions and material strengths are known. The strength used as a basis for comparison should therefore be that corresponding to a strength reduction factor of unity ($\phi = 1.0$), i.e. 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.

R11.5.5.3 Tests[11.9] have indicated the need to increase the minimum area of shear reinforcement as concrete strength increases to prevent sudden shear failures when inclined cracking occurs. Equation (11-13) provides for a gradual increase in the minimum area of transverse reinforcement, while maintaining the previous minimum value.

R11.5.5.4 Tests[11.24] 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.

Eq. (11-14) may be used only for prestressed members meeting the minimum prestress force requirements given in 11.5.5.4. This equation is discussed in Reference 11.24.

R11.5.6 **Design of shear reinforcement**.

Design of shear reinforcement is based on a modified truss analogy. The truss analogy assumes that the total shear is carried by shear reinforcement. However, considerable research on both non-prestressed and prestressed members has indicated that shear reinforcement needs to be designed to carry only the shear exceeding that which causes inclined cracking, provided the diagonal members in the truss are assumed to be inclined at 45 deg.

Eq. (11-15), (11-16), and (11-17) are presented in terms of shear strength V_s attributed to the shear reinforcement. When shear reinforcement perpendicular to axis of member is used, the required area of shear reinforcement A_v and its spacing s are computed by

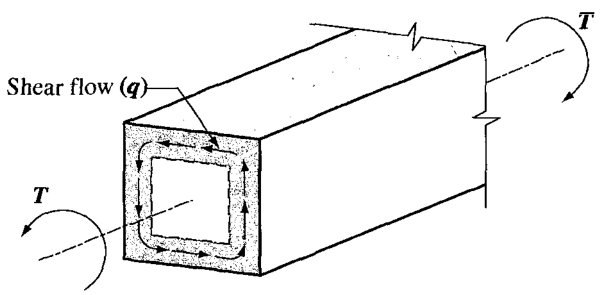
$$\frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_y d}$$

Research[11.25,11.26] has shown that shear behavior of wide beams with substantial flexural reinforcement is improved if the transverse spacing of stirrup legs across the section is reduced.

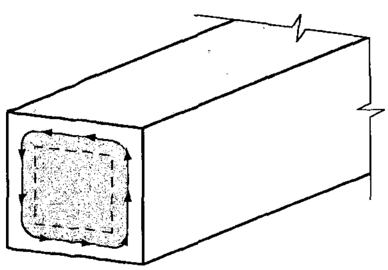
R11.5.6.3 Although the transverse reinforcement in a circular section may not consist of straight legs, tests indicate that Eq. (11-15) is conservative if d *is* taken as defined in 11.3.3.[11.16, 11.17]

SECTION R11.6
DESIGN FOR TORSION

The design for torsion is based on a thin-walled tube, space truss analogy. A beam subjected to torsion is idealized as a thin-walled tube with the core concrete cross section in a solid beam neglected as shown in Fig. R11.6(a). Once a reinforced concrete beam has cracked in torsion, its torsional resistance is provided primarily by closed stirrups and longitudinal bars located near the surface of the member. In the thin-walled tube analogy the resistance is assumed to be provided by the outer skin of the cross section roughly centered on the closed stirrups. Both hollow and solid sections are idealized as thin-walled tubes both before and after cracking.



(a) Thin-walled tube



(b) Area enclosed by shear flow path

Fig. R11.6 - (a) Thin-walled tube; (b) area enclosed by shear flow path

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. R11.6(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. R11.6(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 shearflow. For a hollow member with continuous walls, A_o includes the area of the hole.

The elliptical interaction between the shear carried by the concrete, V_c and the torsion carried by the concrete is not considered. V_c remains constant at the value it has when there is no torsion, and the torsion carried by the concrete is always taken as zero.

The design procedure is derived and compared with test results in References 11.27 and 11.28.

R11.6.1 Threshold torsion.

Torques that do not exceed approximately one-quarter of the cracking torque T_{cr} will not cause a structurally significant reduction in either the flexural or shear strength and can be ignored. The cracking torsion under pure torsion T_{cr} is derived by replacing the actual section with an equivalent thin-walled tube with a wall thickness t prior to cracking of $0.75A_{cp}/P_{cp}$ and an area enclosed by the wall centerline A_o equal to $2A_{cp}/3$. Cracking is assumed to occur when the principal tensile stress reaches $(1/3)\sqrt{f_c'}$. In a nonprestressed beam loaded with torsion alone, the principal tensile stress is equal to the torsional shear stress, $\tau = T/(2A_o t)$. Thus, cracking occurs when τ reaches $(1/3)\sqrt{f_c'}$ giving the cracking torque T_{cr} as:

$$T_{cr} = \frac{1}{3}\sqrt{f_c'}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

For solid members, the interaction between the cracking torsion and the inclined cracking shear is approximately circular or elliptical. For such a relationship, a torque of $0.25T_{cr}$ as used in 11.6.1, corresponds to a reduction of 3% in the inclined cracking shear. This reduction in the inclined cracking shear was considered negligible. The stress at cracking $(1/3)\sqrt{f_c'}$ 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 $(1/3)\sqrt{f_c'}$ is $\sqrt{1+3f_{pc}/\sqrt{f'_c}}$ times the corresponding torque in a nonprestressed beam. A similar modification is made in part (c) of 11.6.1 for members subjected to axial load and torsion.

For torsion, a hollow member is defined as having one or more longitudinal voids, such as a single-cell or multiple-cell box girder. Small longitudinal voids, such as un-grouted post-tensioning ducts that result in A_g / A_{cp} greater than or equal to 0.95, can be ignored when computing the threshold torque in 11.6.1. The interaction between torsional cracking and shear cracking for hollow sections is assumed to vary from the elliptical relationship for members with small voids, to a straight-line relationship for thin-walled sections with large voids. For a straight-line interaction, a torque of $0.25T_{cr}$ would cause a reduction in the inclined cracking shear of about 25%. This reduction was judged to be excessive.

Two changes are made to modify 11.6.1 to apply to hollow sections. First, the minimum torque limits are multiplied by A_g / A_{cp} because tests of solid and hollow beams[11.29] indicate that the cracking torque of a hollow section is approximately A_g / A_{cp} times the cracking torque of a solid section with the same outside dimensions. The second change is 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.

R11.6.2 Calculation of factored torsional moment T_u

R11.6.2.1 and
R11.6.2.2 In designing for torsion in reinforced concrete structures, two conditions may be identified:[11.30,11.31]

(a) The torsional moment cannot be reduced by redistribution of internal forces (11.6.2.1). This is referred to as equilibrium torsion, since the torsional moment is required for the structure to be in equilibrium.

For this condition, illustrated in Fig. R11.6.2.1, torsion reinforcement designed according to 11.6.3 through 11.6.6 must be provided to resist the total design torsional moments.

(b) The torsional moment can be reduced by redistribution of internal forces after cracking (11.6.2.2) if the torsion arises from the member twisting to maintain compatibility of deformations. This type of torsion is referred to as compatibility torsion.

For this condition, illustrated in Fig. R11.6.2.2, the torsional stiffness before cracking corresponds to that of the uncracked section according to St. Venant's theory. At torsional cracking, however, a large twist occurs under an essentially constant torque, resulting in a large redistribution of forces in the structure.[11.30,11.31] The cracking torque under combined shear, flexure, and torsion corresponds to a principal tensile stress somewhat less than the $(1/3)\sqrt{f_c'}$ quoted in R11.6.1.

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 11.6.1, is not applied here. Thus, the torque after redistribution is larger and hence more conservative.

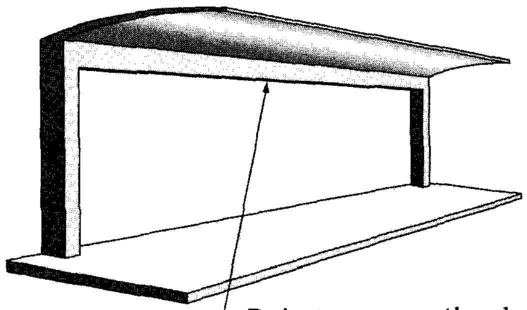


Fig. R11.6.2.1 - Design torque may not be reduced (11.6.2.1)

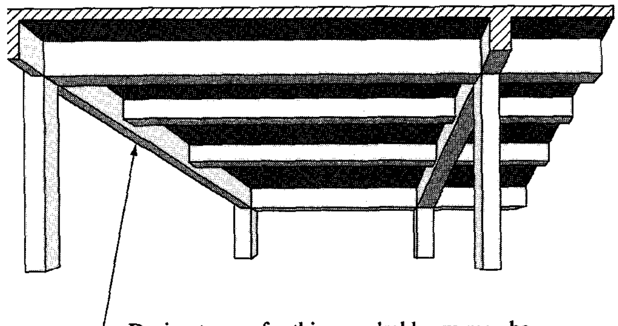


Fig. R11.6.2.2 - Design torque may be reduced (11.6.2.2)

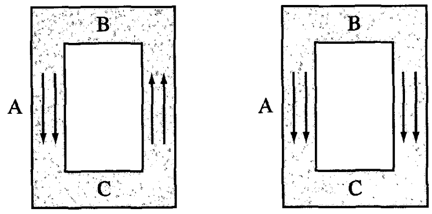
Section 11.6.2.2 applies to typical and regular framing conditions. With layouts that impose significant torsional rotations within a limited length of the member, such as a heavy in 11.6.2.2. Torque loading located close to a stiff column, or a column that rotates in the reverse directions because of other loading, a more exact analysis is advisable.

When the factored torsional moment from an elastic analysis based on uncracked section properties is between the values in 11.6.1 and the values given in this section, torsion reinforcement should be designed to resist the computed torsional moments.

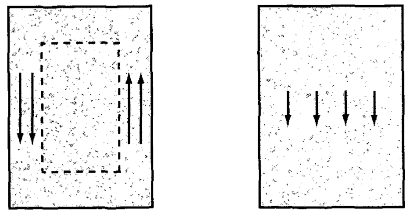
R11.6.2.4 and
R11.6.2.5 It is not uncommon for a beam to frame into one side of a girder near the support of the girder. In such a case a concentrated shear and torque are applied to the girder.

R11.6.3 Torsional moment strength

R11.6.3.1 The size of a cross section is limited for two reasons, first to reduce unsightly cracking and second to prevent crushing of the surface concrete due to inclined compressive stresses due to shear and torsion. In Eq. (11-18) and (11-19), the two terms on the left hand side are the shear stresses due to shear and torsion. The sum of these stresses may not exceed the stress causing shear cracking plus $2/3\sqrt{f_c'}$ similar to the limiting strength given in 11.5.6.9 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 necessary to check against crushing of the web since this happens at higher shear stresses.



(a) Hollow section



(b) Solid section

Fig. R11.6.3.1 - Addition of torsional and shear stresses

In a hollow section, the shear stresses due to shear and torsion both occur in the walls of the box as shown in Fig. 11.6.3.1(a) and hence are directly additive at point A as given in Eq. (11-19). In a solid section the shear stresses due to torsion act in the "tubular" outside section while the shear stresses due to V_u. are spread across the width of the section as shown in Fig. R11.6.3.1(b). For this reason stresses are combined in Eq. (11-18) using the square root of the sum of the squares rather than by direct addition.

R11.6.3.2 Generally, the maximum will be on the wall where the torsional and shearing stresses are additive [Point A in Fig. R11.6.3.1(a)]. If the top or bottom flanges are thinner than the vertical webs, it may be necessary to evaluate Eq. (11-19) at points B and C in Fig. R11.6.3.1(a). At these points the stresses due to the shear force are usually negligible.

R11.6.3.4 Limiting the design yield strength of torsion reinforcement to 420 MPa provides a control on diagonal crack width.

R11.6.3.5 The factored torsional resistance ϕT_n must equal or exceed the torsion T_n due to the factored loads. In the calculation of T_n all the torque is assumed to be resisted by stirrups and longitudinal steel with $T_c = 0$. At the same time, the shear resisted by concrete V_c is assumed to be unchanged by the presence of torsion. For beams with V_u greater than about $0.8\phi V_c$ the resulting amount of combined shear and torsional reinforcement is essentially the same. For smaller values of V_u, more shear and torsion reinforcement will be required.

R11.6.3.6 Eq. (11-21) is based on the space truss analogy shown in Fig. R11.6.3.6(a) with compression diagonals at an angle θ, assuming the concrete carries no tension and the reinforcement yields. After torsional cracking develops, the torsional resistance is provided mainly by closed stirrups, longitudinal bars, and compression diagonals. The concrete outside these stirrups is relatively ineffective. For this reason A_o , the area enclosed by the shear flow path around the perimeter of the tube, is defined after cracking in terms of A_{oh} , the area enclosed by the centerline of the outermost closed hoops. The area A_{oh} is shown in Fig. R11.6.3.6(b) for various cross sections. In an I-, T-, or L-shaped section, A_{oh} is taken as that area enclosed by the outermost legs of interlocking stirrups as shown in Fig. R11.6.3.6(b). The expression for A_o given in Reference 11.32 may be used if greater accuracy is desired.

The shear flow q in the walls of the tube, discussed in R11.6, can be resolved into the shear forces V_1 to V_4 acting in the individual sides of the tube or space truss, as shown in Fig. R11.6.3.6(a).

The angle θ can be obtained by analysis[11.32] or may be taken to be equal to the values given in 11.6.3.6 (a) or (b). The same value of θ should be used in both Eq. (11-21) and (11-22). As θ gets smaller, the amount of stirrups required by Eq. (11-21) decreases. At the same time the amount of longitudinal steel required by Eq. (11-22) increases.

R11.6.3.7 Fig. R11.6.3.6(a) shows the shear forces V_1 to V_4 resulting from the shear flow around the walls of the tube. On a given wall of the tube, the shear flow V_i is resisted tube 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 he longitudinal steel to complete the resolution of V_i.

Fig. R11.6.3.7 shows the diagonal compressive stresses and the axial tension force, N_i, acting on a short segment along one wall of the tube. Because the shear flow due to torsion is constant at all points around the perimeter of the tube, the resultants of D_i and N_i act through the midheight of side i. As a result, half of N_i can be assumed to be resisted by each of the top and bottom chords as shown. Longitudinal reinforcement with a capacity $A_e f_{ye}$ should be provided to resist the sum of the N_i forces, $\sum N_i$, acting in all of the walls of the tube.

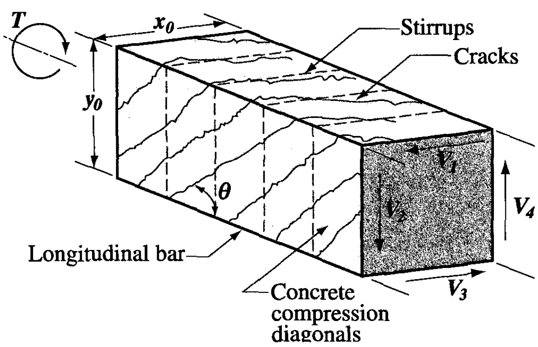


Fig. R11.6.3.6(a) - Space truss analogy

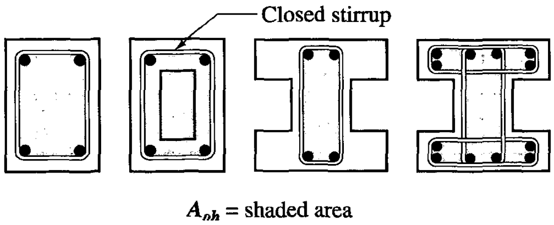


Fig. R11.6.3.6(b) - Definition of A_{oh}

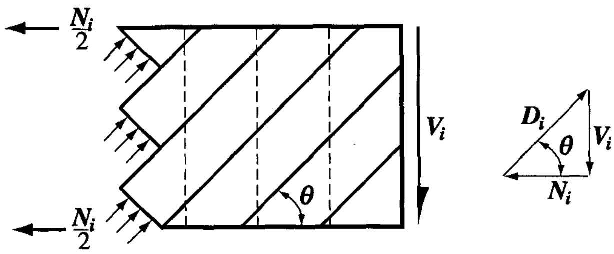


Fig. R11.6.3.7 - Resolution of shear force V_i; into diagonal compression force D_i; and axial tension force N_i in one wall of the tube

In the derivation of Eq. (11-22), axial tension forces are summed along the sides of the area A_o. These sides form a perimeter length, p_o, approximately equal to the length of the line joining the centers of the bars in the corners of the tube. For ease in computation this has been replaced with the perimeter of the closed stirrups, p_h.

Frequently, the maximum allowable stirrup spacing governs the amount of stirrups provided. Furthermore, when combined shear and torsion act, the total stirrup area is the sum of the amounts provided for shear and torsion. To avoid the need to provide excessive amounts of longitudinal reinforcement, 11.6.3.7 states that the A_t / s used in calculating A_l at any given section should be taken as the A_t / s calculated at that section using Eq. (11-21).

R11.6.3.8 The stirrup requirements for torsion and shear are added and stirrups are provided to supply at least the total amount required. Since 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:

$$Total\left(\frac{A_v + t}{s}\right) = \frac{A_v}{s} + 2\frac{A_t}{s}$$

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 since 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 d, but not less than $12d_b$, past where it is no longer needed for flexure as required in 12.10.3.

R11.6.3.9 The longitudinal tension due to torsion is offset in part by the compression in the flexural compression zone, allowing a reduction in the longitudinal torsion steel required in the compression zone.

R11.6.3.10 As explained in R11.6.3.7, torsion causes an axial tension force. In a nonprestressed beam this force is resisted by longitudinal reinforcement having an axial tensile capacity of $A_\ell f_{y\ell}$. This steel is in addition to the flexural reinforcement and is distributed uniformly around the sides of the perimeter so that the resultant of $A_\ell f_{y\ell}$ acts along the axis of the member.

In a prestressed beam, the same technique (providing additional reinforcing bars with capacity $A_\ell f_{y\ell}$) can be followed, or the designer can use any overcapacity of the prestressing steel to resist some of the axial force $A_\ell f_{y\ell}$ outlined in the next paragraph.

In a prestressed beam, the prestressing steel stress at ultimate at the section of the maximum moment is f_{ps}. At other sections, the prestressing steel stress at ultimate will be between f_{se} and f_{ps}. A portion of the $A_\ell f_{y\ell}$ force acting on the sides of the perimeter where the prestressing steel is located can be resisted by a force $A_{ps}\Delta f_p$ in the prestressing steel, where Δf_p is f_{ps} minus the prestressing steel stress due to flexure at the ultimate load at the section in question. This can be taken as M_u at the section, divided by $(\phi 0.9 d_p A_{ps})$, but Δf_p should not be more than 420 MPa.

Longitudinal reinforcing bars will be required on the other sides of the member to provide the remainder of the $A_\ell f_{y\ell}$ force, or to satisfy the spacing requirements given in 11.6.6.2, or both.

R11.6.4 Details of torsional reinforcement

R11.6.4.1 Both longitudinal and closed transverse reinforcement are required to resist the diagonal tension stresses due to torsion. The stirrups must be closed, since 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 spoils off at high torques.[11.33] This renders lapped-spliced stirrups ineffective, leading to a premature torsional failure.[11.34] In such cases, closed stirrups should not be made up of pairs of U-stirrups lapping one another.

R11.6.4.2 When a rectangular beam fails in torsion, the corners of the beam tend to spoil off due to the inclined compressive stresses in the concrete diagonals of the space truss changing direction at the corner as shown in Fig. R11.6.4.2 (a). In tests,[11.33] closed stirrups anchored by 90 deg hooks failed when this occurred. For this reason, 135 deg hooks are preferable for torsional stirrups in all cases. In regions where this spalling is prevented by an adjacent slab or flange, 11.6.4.2(b) relaxes this and allows 90 deg hooks.

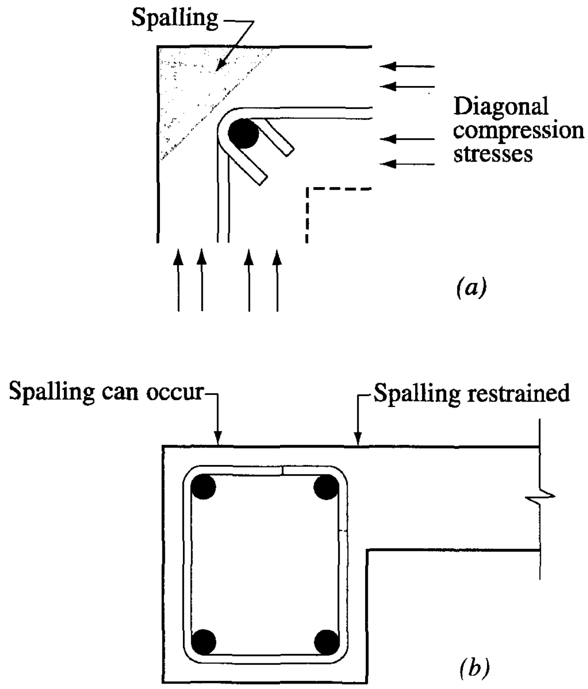


Fig. R11.6.4.2 - Spalling of corners of beams loaded in torsion

R11.6.4.3 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.

R11.6.4.4 The closed stirrups provided for torsion in a hollow section should be located in the outer half of the wall thickness effective for torsion where the wall thickness can be taken as A_{oh} / p_h.

R11.6.5 Minimum torsion reinforcement

R11.6.5.1 and
R11.6.5.2 If a member is subject to a factored torsional moment T_u greater than the values specified in 11.6.1, the minimum amount of transverse web reinforcement for combined shear and torsion is $0.35 b_w s / f_y$. The differences in the definition of A_v, and the symbol A_t should be noted; A_v is the area of two legs of a closed stirrup while A_t is the area of only one leg of a closed stirrup.

Tests[11.9] of high-strength reinforced concrete beams have indicated the need to increase the minimum area of shear reinforcement to prevent shear failures when inclined cracking occurs. Although there are a limited number of tests of high-strength concrete beams in torsion, the equation for the minimum area of transverse closed stirrups has been changed for consistency with calculations required for minimum shear reinforcement.

R11.6.5.3 Reinforced concrete beam specimens with less than 1 percent torsional reinforcement by volume have failed in pure torsion at torsional cracking.[11.27]

R11.6.6 **Spacing of torsion reinforcement**

R11.6.6.1 The spacing of the stirrups is limited to ensure the development of the ultimate torsional strength of the beam, to prevent excessive loss of torsional stiffness after cracking, and to control crack widths. For a square cross section the $p_h/8$ limitation requires stirrups at $d/2$, which corresponds to 11.5.4.1.

R11.6.6.2 In R11.6.3.7 it was shown that longitudinal reinforcement is needed to resist the sum of the longitudinal tensile forces due to torsion in the walls of the thin-walled tube. Since the force acts along the centroidal axis of the section, the centroid of the additional longitudinal reinforcement for torsion should approximately coincide with the centroid of the section. The code accomplishes this by requiring the longitudinal torsional reinforcement to be distributed around the perimeter of the closed stirrups. Longitudinal bars or tendons are required in each corner of the stirrups to provide anchorage for the legs of the stirrups. Corner bars have also been found to be very effective in developing torsional strength and in controlling cracks.

R11.6.6.3 The distance $(b_t + d)$ beyond the point theoretically required for torsional reinforcement is larger than that used for shear and flexural reinforcement because torsional diagonal tension cracks develop in a helical form.

SECTION R11.7
SHEAR-FRICTION

R11.7.1 With the exception of 11.7, virtually all provisions regarding shear are intended to prevent diagonal tension failures rather than direct shear transfer failures. The purpose of 11.7 is to provide design methods for conditions where shear transfer should be considered: an interface between concretes cast at different times, an interface between concrete and steel, reinforcement details for precast concrete structures, and other situations where it is considered appropriate to investigate shear transfer across a plane in structural concrete. (See References 11.35 and 11.36).

R11.7.3 Although untracked concrete is relatively strong in direct shear there is always the possibility that a crack will form in an unfavorable location. The shear-friction concept assumes that such a crack will form, and that reinforcement must be provided across the crack to resist relative displacement along it. When shear acts along a crack, one crack face slips relative to the other. If the crack faces are rough and irregular, this slip is accompanied by separation of the crack faces. At ultimate, the separation is sufficient to stress the reinforcement crossing the crack to its yield point. The reinforcement provides a clamping force $A_{vf} f_y$ across the crack faces. The applied shear is then resisted by friction between the crack faces, by resistance to the shearing off of protrusions on the crack faces, and by dowel action of the reinforcement crossing the crack. Successful application of 11.7 depends on proper selection of the location of an assumed crack.[11.18.11.35]

The relationship between shear-transfer strength and the reinforcement crossing the shear plane can be expressed in various ways. Eq. (11-25) and (11-26) of 11.7.4 are based on the shear-friction model. This gives a conservative prediction of shear-transfer strength. Other relationships that give a closer estimate of shear-transfer strength[11.18,11.37,11.38] can be used under the provisions of 11.7.3. For example when the shear-friction reinforcement is perpendicular to the shear plane, the shear strength V_n is given by[11.37, 11.38]

$$V_n = 0.8 A_{vf} f_v + A_c K_1$$

where A_c is the area of concrete section resisting shear transfer (mm^2) and K_1 = 2.8 MPa for normalweight concrete, 1.4 MPa for all-lightweight concrete, and 1.7 MPa for sand-lightweight concrete. These values of K_1 apply to both monolithically cast concrete and to concrete cast against hardened concrete with a rough surface, as defined in 11.7.9.

In this equation, the first term represents the contribution of friction to shear-transfer resistance (0.8 representing the coefficient of friction). The second term represents the sum of 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

$$V_n = A_{vf} f_y \left(0.8 \sin\alpha_f + \cos\alpha_f\right) + A_c K_1 \sin^2\alpha_f$$

where α_f is the angle between the shear-friction reinforcement and the shear plane, (i.e. $0 < \alpha_f < 90$ deg).

When using the modified shear-friction method, the terms $\left(A_{vf} f_y / A_c\right)$ or $\left(A_{vf} f_y \sin\alpha_f / A_c\right)$ should not be less than 1.4 MPa for the design equations to be valid.

R11.7.4 Shear-friction design method

R11.7.4.1 The required area of shear-transfer reinforcement A_{vf} is computed using

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$

The specified upper limit on shear strength should also be observed.

R11.7.4.2 When the shear-friction reinforcement is inclined to the shear plane, such that the component of the shear force parallel to the reinforcement tends to produce tension in the reinforcement, as shown in Fig. R11.7.4, part of the shear is resisted by the component parallel to the shear plane of the tension force in the reinforcement. Eq. (11-26) should be used only when the shear force component parallel to the reinforcement produces tension in the reinforcement, as shown in Fig. R11.7.4. When α_f is greater than 90 deg, the relative movement of the surfaces tends to compress the bar and Eq. (11-26) is not valid.

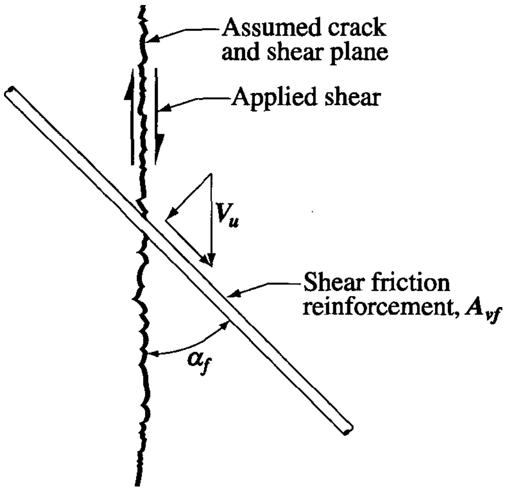


Fig. R11.7.4 - Shear-friction reinforcement at an angle to assumed crack

R11.7.4.3 In the shear-friction method of calculation, it is assumed that all the shear resistance is due to the friction between the crack faces. It is, therefore, necessary to use artificially high values of the coefficient of friction in the shear-friction equations so that the calculated shear strength will be in reasonable agreement with test results. For concrete cast against hardened concrete not roughened in accordance with 11.7.9, shear resistance is primarily due to dowel action of the reinforcement and tests[1139] indicate that reduced value of $\mu = 0.6\lambda$ specified for this case is appropriate.

The value of for concrete placed against as-rolled structural steel relates μ specified to the design of connections between precast concrete members, or between structural steel members and structural concrete members. The shear-transfer reinforcement may be either reinforcing bars or headed stud; shear connectors; also, field welding to steel plates after casting of concrete is common. The design of shear connectors for composite action of concrete slabs and steel beams is not covered by these provisions, but should be in accordance with Reference 11.40.

R11.7.5 This upper limit on shear strength is specified because Eq. (11-25) and (11-26) become un-conservative if V_n has a greater value.

R11.7.7 If a resultant tensile force acts across a shear plane, reinforcement to carry that tension 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.[11.41]

It has also been demonstrated experimentally [11.36] that if a resultant compressive force acts across a shear plane, the shear-transfer strength is a function of the sum of the resultant compressive force and the force $A_{vf} f_y$ in the shear-friction reinforcement. In design, advantage should be taken of the existence of a compressive force across the shear plane to reduce the amount of shear-friction reinforcement required, only if it is certain that the compressive force is permanent.

R11.7.8 If no moment acts across the shear plane, reinforcement should be uniformly distributed along the shear plane to minimize crack widths. If a moment acts across the shear plane, it is desirable to distribute the shear-transfer reinforcement primarily in the flexural tension zone.I

Since 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 see Reference 11.18.

SECTION R11.8
DEEP BEAMS

R11.8.1 The behavior of a deep beam is discussed in References 11.5 and 11.38. For a deep beam supporting gravity loads, this section applies if the loads are applied on the top of the beam and the beam is supported on its bottom face. If the loads are applied through the sides or bottom of such a member, the design for shear should be the same as for ordinary beams.

The longitudinal reinforcement in deep beams should be extended to the supports and adequately anchored by embedment, hooks, or welding to special devices. Bent-up bars are not recommended.

R11.8.2 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 nonlinear stress fields when proportioning deep beams. Such analyses should consider the effects of cracking on the stress distribution.

R11.8.3 and
R11.8.4 –
R11.8.4 and
R11.8.5 The relative amounts of horizontal and vertical shear reinforcement are set based on tests[11.42,11.43,11.44] that have shown that vertical shear reinforcement is more effective than horizontal shear reinforcement. The maximum spacing of bars is limited to 300 mm to restrain the width of the cracks.

SECTION R11.9
SPECIAL PROVISIONS FOR BRACKETS AND CORBELS

Brackets and corbels are cantilevers having shear span-to-depth ratios not greater than unity, which tend to act as simple trusses or deep beams, rather than flexural members designed for shear according to 11.3.

The corbel shown in Fig. R11.9.1 may fail by shearing along the interface between the column and the corbel, by yielding of the tension tie, by crushing or splitting of the compression strut, or by localized bearing or shearing failure under the loading plate. These failure modes are illustrated and are discussed more fully in Reference 11.1. The notation used in 11.9 is illustrated in Fig. R11.9.2.

R11.9.1 An upper limit of 1.0 for a/d is imposed for design by 11.9.3 and 11.9.4 for two reasons. First, for shear span-to-depth ratios exceeding unity, the diagonal tension cracks are less steeply inclined and the use of horizontal stirrups alone as specified in 11.9.4 is not appropriate. Second, this method of design has only been validated experimentally for a/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.

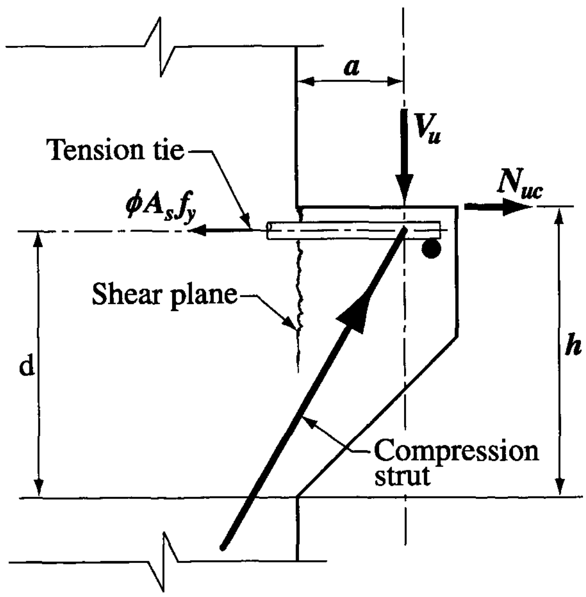


Fig. R11.9.1 - Structural action of a corbel

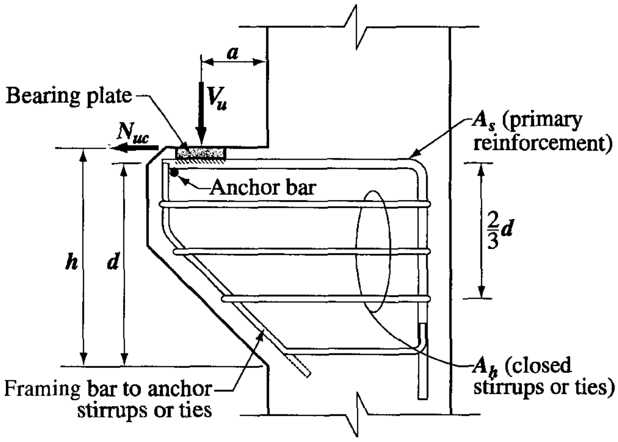


Fig. R11.9.2 - Notation used in Section 11.9

R11.9.2 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 [11.45] in corbels having depths at the outside edge of the bearing area less than required in this section of the code.

R11.9.3.1 Corbel and bracket behavior is predominantly controlled by shear; therefore, a single value of $\phi = 0.75$ is required for all design conditions.

R11.9.3.2.2 Tests [11.46] have shown that the maximum shear strength of lightweight concrete corbels or brackets is a function of both f_c' and a/d. No data are available for corbels or brackets made of sand-lightweight concrete. As a result, the same limitations have been placed on both all-lightweight and sand-lightweight brackets and corbels.

R11.9.3.3 Reinforcement required to resist moment can be calculated using flexural theory. The factored moment is calculated by summing moments about the flexural reinforcement at the face of support.

R11.9.3.4 Because the magnitude of horizontal forces acting on corbels or brackets cannot usually be determined with great accuracy, it is required that N_{uc} be regarded as a live load.

R11.9.3.5 Tests [11.46] suggest that the total amount of reinforcement $(A_s + A_h)$ required to cross the face of support should be the greater of:

a) The sum of A_{vf} calculated according to 11.9.3.2 and A_n calculated according to 11.9.3.4;

b) The sum of 1.5 times A_f calculated according to 11.9.3.3 and A_n calculated according to 11.9.3.4.

If (a) controls, $A_s = (2A_{vf}/3 + A_n)$ is required as primary tensile reinforcement, and the remaining $A_{vf}/3$ should be provided as closed stirrups parallel to A_s and distributed within $2d/3$, adjacent to A_s. Section 11.9.4 satisfies this by requiring $A_h = 0.5\left(2A_{vf}/3\right)$.

If (b) controls, $A_s = \left(A_f + A_n\right)$ is required as primary tension reinforcement, and the remaining $A_f/2$ should be provided as closed stirrups parallel to A_s and distributed within $2d/3$, adjacent to A_s. Again 11.9.4 satisfies this requirement.

R11.9.4 Closed stirrups parallel to the primary tension reinforcement are necessary to prevent a premature diagonal tension failure of the corbel or bracket. The required area of closed stirrups $A_h = 0.5(A_s - A_n)$ automatically yields the appropriate amounts, as discussed in R11.9.3.5 above.

R11.9.5 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}.

R11.9.6 Because the horizontal component of the inclined concrete compression strut (see Fig. R11.9.1) is transferred to the primary tension reinforcement at the location of the vertical load, the reinforcement A_s is essentially uniformly stressed from the face of the support to the point where the vertical load is applied. It should, therefore, be anchored at its outer end and in the supporting column, so as to be able to develop its yield strength from the face of support to the vertical load. Satisfactory anchorage at the outer end can be obtained by bending the A_s bars in a horizontal loop as specified in (b), or by welding a bar of equal diameter or a suitably sized angle across the ends of the A_s bars. The

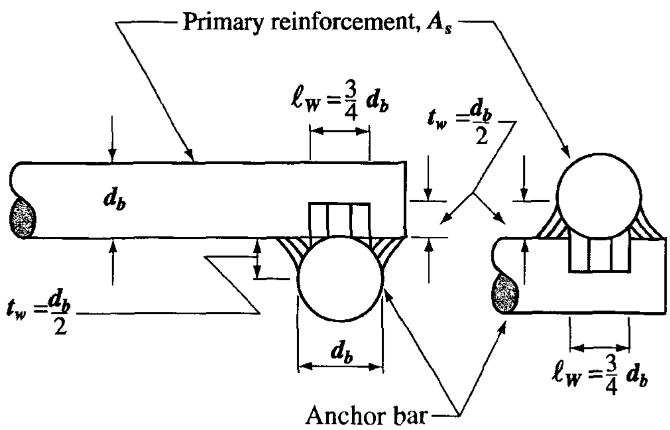


Fig. R11.9.6 - Weld details used in tests of Reference 11.45

welds should be designed to develop the yield strength of the reinforcement A_s. The weld detail used successfully in the corbel tests reported in Reference 11.45 is shown in Fig. R11.9.6. The reinforcement A_s should be anchored within the supporting column in accordance with the requirements of Chapter 12. See additional discussion on end anchorage in R12.10.6.

R11.9.7 The restriction on the location of the bearing area is necessary to ensure development of the yield strength of the reinforcement A_s near the load. When corbels are designed to resist horizontal forces, the bearing plate should be welded to the tension reinforcement A_s.

SECTION R11.10
SPECIAL PROVISIONS FOR WALLS

R11.10.1 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.

R11.10.3 Although the width-to-depth ratio of shear-walls is less than that for ordinary beams, tests [11.47] on shear-walls with a thickness equal to $\ell_w/25$ have indicated that ultimate shear stresses in excess of $(5/6)\sqrt{f_c'}$; can be obtained.

R11.10.5 and
R11.10.6 Eq. (11-29) and (11-30) may be used to determine the inclined cracking strength at any section through a shear wall. Eq. (11-29) corresponds to the occurrence of a principal tensile stress of approximately $(1/3)\sqrt{f_c'}$ at the centroid of the shear wall cross section. Eq. (11-30) corresponds approximately to the occurrence of a flexural tensile stress of $(1/2)\sqrt{f_c'}$ 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. (11-29) will control before this term becomes negative. When this term becomes negative Eq. (11-29) should be used.

R11.10.7 The values of V_c computed from Eq. (11-29) and (11-30) at a section located a lesser distance of $\ell_w/2$ or $h_w/2$ above the base apply to that and all sections between this section and the base. However, the maximum factored shear force V_u at any section, including the base of the wall, is limited to ϕV_n in accordance with 11.10.3.

R11.10.9 **Design of shear reinforcement for walls**
Both horizontal and vertical shear reinforcement are required for all walls. For low walls, test data[11.48] indicate that horizontal shear reinforcement becomes less effective with vertical reinforcement becoming more effective. This change in effectiveness of the horizontal versus vertical reinforcement is recognized in Eq. (11-32); when h_w/ℓ_w is less than 0.5, the amount of vertical reinforcement is equal to the amount of horizontal reinforcement.

When h_w/ℓ_w is greater than 2.5, only a minimum amount of vertical reinforcement is required $(0.0025 s_1 h)$.

Eq. (11-31) 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 11.10.9.4 within the spacing limitation of 11.10.9.5.

SECTION R11.11
TRANSFER OF MOMENTS TO COLUMNS

R11.11.1 Tests[11.49] have shown that the joint region of a beam-to-column connection in the interior of a building does not require shear reinforcement if the joint is confined on four sides by beams of approximately equal depth. However, joints without lateral confinement, such as at the exterior of a building, need shear reinforcement to prevent deterioration due to shear cracking[11.50].

SECTION R11.12
SPECIAL PROVISIONS FOR SLABS AND FOOTINGS

11.12.1 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.

R11.12.1.2 The critical section for shear in slabs subjected to bending in two directions follows the perimeter at the edge of the loaded area.[11.3] The shear stress acting on this section at factored loads is a function of $\sqrt{f_c'}$, and the ratio of the side dimension of the column to the effective slab depth. A much simpler design equation results by assuming a pseudocritical section located at a distance $d/2$ from the periphery of the concentrated load. When this is done, the shear strength is almost independent of the ratio of column size to slab depth. For rectangular columns, this critical section was defined by straight lines drawn parallel to and at a distance $d/2$ from the edges of the loaded area. Section 11.12.1.3 allows the use of a rectangular critical section.

For slabs of uniform thickness, it is sufficient to check shear on one section. For slabs with changes in thickness, such as the edge of drop panels, it is necessary to check shear at several sections.

For edge columns at points where the slab cantilevers beyond the column, the critical perimeter will either be three-sided or four-sided.

R11.12.2.1 For square columns, the shear stress due to ultimate loads in slabs subjected to bending in two directions is limited to $(1/3)\sqrt{f_c'}$. However, tests (Ref. 11.51) have indicated that the value of $(1/3)\sqrt{f_c'}$ is unconservative when the ratio β_c of the lengths of the long and short sides of a rectangular column or loaded area is larger than 2.0. In such cases, the actual shear stress on the critical section at punching shear failure varies from a maximum of about $(1/3)\sqrt{f_c'}$ around the corners of the column or loaded area, down to $(1/6)\sqrt{f_c'}$ or less along the long sides between the two end sections. Other tests (Ref. 11.52) indicates that v_c, decreases as the ratio b_o/d increases. Eq. (11-33) and (11-34) were developed to account for these two effects. The words "interior," "edge," and "corner columns" in 11.12.2.1(b) refer to critical sections with 4, 3, and 2 sides respectively.

For shapes other than rectangular, β_c is taken to be the ratio of the longest overall dimension of the effective loaded area to the largest overall perpendicular dimension of the effective loaded area, as illustrated for an L-shaped reaction area in Fig. R11.12.2. The effective loaded area is that area totally enclosing the actual loaded area, for which the perimeter is a minimum.

R11.12.2.2 For prestressed slabs and footings, a modified form of code Eq. (11-33) and (11-36) is specified for two-way action shear strength. Research[11.53,11.54] indicates that the shear strength of two-way prestressed slabs around interior columns is conservatively predicted by Eq. (11-36). V_c from Eq. (11-36) corresponds to a diagonal tension failure of the concrete initiating at the critical section defined in 11.12.1.2. The mode of failure differs from a punching shear failure of the concrete compression zone around the perimeter of the loaded area predicted by Eq. (11-33). Consequently, the term β_c does not enter into Eq. (11-36). Design values for f_c' and f_{pc} are restricted due to limited test data available for higher values. When computing f_{pc}, loss of prestress due to restraint of the slab by shearwalls and other structural elements should be taken into account.

In a prestressed slab with distributed tendons, the V_p term in Eq. (11-36) 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 the total perimeter b_a of the critical section. Shear strength in this case is therefore conservatively taken the same as for a nonprestressed slab.

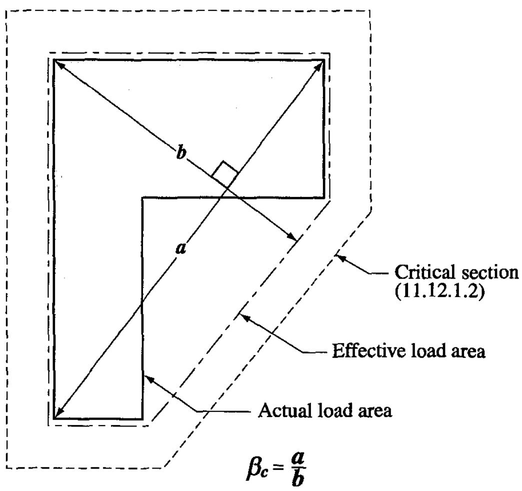


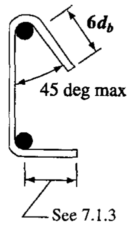
Fig. R11.12.2 - Value of β_c for a nonrectangular loaded area

R11.12.3 Research [11.55-11.59] has shown that shear reinforcement consisting of properly anchored bars or wires and single- or multiple-leg stirrups, or closed stirrups, can increase the punching shear resistance of slabs. The spacing limits given in 11.12.3.3 correspond to slab shear reinforcement details that have been shown to be effective. Sections 12.13.2 and 12.13.3 give anchorage requirements for stirrup-type shear reinforcement that should also be applied for bars or wires used as slab shear reinforcement. It is essential that this shear reinforcement engage longitudinal reinforcement at both the top and bottom of the slab, as shown for typical details in Fig. R11.12.3 (a) to (c). Anchorage of shear reinforcement according to the requirements of 12.13 is difficult in slabs thinner than 250 mm. Shear reinforcement consisting of vertical bars mechanically anchored at each end by a plate or head capable of developing the yield strength of the bars has been used successfully.[11.59]

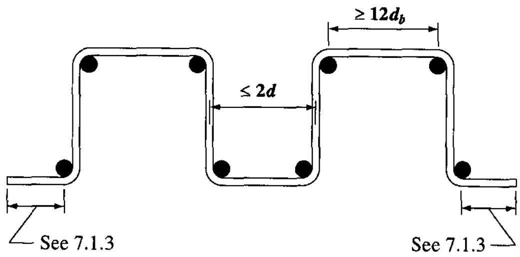
In a slab-column connection for which the moment transfer is negligible, the shear reinforcement should be symmetrical about the centroid of the critical section (Fig. R11.12.3 (d)). Spacing limits defined in 11.12.3.3 are also shown in Fig. R11.12.3 (d) and (e). At edge columns or for interior connections where moment transfer is significant, closed stirrups are recommended in a pattern as symmetrical as possible. Although the average shear stresses on faces *AD* and *BC* of the exterior column in Fig. R11.12.3 (e) are lower than on face *AB,* the closed stirrups extending from faces *AD* and *BC* provide some torsional capacity along the edge of the slab.

R11.12.4 Based on reported test data,[11.60] design procedures are presented for shearhead reinforcement consisting of structural steel shapes. For a column connection transferring moment, the design of shearheads is given in 11.12.6.3.

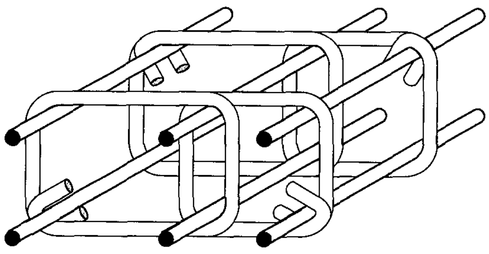
Three basic criteria should be considered in the design of shearhead reinforcement for connections transferring shear due to gravity load. First, a minimum flexural strength should be provided to ensure that the required shear strength of the slab is reached before the flexural strength of the shearhead 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 designer can reduce the negative slab reinforcement in proportion to the moment contribution of the shearhead at the design section.



(a) single-leg stirrup or bar



(b) multiple-leg stirrup or bar



(c) closed stirrups

Fig. R11.12.3(a-c) - Single or multiple-leg stirrup type slab shear reinforcement

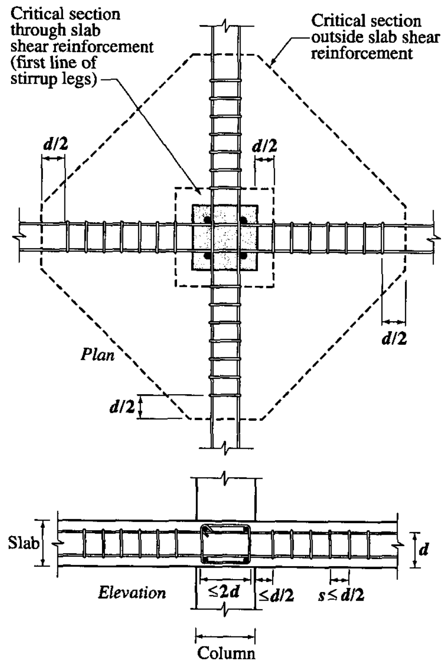


Fig. R11.12.3(d) - Arrangement of stirrup shear reinfor-cement, interior column.

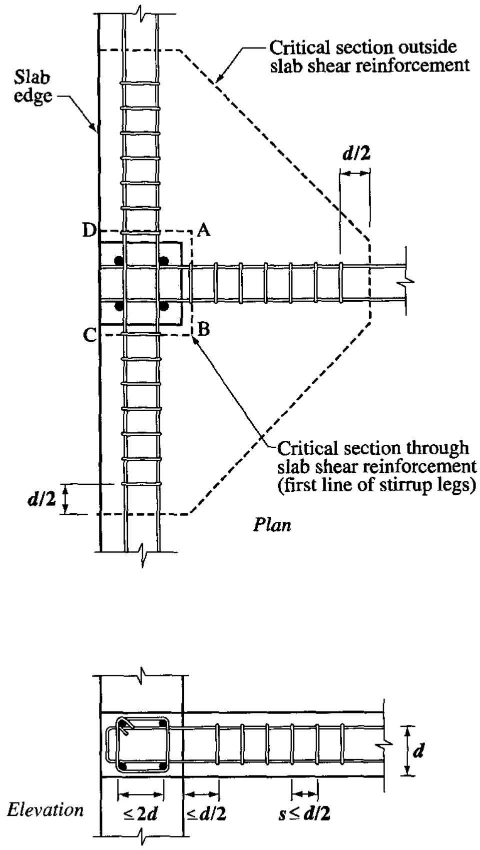


Fig. R11.12.3(e) - Arrangement of stirrup shear reinforcement, edge column

R11.12.4.5 and
R11.12.4.6 The assumed idealized shear distribution along an arm of a shearhead at an interior column is shown in Fig. R11.12.4.5. The shear along each of the arms is taken as $\alpha_v V_c / \eta$, where α_v and η are defined in 11.12.4.5 and 11.12.4.6, and V_c is defined in 11.12.2.1. However, the peak shear at the face of the column is taken as the total shear considered per arm $V_u / \phi\eta$ minus the shear considered carried to the column by the concrete compression zone of the slab. The latter term is expressed as $(V_c / \eta)(1 - \alpha_v)$, so that it approaches zero for a heavy shearhead and approaches $V_u / \phi\eta$ when a light shearhead is used. Equation (11-37) then follows from the assumption that ϕV_c is about one-half the factored shear force V_u. In this equation, M_p is the required plastic moment strength of each shearhead arm necessary to ensure that factored shear V_u is attained as the moment strength of the shearhead is reached. The quantity ℓ_v is the length from the center of the column to the point at which the shearhead is no longer required, and the distance $c_1 / 2$ is one-half the dimension of the column in the direction considered.

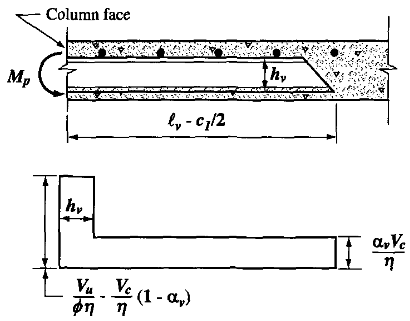


Fig. R11.12.4.5 - Idealized shear acting on shear head.

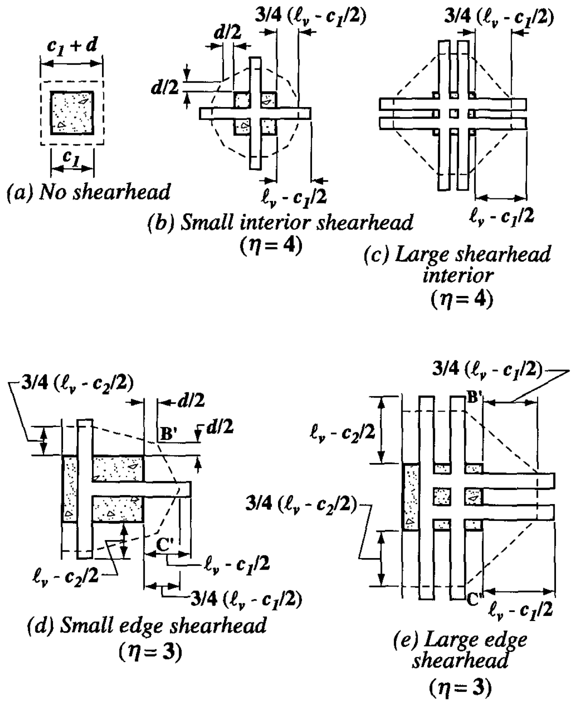


Fig. R11.12.4.7 - Location of critical section defined in 11.12.4.

R11.12.4.7 The test results. 60 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 $(1/3)\sqrt{f_c'}$. Although the use of over-reinforcing shearheads brought the shear strength back to about the equivalent of $(1/3)\sqrt{f_c'}$, the limited test data suggest that a conservative design is desirable. Therefore, the shear strength is calculated as $(1/3)\sqrt{f_c'}$ on an assumed critical section located inside the end of the shearhead reinforcement.

The critical section is taken through the shearhead arms three-fourths of the distance $[\ell_v - (c_1/2)]$ from the face of the column to the end of the shearhead. However, this assumed critical section need not be taken closer than $d/2$ to the column. See Fig. R11.12.4.7.

R11.12.4.9 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 contribution of the shearhead M_v can be conservatively computed from Eq. (11-38), in which ϕ is the factor for flexure.

R11.12.4.10 See R11.12.6.3.

R11.12.5 Openings in slabs

Provisions for design of openings in slabs (and footings) were developed in Reference 11.3. The locations of the effective portions of the critical section near typical openings and free edges are shown by the dashed lines in Fig. R11.12.5. Additional research11.51 has confirmed that these provisions are conservative.

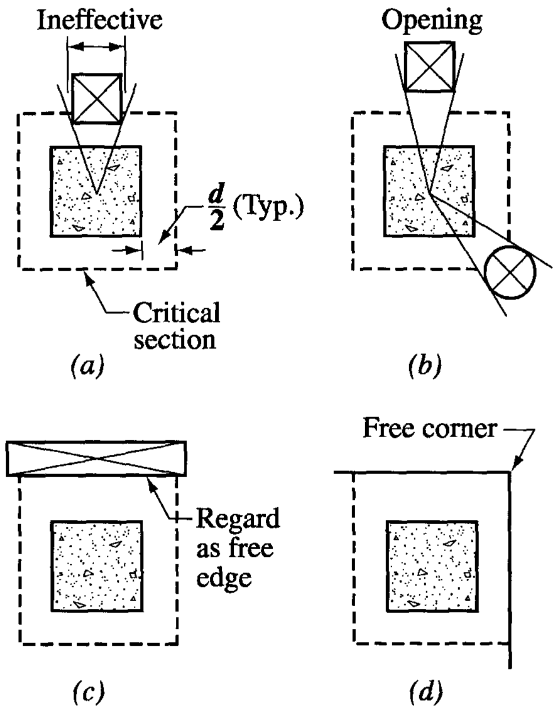


Fig. R11.12.5 - Effect of openings and free edges (effective perimeter shown with dashed lines)

R11.12.6 Transfer of moment in slab-column connections

R11.12.6.1 In Reference 11.61 it was found that where moment is transferred between a column and a slab, 60 percent of the moment should be considered transferred by flexure across the perimeter of the critical section defined in 11.12.1.2, and 40 percent by eccentricity of the shear about the centroid of the critical section. For rectangular columns, the portion of the moment transferred by flexure increases as the width of the face of the critical section resisting the moment increases, as given by Eq. (13-1).

Most of the data in Reference 11.61 were obtained from tests of square columns, and little information is available for round columns. These can be approximated as square columns. Fig. R13.6.2.5 shows square supports having the same area as some nonrectangular members.

R11.12.6.2 The stress distribution is assumed as illustrated in Fig. R11.12.6.2 for an interior or exterior column. The perimeter of the critical section, *ABCD,* is determined in accordance with 11.12.1.2. The factored shear force V_u and unbalanced moment M_u are determined at the centroidal axis c-c of the critical section. The maximum factored shear stress may be calculated from:
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. (11-39). For an interior column, A_c and J_c may be calculated by

A_c = area of concrete of assumed critical section

= $2d(c_1 + c_2 + 2d)$

J_c = property of assumed critical section analogous to polar moment of inertia

$$= \frac{d(c_1+d)^3}{6} + \frac{d(c_1+d)d^3}{6} + \frac{d(c_2+d)(c_1+d)^2}{2}$$

Similar equations may be developed for A_c and J_c for columns located at the edge or corner of a slab.

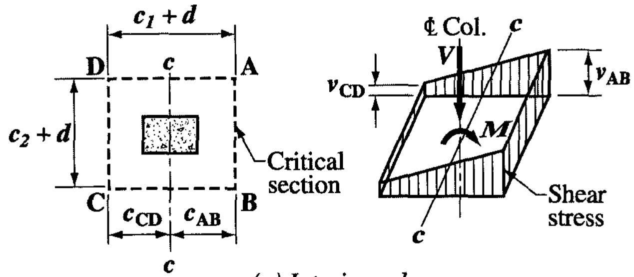
The fraction of the unbalanced moment between slab and column not transferred by eccentricity of the shear should be transferred by flexure in accordance with 13.5.3. A conservative method assigns the fraction transferred by flexure over an effective slab width defined in 13.5.3.2. Often designers concentrate column strip reinforcement near the column to accommodate this unbalanced moment. Available test data[11.61] seems to indicate that this practice does not increase shear strength but may be desirable to increase the stiffness of the slab-column junction.

Test data[11.62] indicate that the moment transfer capacity of a prestressed slab to column connection can be calculated using the procedures of 11.12.6 and 13.5.3.

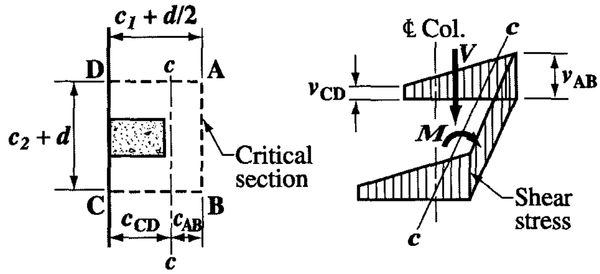
Where shear reinforcement has been used, the critical section beyond the shear reinforcement generally has a polygonal shape (Fig. R11.12.3(d) and (e)). Equations for calculating shear stresses on such sections are given in Reference 11.58.

R11.12.6.3 Tests[11.63] indicate that the critical sections are defined in 11.12.1.2(a) and 11.12.1.3 and are appropriate for calculations of shear stresses caused by transfer of moments even when shearheads are used. Then, even though the critical sections for direct shear and shear due to moment transfer differ, they coincide or are in close proximity at the column corners where the failures initiate. Because a shearhead attracts most of the shear as it funnels toward the column, it is conservative to take the maximum shear stress as the sum of the two components.

Section 11.12.4.10 requires the moment M_p 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.



(a) Interior column



(b) Edge column

Fig. R11.12.6.2 - Assumed distribution of shear stress

CHAPTER 12
DEVELOPMENT AND SPLICES OF REINFORCEMENT

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 in 12.10.2.

The strength reduction factor ϕ is not used in the development length and lap splice equations. An allowance for strength reduction is already included in the expressions for determining development and splice lengths.

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

SECTION R12.1
DEVELOPMENT OF REINFORCEMENT GENERAL

From a point of peak stress in reinforcement, some length of reinforcement or anchorage is necessary to develop the stress. This development length or anchorage is necessary on both sides of such peak stress points. Often the reinforcement continues for a considerable distance on one side of a critical stress point so that calculations need involve only the other side, for example, the negative moment reinforcement continuing through a support to the middle of the next span.

SECTION R12.2
DEVELOPMENT OF DEFORMED BARS AND DEFORMED WIRE IN TENSION

The general development length equation (Eq. (12-1)) is given in 12.2.3. The equation is based on the expression for development length endorsed in references: 12.2, 12.3. In Eq. (12-1), c 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 the bars or wires. k_{tr} is a factor that represents the contribution of confining reinforcement across potential splitting planes. α is the traditional reinforcement location factor to reflect the adverse effects of the top reinforcement casting position. β is a coating factor reflecting the effects of epoxy coating. There is a limit on the product $\alpha\beta$. γ is a reinforcement size factor that reflects the more favorable performance of smaller diameter reinforcement. λ is a factor reflecting the lower tensile strength of lightweight concrete and the resulting reduction of the splitting resistance, which increases the development length in lightweight concrete. A limit of 2.5 is placed on the term $(c + k_{tr})/d_b$. When $(c + k_{tr})/d_b$ is less than 2.5, splitting failures are likely to occur. For values above 2.5, a pullout failure is expected and an increase in cover or transverse reinforcement is unlikely to increase the anchorage capacity.

Equation (12-1) allows the designer to see the effect of all variables controlling the development length. The designer is permitted to disregard terms when such omission results in longer and hence, more conservative, development lengths.

The provisions of 12.2.2 and 12.2.3 give a two-tier approach. The user can either calculate ℓ_d based on the actual $(c + k_{tr})/d_b$ (12.2.3) or calculate ℓ_d using 12.2.2, which is based on two preselected values of $(c + k_{tr})/d_b$.

Section 12.2.2 recognizes that many current practical construction cases utilize spacing and cover values along with confining reinforcement, such as stirrups or ties, that result in a value of $(c + k_{tr})/d_b$ of at least 1.5. Examples include a minimum clear cover of d_b along with either minimum clear spacing of $2d_b$, or a combination of minimum clear spacing of d_b and minimum ties or stirrups. For these frequently occurring cases, the development length for larger bars can be taken as $\ell_d = [(3/5 f_y \alpha\beta\lambda \;/ \sqrt{f_c'})] d_b$. For Dia 20 mm deformed bars and smaller, as well as for deformed wire, the development lengths could be reduced 20% using γ = 0.80. This is the basis for the middle column of the table in 12.2.2. With less cover and in the absence of minimum ties or stirrups, the minimum clear spacing limits of 7.6.1 and the minimum concrete cover requirements of 7.7 result in minimum values of c of d_b. Thus, for "other cases," the values are based on using $(c + k_{tr})/d_b$ = 1.0 in Eq. (12-1).

The user may easily construct simple, useful expressions. For example, in all structures with normal weight concrete (λ = 1.0), uncoated reinforcement (β = 1.0), Dia 22 mm or larger bottom bars (α = 1.0) with f_c' = 30 MPa and Grade 420 reinforcement, the equations reduce to.

$$\ell_d = \frac{(3)(420)\,(1.0)(1.0)(1.0)}{5\,\sqrt{30}}\,d_b = 46\,d_b$$

or

$$\ell_d = \frac{9(420)(1.0)(1.0)(1.0)}{10\,\sqrt{30}}\,d_b = 69\,d_b$$

Thus, as long as minimum cover of d_b is provided along with a minimum clear spacing of $2d_b$, or a minimum clear cover of d_b and a minimum clear spacing of d_b are provided along with minimum ties or stirrups, a designer knows that $\ell_d = 46 d_b$. The penalty for spacing bars closer or providing less cover is the requirement that $\ell_d = 69 d_b$.

Many practical combinations of side cover, clear cover, and confining reinforcement can be used with 12.2.3 to produce significantly-shorter, development lengths than allowed by 12.2.2. For example, bars or wires with minimum clear cover not less than $2d_b$ and minimum clear spacing not less than $4d_b$ and without any confining reinforcement would have $a\,(c + k_{tr})/d_b$ value of 2.5 and would require a development length of only $28d_b$ for the example above.

R12.2.4 The reinforcement location factor α accounts for position of the reinforcement in freshly placed concrete. Section 12.2.4 allows a lower value to be used for factor λ when the splitting tensile strength of the lightweight concrete is specified. See 5.1.4.

Studies[12.3, 12.4, 12.5] of the anchorage of epoxy-coated bars show that bond strength is reduced because the coating prevents adhesion and friction between the bar and the concrete. The factors reflect the type of anchorage failure likely to occur. When the cover or spacing is small, a splitting failure can occur and the anchorage or bond strength is substantially reduced. If the cover and spacing between bars is large, a splitting failure is precluded and the effect of the epoxy coating on anchorage strength is not as large. Studies [12.6] have shown that although the cover or spacing may be small, the anchorage strength may be increased by adding transverse steel crossing the plane of splitting, and restraining the splitting crack.

Because the bond of epoxy-coated bars is already reduced due to the loss of adhesion between the bar and the concrete, an upper limit of 1.7 is established for the product of the top reinforcement and epoxy-coated reinforcement factors.

Although there is no requirement for transverse reinforcement along the tension development or splice length, recent research [12.7, 12.8] indicates that in concrete with very high compressive strength, brittle anchorage failure occurred in bars with inadequate transverse reinforcement. In splice tests of Dia 25 and Dia 36 mm bars in concrete with an $f_c^{'}$ of approximately 100 MPa, transverse reinforcement improved ductile anchorage behavior.

R12.2.5 **Excess reinforcement.** The reduction factor based on area is not to be used in those cases where anchorage development for full f_y , is required. For example, the excess reinforcement factor does not apply for development of positive moment reinforcement at supports according to 12.1 1.2, for development of shrinkage and temperature reinforcement according to 7.12.2.3, or for development of reinforcement provided according to 7.13 and 13.3.8.5.

SECTION R12.3
DEVELOPMENT OF DEFORMED BARS AND DEFORMED WIRE IN COMPRESSION

The weakening effect of flexural tension cracks is not present for bars and wire in compression, and usually end bearing of the bars on the concrete is beneficial. Therefore, shorter development lengths are specified for compression than for tension. The development length may be reduced 25 percent when the reinforcement is enclosed within spirals or ties. A reduction in development length is also permitted if excess reinforcement is provided.

SECTION R12.4
DEVELOPMENT OF BUNDLED BARS

R12.4.1 An increased development length for individual bars is required when three or four bars are bundled together. The extra extension is needed because the grouping makes it more difficult to mobilize bond resistance from the core between the bars.

The designer should also note 7.6.6.4 relating to the cutoff points of individual bars within a bundle and 12.14.2.2 relating to splices of bundled bars. The increases in development length of 12.4 do apply when computing splice lengths of bundled bars in accordance with 12.14.2.2. The development of bundled bars by a standard hook of the bundle is not covered by the provisions of 12.5.

R12.4.2 Although splice and development lengths of bundled bars are based on the diameter of individual bars increased by 20 or 33 percent, as appropriate, it is necessary to use an equivalent diameter of the entire bundle derived from the equivalent total area of bars when determining factors in 12.2, which considers cover and clear spacing and represents the tendency of concrete to split.

SECTION R12.5
DEVELOPMENT OF STANDARD HOOKS IN TENSION

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, which governs the magnitude of compressive stresses on the inside of the hook. Only standard hooks (see 7.1) are considered and the influence of larger radius of bend cannot be evaluated by 12.5.

The hooked bar anchorage provisions give the total hooked bar embedment length as shown in Fig. R12.5. The development length ℓ_{dh} is measured from the critical section to the outside end (or edge) of the hook.

The development length for standard hooks ℓ_{dh} of 12.5.2 can be reduced by all applicable modification factors of 12.5.3. As an example, if the conditions of both 12.5.3(a) and (c) are met, both factors may be applied.

The effects of bar yield strength, excess reinforcement, lightweight concrete, and factors to reflect the resistance to splitting provided from confinement by concrete and transverse ties or stirrups are based on recommendations from References 12.1 and 12.2.

Tests[12.9] indicate that closely spaced ties at or near the bend portion of a hooked bar are most effective in confining the hooked bar. For construction purposes, this is not always practicable. The cases where the modification factor of 12.5.3(b) may be used are illustrated in Fig. R12.5.3(a) and (b). Figure R12.5.3(a) shows placement of ties or stirrups perpendicular to the bar being developed, spaced along the development length, ℓ_{dh} of the hook. Figure R12.5.3(b) shows placement of ties or stirrups parallel to the bar being developed along the length of the tail extension of the hook plus bend. The latter configuration would be typical in a beam column joint.

The factor for excess reinforcement in 12.5.3(d) applies only where anchorage or development for full f_y is not specifically required. Unlike straight bar development, no distinction is made between top bars and other bars; such a distinction is difficult for hooked bars in any case. A minimum value of ℓ_{dh} is specified to prevent failure by direct pullout in cases where a hook may be located very near the critical section. Hooks cannot be considered effective in compression.

Tests[12.10] indicate that the development length for hooked bars should be increased by 20 percent to account for reduced bond when reinforcement is epoxy coated.

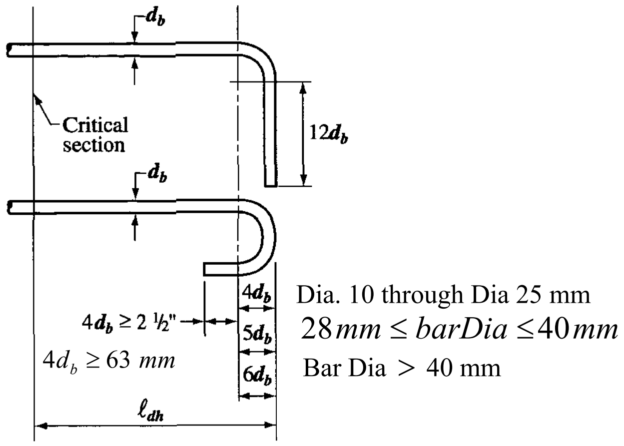


Fig. R12.5 - Hooked bar details for development of standard hooks

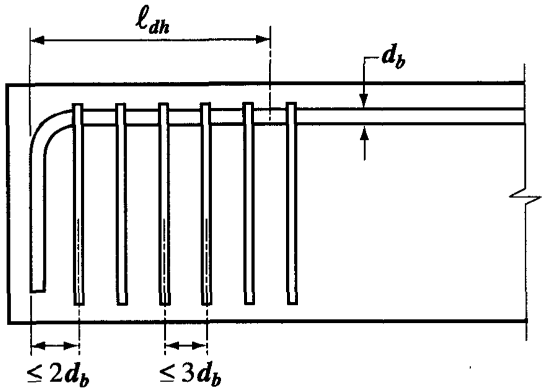


Fig. R12.5.3(a) - Ties or stirrups placed perpendicular to the bar being developed, spaced along the development length ℓ_{dh}.

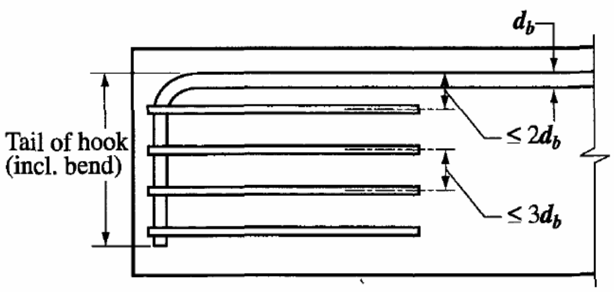


Fig. R12.5.3(b) - Ties or stirrups placed parallel to the bar being developed, spaced along the length of the tail extension of the hook plus bend.

R12.5.4 Bar hooks are especially susceptible to a concrete splitting failure if both side cover (normal to plane of hook) and top or bottom cover (in plane of hook) are small. See Fig. R12.5.4.

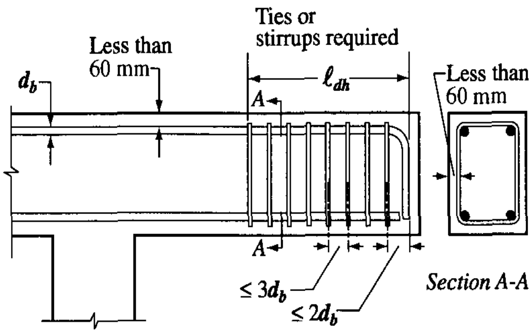


Fig. R12.5.4 - Concrete cover per 12.5.4

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. Also, provisions of 12.5.4 do not apply for hooked bars at discontinuous ends of slabs with confinement provided by the slab continuous on both sides normal to the plane of the hook.

R12.5.5 In compression, hooks are ineffective and may not be used as anchorage.

SECTION R12.6 MECHANICAL ANCHORAGE

R12.6.1 Mechanical anchorage can be made adequate for strength both for tendons and for bar reinforcement.

R12.6.3 Total development of a bar consists of the sum of all the parts that contribute to anchorage. When a mechanical anchorage is not capable of developing the required design strength of the reinforcement, additional embedment length of reinforcement should be provided between the mechanical anchorage and the critical section.

SECTION R12.7 DEVELOPMENT OF WELDED DEFORMED WIRE FABRIC IN TENSION

Fig. R12.7 shows the development requirements for deformed wire fabric with one cross wire within the development length. ASTM A 497 for deformed wire fabric requires the same strength of the weld as required for plain wire fabric (ASTM A 185). Some of the development is assigned to welds and some assigned to the length of deformed wire. The factors in 12.7.2 are applied to the deformed wire development length computed from 12.2, but with an absolute minimum of 200 mm.

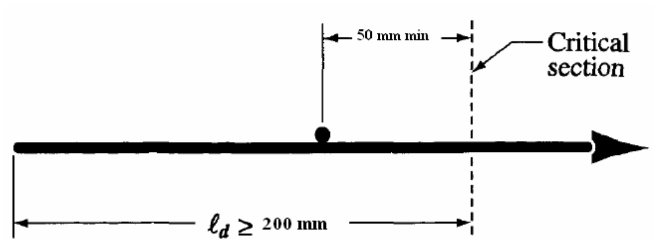


Fig. R12.7 - Development of welded deformed wire fabric

Tests[12.11] have indicated that epoxy-coated welded wire fabric has essentially the same development and splice strengths as uncoated fabric since the cross wires provide the primary anchorage for the wire. Therefore, an epoxy-coating factor of 1.0 is used for development and splice lengths of epoxy-coated welded wire fabric with cross wires within the splice or development length.

SECTION R12.8 DEVELOPMENT OF WELDED PLAIN WIRE FABRIC IN TENSION

Fig. R12.8 shows the development requirements for plain wire fabric with development primarily dependent on the location of cross wires. For fabrics made with the smaller wires, an embedment of at least two cross wires 50 mm or more beyond the point of critical section is adequate to develop the full yield strength of the anchored wires. However, for fabrics made with larger closely spaced wires, a longer embedment is required and a minimum development length is provided for these fabrics.

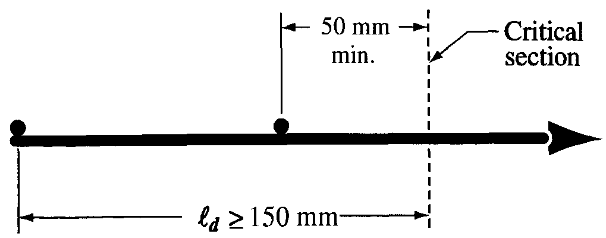


Fig. R12.8 - Development of welded plain wire fabric

SECTION R12.9 DEVELOPMENT OF PRESTRESSING STRAND

The development requirements for prestressing strand are intended to provide bond integrity for the strength of the member. The provisions are based on tests performed on normalweight concrete members with a minimum cover of 50 mm. These tests may not represent the behavior of strand in low water-cementitious materials ratio, no-slump concrete. Fabrication methods should ensure consolidation of concrete around the strand with complete contact between the steel and concrete. Extra precautions should be exercised when low water-cementitious materials ratio, no-slump concrete is used.

The first term in Eq. (12-2) represents the transfer length of the strand, that is, the distance over which the strand should be bonded to the concrete to develop the prestress f_{se} in the strand. The second term represents the additional length over which the strand should be bonded so that a stress f_{ps} may develop in the strand at nominal strength of the member.

The bond of strand is a function of a number of factors, including the configuration and surface condition of the steel, the stress in the steel, the depth of concrete beneath the strand, and the method used to transfer the force in the strand to the concrete. For bonded applications, quality assurance procedures should be used to confirm that the strand is capable of adequate bond.[12.12,12.13] The precast concrete manufacturer may rely on certification from the strand manufacturer that the strand has bond characteristics that comply with this section. Strand with a slightly rusted surface can have an appreciably shorter transfer length than clean strand. Gentle release of the strand will permit a shorter transfer length than abruptly cutting the strands.

The provisions of 12.9 do not apply to plain wires or to end-anchored tendons. The length for smooth wire could be expected to be considerably greater due to the absence of mechanical interlock. Flexural bond failure would occur with plain wire when first slip occurred.

R12.9.1.1 Figure R12.9 shows the relationship between steel stress and the distance over which the strand is bonded to the concrete represented by Eq. (12-2). This idealized variation of strand stress may be used for analyzing sections within the development region.[12.14,12.15] The expressions for transfer length, and for the

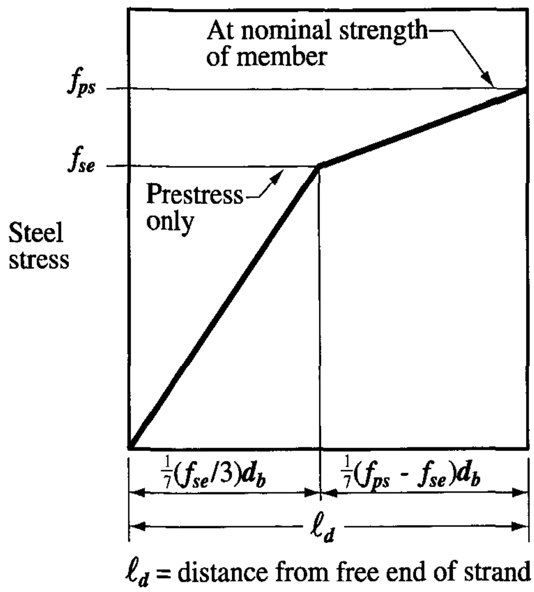


Fig. R12.9 - Idealized bilinear relationship between steel stress and distance from the free end of strand.

additional bonded length necessary to develop an increase in stress of $(f_{ps} - f_{se})$, are based on tests of members prestressed with clean, 6, 9, and 12 mm diameter strands for which the maximum value of f_{ps} was 2600 MPa. See References 12.16, 12.17, and 12.18.

R12.9.2 Where bonding of one or more strands does not extend to the end of the member, critical sections may be at locations other than where full design strength is required to be developed, and detailed analysis may be required. References 12.14

and 12.15 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.

R12.9.3 Exploratory tests (Reference: 12.16) that study the effect of debonded strand (bond not permitted to extend to the ends of members) on performance of pretensioned girders indicated that the performance of these girders with embedment lengths twice those required by 12.9.1 closely matched the flexural performance of similar pretensioned girders with strand fully bonded to ends of girders. Accordingly, doubled development length is required for strand not bonded through to the end of a member. Subsequent tests[12.19] indicated that in pretensioned members designed for zero tension in the concrete under service load conditions (see 18.4.2), the development length for debonded strands need not be doubled. For analysis of sections with debonded strands at locations where strand is not fully developed, it is usually assumed that both the transfer length and development length are doubled.

SECTION R12.10
DEVELOPMENT OF FLEXURAL REINFORCEMENT – GENERAL

R12.10.2 Critical sections for a typical continuous beam are indicated with a "c" or an "x" in Fig. R12.10.2. For uniform loading, the positive reinforcement extending into the support is more apt to be governed by the requirements of 12.11.3 rather than by development length measured from a point of maximum moment or bar cutoff.

R12.10.3 The moment diagrams customarily used in design are approximate; some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, lateral loads, or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location of the calculated tensile stress approximately a distance d towards a point of zero moment. When stirrups are provided, this effect is less severe, although still present to some extent.
To provide for shifts in the location of maximum moments, the SBC 304 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. R12.10.2.
When bars of different sizes are used, cutoff points of bars to meet this requirement are illustrated in Fig. R12.10.2. The extension should be in accordance with the diameter of bar being terminated. A bar bent to the far face of a beam and continued there may logically be considered effective, in satisfying this section, to the point where the bar crosses the mid-depth of the member.

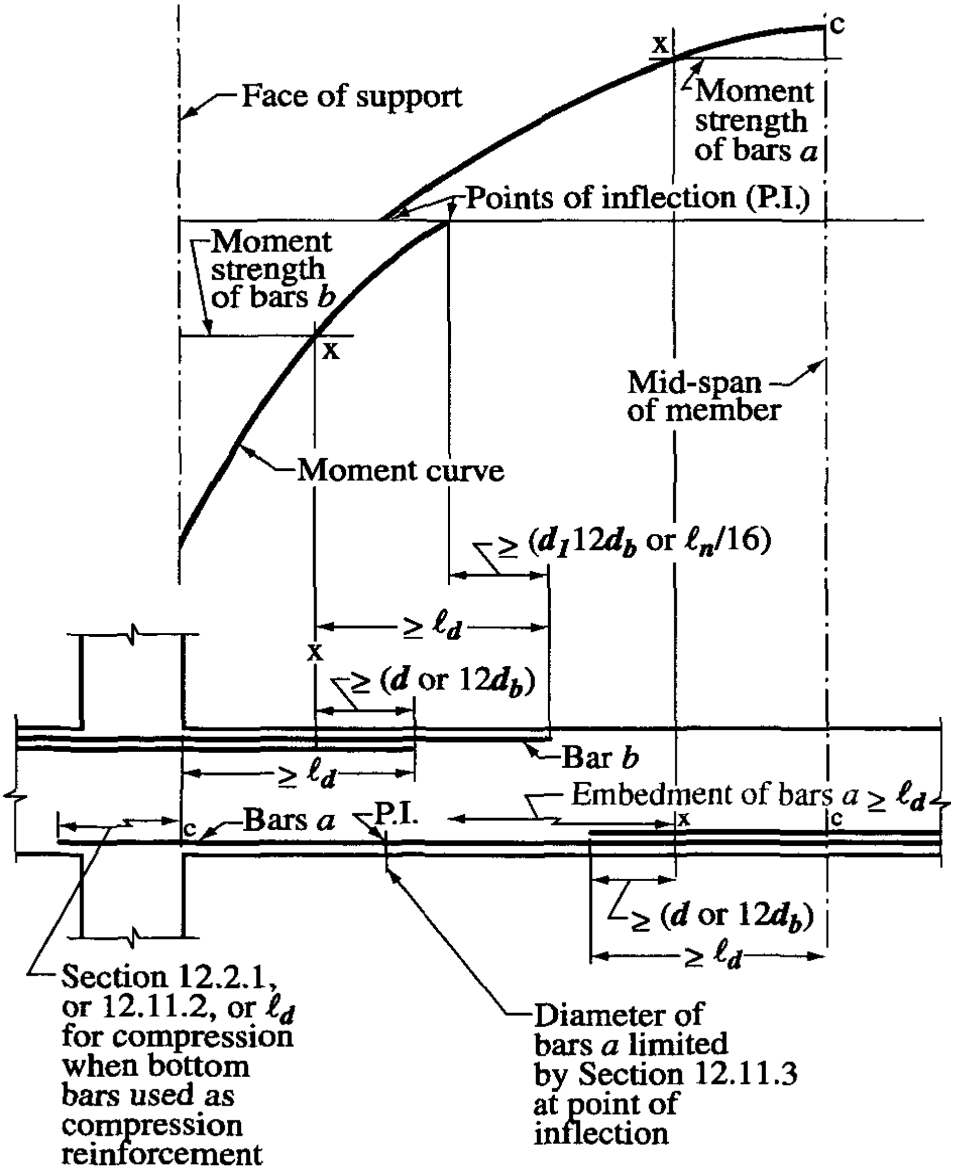


Fig. R12.10.2 - Development of flexural reinforcement in a typical continuous beam

R12.10.4 Peak stresses exist in the remaining bars wherever adjacent bars are cutoff, or bent, in tension regions. In Fig. R12.10.2 an "x" is used to indicate the peak stress points remaining in continuing bars after part of the bars have been cutoff. If bars are cutoff 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.

R12.10.5 Reduced shear strength and loss of ductility when bars are cutoff in a tension zone, as in Fig. R12.10.2, have been reported. The SBC 304 does not permit flexural reinforcement to be terminated in a tension zone unless special conditions are satisfied. Flexure cracks tend to open early wherever any reinforcement is terminated in a tension zone. If the steel stress in the continuing reinforcement and the shear strength are each near their limiting values, diagonal tension cracking tends to develop prematurely from these flexure cracks. Diagonal cracks are less likely to form where shear stress is low (see 12.10.5.1). Diagonal cracks can be restrained by closely spaced stirrups (see 12.10.5.2). A lower steel stress reduces the probability of such diagonal cracking (see 12.10.5.3). These requirements are not intended to apply to tension splices which are covered by 12.2, 12.13.5, and the related 12.15.

R12.10.6 Brackets, members of variable depth, and other members where steel stress f_s does not decrease linearly in proportion to a decreasing moment require special consideration for proper development of the flexural reinforcement. For the bracket shown in Fig. R12.10.6, the stress at ultimate in the reinforcement is almost constant at approximately f_y from the face of support to the load point. In such a case, development of the flexural reinforcement depends largely on the end anchorage provided at the loaded end. Reference 12.20 suggests a welded cross bar of equal diameter as a means of providing effective end anchorage. An end hook in the vertical plane, with the minimum diameter bend, is not totally effective because an essentially plain concrete corner will exist near loads applied close to the corner. For wide brackets (perpendicular to the plane of the figure) and loads not applied close to the corners, U-shaped bars in a horizontal plane provide effective end hooks.

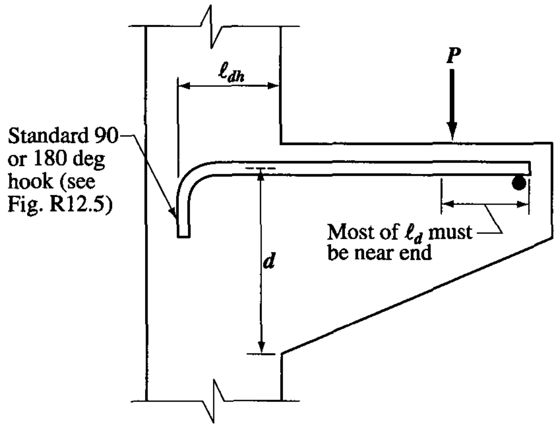


Fig. R12.10.6 - Special member largely dependent on end anchorage

SECTION R12.11
DEVELOPMENT OF POSITIVE MOMENT REINFORCEMENT

R12.11.1 Positive moment reinforcement is carried into the support to provide for some shifting of the moments due to changes in loading, settlement of supports, and lateral loads.

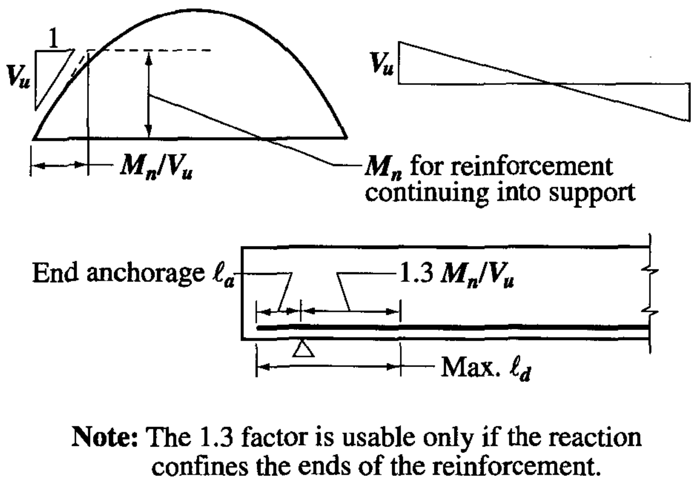
R12.11.2 When a flexural member is part of a primary lateral load resisting system, loads greater than those anticipated in design may cause reversal of moment at supports; some positive reinforcement should be well anchored into the support. This anchorage is required to ensure ductility of response in the event of serious overstress, such as from blast or earthquake. It is not sufficient to use more reinforcement at lower stresses.

R12.11.3 At simple supports and points of inflection such as "P.I." in Fig. R12.10.2, the diameter of the positive reinforcement should be small enough so that computed development length of the bar ℓ_d does not exceed $M_n / V_u + \ell_a$, or under favorable support conditions, $1.3M_n/V_u+\ell_a$. Fig.R12.11.3(a) illustrates the use of the provision.

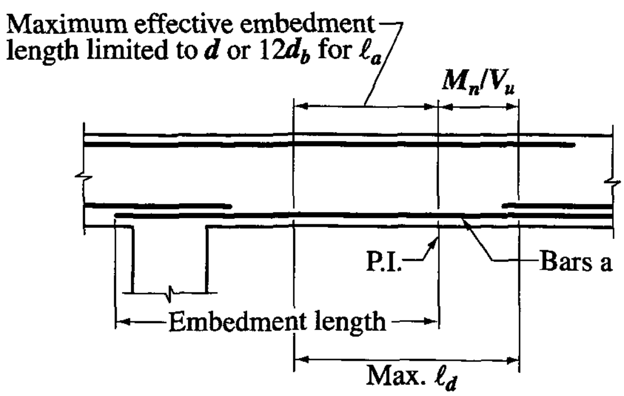
At the point of inflection the value of ℓ_a should not exceed the actual bar extension used beyond the point of zero moment. The M_n/V_u, portion of the available length is a theoretical quantity not generally associated with an obvious maximum stress point. M_n is the nominal strength of the cross section without the ϕ-factor and is not the applied factored moment.

The 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. Fig. R12.11.3(b) illustrates this provision at points of inflection. The ℓ_a limitation is added since test data are not available to show that a long end anchorage length will be fully effective in developing a bar that has only a short length between a point of inflection and a point of maximum stress.

R12.11.4 The use of the strut and tie model for the design of reinforced concrete deep flexural members clarifies that there is significant tension in the reinforcement at the face of the support. This requires the tension reinforcement to be continuous or be developed through and beyond the support.[12.21]



(a) Maximum size of bar at simple support



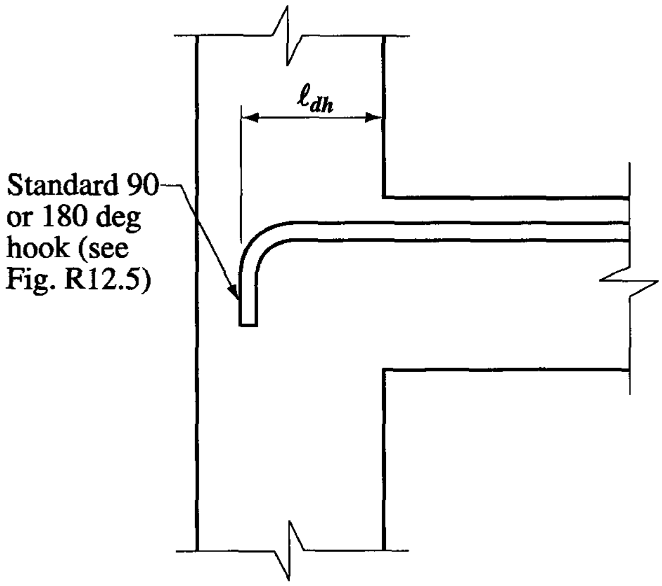
(b) Maximum size of bar "a" at point of inflection

Fig. R12.11.3 - Concept for determining maximum bar size per 12.11.3

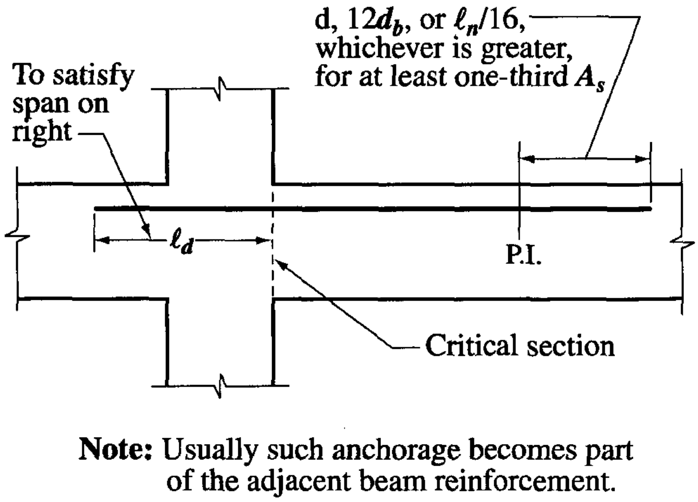
SECTION R12.12
DEVELOPMENT OF NEGATIVE MOMENT REINFORCEMENT

Fig. R12.12 illustrates two methods of satisfying requirements for anchorage of tension reinforcement beyond the face of support. For anchorage of reinforcement with hooks, see R12.5.

Section 12.12.3 provides for possible shifting of the moment diagram at a point of inflection, as discussed under R12.10.3. This requirement may exceed that of 12.10.3, and the more restrictive of the two provisions governs.



(a) Anchorage into exterior column



(b) Anchorage into adjacent beam

Fig. R12.12 - Development of negative moment reinforcement

SECTION R12.13
DEVELOPMENT OF WEB REINFORCEMENT

R12.13.1 Stirrups should be carried as close to the compression face of the member as possible because near ultimate load the flexural tension cracks penetrate deeply.

R12.13.2.1 For a Dia 16 mm bar or smaller bar, anchorage is provided by a standard stirrup hook, as defined in 7.1.3.

R12.13.2.2 Since it is not possible to bend a Dia 20, Dia 22, or Dia 25 mm stirrup tightly around a longitudinal bar and due to the force in a bar with a design stress greater than 300 MPa, 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. Since such a stirrup hook cannot fail by splitting parallel to the plane of the hooked bar, the hook strength as utilized in 12.5.2 has been adjusted to reflect cover and confinement around the stirrup hook.

For stirrups with f_y of only 300 MPa, a standard stirrup hook provides sufficient anchorage and these bars are covered in 12.13.2.1. For bars with higher strength, the embedment should be checked. A 135 deg or 180 deg hook is preferred, but a 90 deg hook may be used provided the free end of the 90 deg hook is extended the full 12 bar diameters as required in 7.1.3.

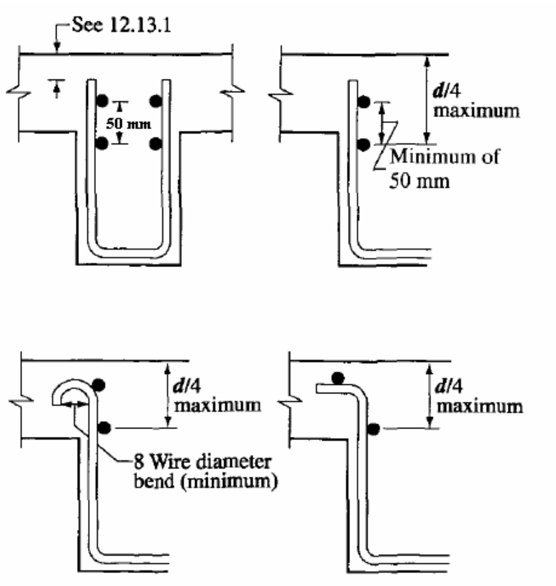


Fig. R12.13.2.3 - Anchorage in compression zone of welded plain wire fabric-U-stirrups

R12.13.2.3 The requirements for anchorage of welded plain wire fabric stirrups are illustrated in Fig. R12.13.2.3.

R12.13.2.4 Use of welded wire fabric for shear reinforcement has become commonplace in the precast, prestressed concrete industry. The rationale for acceptance of straight sheets of wire fabric as shear reinforcement is based on the findings reported in Reference 12.22.

The provisions for anchorage of single leg welded wire fabric in the tension face emphasize the location of the longitudinal wire at the same depth as the primary flexural reinforcement to avoid a splitting problem at the tension steel level. Fig. R12.13.2.4 illustrates the anchorage requirements for single leg, welded wire fabric. For anchorage of single leg, welded wire fabric, the SBC 304 has permitted hooks and embedment length in the compression and tension faces of members (see 12.13.2.1 and 12.13.2.3), and embedment only in the compression face (see 12.13.2.2). Section 12.13.2.4 provides for anchorage of straight, single leg, welded wire fabric using longitudinal wire anchorage with adequate embedment length in compression and tension faces of members.

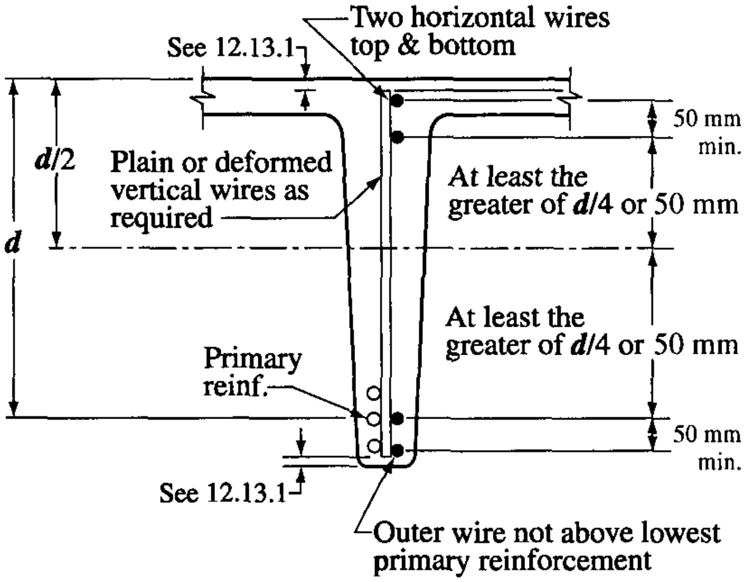


Fig. R12.13.2.4 - Anchorage of single leg welded wire fabric shear reinforcement

R12.13.2.5 In joists, a small bar or wire can be anchored by a standard hook not engaging longitudinal reinforcement, allowing a continuously bent bar to form a series of single-leg stirrups in the joist.

R12.13.5 These requirements for lapping of double U-stirrups to form closed stirrups control over the provisions of 12.15.

SECTION R12.14
SPLICES OF REINFORCEMENT - GENERAL

Splices should, if possible, be located away from points of maximum tensile stress. The lap splice requirements of 12.15 encourage this practice.

R12.14.2 Lap splices

R12.14.2.1 Because of lack of adequate experimental data on lap splices of Dia 40 and Dia 56 mm bars in compression and in tension, lap splicing of these bar sizes is prohibited except as permitted in 12.16.2 and 15.8.2.3 for compression lap.

R12.14.2.2 The increased length of lap required for bars in bundles is based on the reduction in the exposed perimeter of the bars. Only individual bars are lap spliced along the bundle.

R12.14.2.3 If individual bars in noncontact lap splices are too widely spaced, an unreinforced section is created. Forcing a potential crack to follow a zigzag line (5 to 1 slope) is considered a minimum precaution. The 150 mm maximum spacing is added because most research available on the lap splicing of deformed bars was conducted with reinforcement within this spacing.

R12.14.3 Mechanical and welded splices

R12.14.3.2 The maximum reinforcement stress used in design under the SBC 304 is the specified yield strength. To ensure sufficient strength in splices so that yielding can be achieved in a member and thus brittle failure avoided, the 25 percent increase above the specified yield strength was selected as both an adequate minimum for safety and a practicable maximum for economy.

R12.14.3.3 **A** full welded splice is primarily intended for large bars (Dia 20 mm and larger) in main members. The tensile strength requirement of 125 percent of specified yield strength is intended to provide sound welding that is also adequate for compression. See the discussion on strength in R12.14.3.2.

R12.14.3.4 The use of mechanical or welded splices of less strength than 125 percent of specified yield strength is permitted if the minimum design criteria of 12.15.4 are met. Therefore, lap welds of reinforcing bars, either with or without backup material, welds to plate connections, and end-bearing splices are allowed under certain conditions.

SECTION R12.15
SPLICES OF DEFORMED BARS AND DEFORMED WIRE IN TENSION

R12.15.1 Lap splices in tension are classified as Type A or B, with length of lap a multiple of the tensile development length ℓ_d. The development length ℓ_d used to obtain lap length should be based on f_y because the splice classifications already reflect any excess reinforcement at the splice location; therefore, the factor from 12.2.5 for excess A_s should not be used. When multiple bars located in the same plane are spliced at the same section, the clear spacing is the minimum clear distance between the adjacent splices. For splices in columns with offset bars, Fig. R12.15.1(a) illustrates the clear spacing to be used. For staggered splices, the clear spacing is the minimum distance between adjacent splices [distance x in Fig. R12.15.1(b)].

R12.15.2 The tension lap splice requirements of 12.15.1 encourage the location of splices away from regions of high tensile stress to locations where the area of steel provided is at least twice that required by analysis. Table R12.15.2 presents the splice requirements in tabular form.

TABLE R12.15.2-TENSION LAP SPLICES

$\frac{A_{s,provided*}}{A_{s,required}}$	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 reinforcement provided to area of reinforcement required by analysis at splice locations.

R12.15.3 A mechanical or welded splice should develop at least 125 percent of the specified yield strength when located in regions of high tensile stress in the reinforcement. Such splices need not be staggered, although such staggering is encouraged where the area of reinforcement provided is less than twice that required by the analysis.

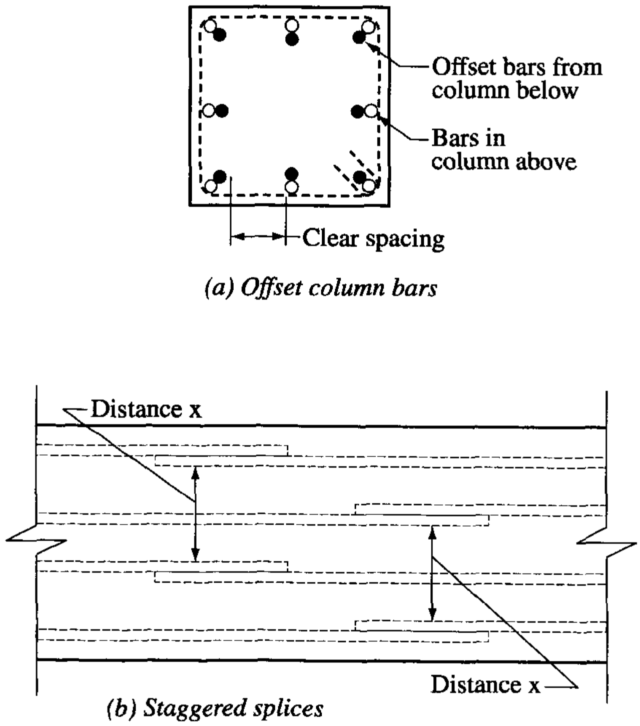


Fig. R12.15.1 - Clear spacing of spliced bars

R12.15.4 See R12.14.3.5. Section 12.15.4 concerns the situation where mechanical or welded splices of strength less than 125 percent of the specified yield strength of the reinforcement may be used. It provides a relaxation in the splice requirements where the splices are staggered and excess reinforcement area is available. The criterion of twice the computed tensile force is used to cover sections containing partial tensile splices with various percentages of total continuous steel. The usual partial tensile splice is a flare groove weld between bars or bar and structural steel piece.

To detail such welding, the length of weld should be specified. Such welds are rated at the product of total weld length times effective size of groove weld (established by bar size) times allowable stress permitted by "Structural Welding SBC 304-Reinforcing Steel" (ANSI/AWS D1.4).

A full mechanical or welded splice conforming to 12.14.3.2 or 12.14.3.4 can be used without the stagger requirement in lieu of the lower strength mechanical or welded splice.

R12.15.5 A tension tie member, has the following characteristics: member having an axial tensile force sufficient to create tension over the cross section; a level of stress in the reinforcement such that every bar must be fully effective; and limited concrete cover on all sides. Examples of members that may be classified as tension ties are arch ties, hangers carrying load to an overhead supporting structure, and main tension elements in a truss.

In determining if a member should be classified as a tension tie, consideration should be given to the importance, function, proportions, and stress conditions of the member related to the above characteristics. For example, a usual large circular tank, with many bars and with splices well staggered and widely spaced should not be classified as a tension tie member, and Class B splices may be used.

SECTION R12.16
SPLICES OF DEFORMED BARS IN COMPRESSION

Bond research has been primarily related to bars in tension. Bond behavior of compression bars is not complicated by the problem of transverse tension cracking and thus compression splices do not require provisions as strict as those specified for tension splices.

R12.16.1 Tests (References: 12.20, 12.23) 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 420 MPa, compression lap lengths are significantly increased, except where spiral enclosures are used (as in spiral columns) where increase is about 10 percent for an increase in yield strength from 420 to 520 MPa.

R12.16.2 The lap splice length is to be computed based on the larger of the compression splice length of the smaller bar; or the compression development length of the larger bar. Lap splices are generally prohibited for Dia 40 mm and larger bars; however, for compression only, lap splices are permitted for Dia 40 mm and larger bars to Dia 36 mm or smaller bars.

R12.16.4 End-bearing splices

R12.16.4.1 Experience with end-bearing splices has been almost exclusively with vertical bars in columns. If bars are significantly inclined from the vertical, special attention is required to ensure that adequate end-bearing contact can be achieved and maintained.

SECTION R12.17
SPECIAL SPLICE REQUIREMENTS FOR COLUMNS

In columns subject to flexure and axial loads, tension stresses may occur on one face of the column for moderate and large eccentricities as shown in Fig. R12.17. When such tensions occur, 12.17 requires tension splices to be used or an adequate tensile resistance to be provided. Furthermore, a minimum tension capacity is required in each face of all columns even where analysis indicates compression only.

The 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.

R12.17.2 Lap splices in columns

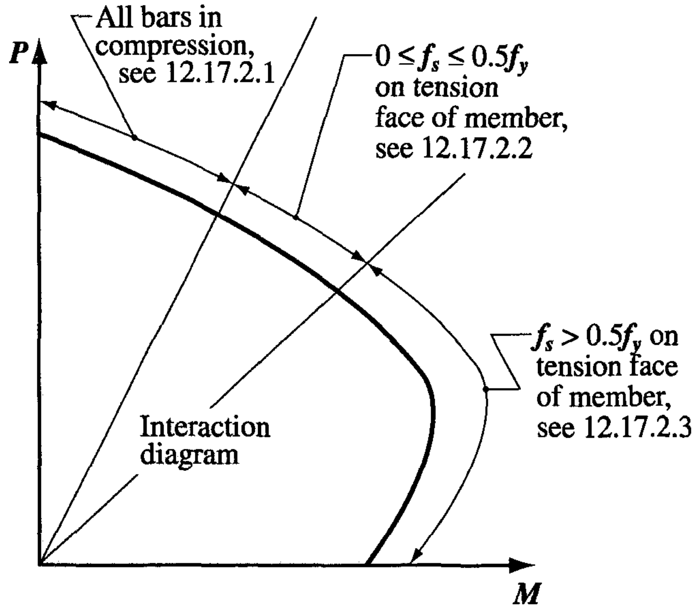


Fig. R12.17 - Special splice requirements for columns

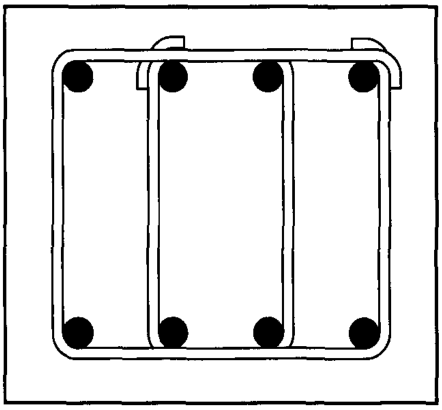


Fig. R.12.17.2 - Tie legs which cross the axis of bending are used to compute effective area. In the case shown, four legs are effective

R12.17.2.4 Reduced lap lengths are allowed when the splice is enclosed throughout its length by minimum ties.

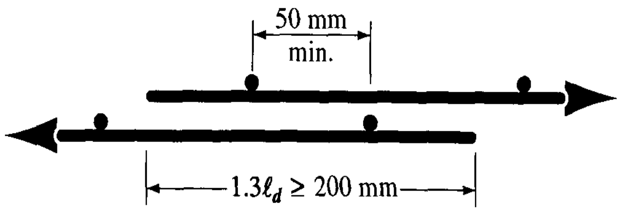
The tie legs perpendicular to each direction are computed separately and the requirement must be satisfied in each direction. This is illustrated in Fig. R12.17.2, where four legs are effective in one direction and two legs in the other direction. This calculation is critical in one direction, which normally can be determined by inspection.

R12.17.2.5 Compression lap lengths may be reduced when the lap splice is enclosed throughout its length by spirals because of increased splitting resistance. Spirals should meet requirements of 7.10.4 and 10.9.3.

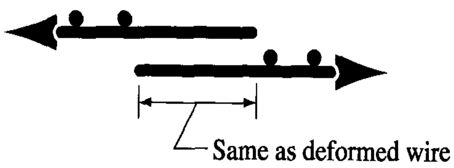
R12.17.3 **Mechanical or welded splices in columns.** Mechanical or welded splices are allowed for splices in columns but should be designed as a full mechanical splice or a full welded splice developing 125 percent f_y as required by 12.14.3.2 or

12.14.3.4. Splice capacity is traditionally tested in tension and full strength is required to reflect the high compression loads possible in column reinforcement due to creep effects. If a mechanical splice developing less than a full mechanical splice is used, then the splice is required to conform to all requirements of end-bearing splices of 12.16.4 and 12.17.4.

R12.17.4 **End-bearing splices in columns.** End-bearing splices used to splice column bars always in compression should have a tension capacity of 25 percent of the yield strength of the steel area on each face of the column, either by staggering the end-bearing splices or by adding additional steel through the splice location. The end-bearing splice should conform to 12.16.4.



(a) Section 12.18.1



(b) Section 12.18.2

Fig. R12.18 - Lap splices of deformed fabric

SECTION R12.18
SPLICES OF WELDED DEFORMED WIRE FABRIC IN TENSION

Splice provisions for deformed fabric are based on available tests.[12.24] The development length ℓ_d is that computed in accordance with the provisions of 12.7 without regard to the 200 mm minimum. The 200 mm applies to the overall splice length. See Fig. R12.18. If no cross wires are within the lap length, the provisions for deformed wire apply.

SECTION R12.19
SPLICES OF WELDED PLAIN WIRE FABRIC IN TENSION

The strength of lap splices of welded plain wire fabric is dependent primarily on the anchorage obtained from the cross wires rather than on the length of wire in the splice. For this reason, the lap is specified in terms of overlap of cross wires rather than in wire diameters or millimeters. The 50 mm additional lap required is to assure overlapping of the cross wires and to provide space for satisfactory consolidation of the concrete between cross wires. Research[12.25] has shown an increased splice length is required when fabric of large, closely spaced wires is lapped and as a consequence additional splice length requirements are provided for these fabrics, in addition to an absolute minimum of 150 mm. The development length ℓ_d is that computed in accordance with the provisions of 12.8 without regard to the 150 mm minimum. Splice requirements are illustrated in Fig. R12.19.

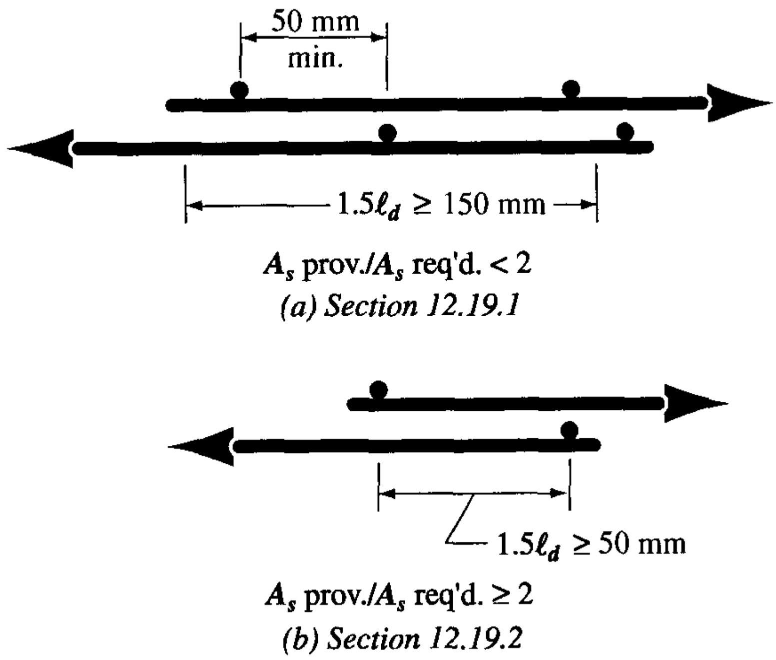


Fig. R12.19 - Lap splices of plain fabric

CHAPTER 13
TWO-WAY SLAB SYSTEM

SECTION R13.0
NOTATION

The design methods given in Chapter 13 are based on analysis of the results of an extensive series of test [13.1-13.7] and the well established performance record of various slab systems. Much of Chapter 13 is concerned with the selection and distribution of flexural reinforcement. The designer is cautioned that the problem related to safety of a slab system is the transmission of load from the slab to the columns by flexure, torsion, and shear. Design criteria for shear and torsion in slabs are given in Chapter 11.

Design aids for use in the engineering analysis and design of two-way slab systems are given in the ACI Design Handbook[13.8]. Design aids are provided to simplify application of the direct design and equivalent frame methods of Chapter 13.

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

SECTION R13.1
SCOPE

The fundamental design principles contained in Chapter 13 are applicable to all planar structural systems subjected to transverse loads. Some of the specific design rules, as well as historical precedents, limit the types of structures to which Chapter 13 applies. General characteristics of slab systems that may be designed according to Chapter 13 are described in this section. These systems include flat slabs, flat plates, two-way slabs, and waffle slabs. Slabs with paneled ceilings are two-way wide-band beam systems.

True one-way slabs, slabs reinforced to resist flexural stresses in only one direction, are excluded. Also excluded are soil-supported slabs, such as slabs on grade that do not transmit vertical loads from other parts of the structure to the soil.

For slabs with beams, the explicit design procedures of Chapter 13 apply only when the beams are located at the edges of the panel and when the beams are supported by columns or other essentially nondeflecting supports at the comers of the panel. Two-way slabs with beams in one direction, with both slab and beams supported by girders in the other direction, may be designed under the general requirements of Chapter 13. Such designs should be based upon analysis compatible with the deflected position of the supporting beams and girders.

For slabs supported on walls, the explicit design procedures in this chapter treat the wall as a beam of infinite stiffness; therefore, each wall should support the entire length of an edge of the panel (see 13.2.3). Wall-like columns less than a full panel length can be treated as columns.

SECTION R13.2
DEFINITION

R13.2.3 **A** panel includes all flexural elements between column centerlines. Thus, the column strip includes the beam, if any.

R13.2.4 For monolithic or fully composite construction, the beams include portions of the slab as flanges. Two examples of the rule are provided in Fig. R13.2.4.

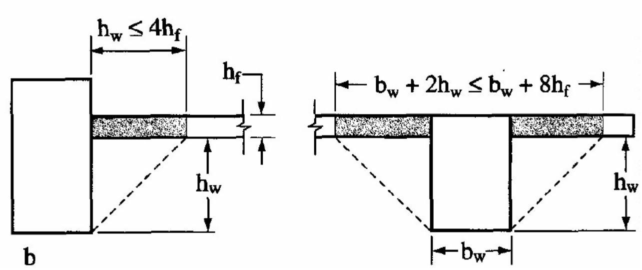


Fig. R13.2.4 - Examples of the portion of slab to be included with the beam under 13.2.4

SECTION R13.3
SLAB REINFORCEMENT

R13.3.2 The requirement that the center-to-center spacing of the reinforcement be not more than two times the slab thickness applies only to the reinforcement in solid slabs, and not to reinforcement joists or waffle slabs.

This limitation is to ensure slab action, cracking, and provide for the possibility of loads concentrated on small areas of the slab. See also R10.6.

R13.3.3-
R13.3.5 Bending moments in slabs at spandrel beams can be subject to great variation. If spandrel beams are built solidly into walls, the slab approaches complete fixity. Without an integral wall, the slab could approach simply supported, depending on the torsional rigidity ofthe spandrel beam or slab edge. These requirements provide for unknown conditions that might normally occur in a structure.

R13.3.8 **Details of reinforcement in slabs without beams**

R13.3.8.4 Bent bars are not recommended because they are difficult to place properly, however, they are permitted if they comply with 13.3.8.3.

For moments resulting from combined lateral and gravity loadings, the minimum lengths and extensions of bars in Fig. 13.3.8 may not be sufficient.

R13.3.8.5 The continuous column strip bottom reinforcement provides the slab some residual ability to span to the adjacent supports should a single support be damaged. The two continuous column strip bottom bars or wires through the column may be termed integrity steel, and are provided to give the slab some residual capacity following a single punching shear failure at a single support.[13.9]

R13.3.8.6 This provision is used 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.

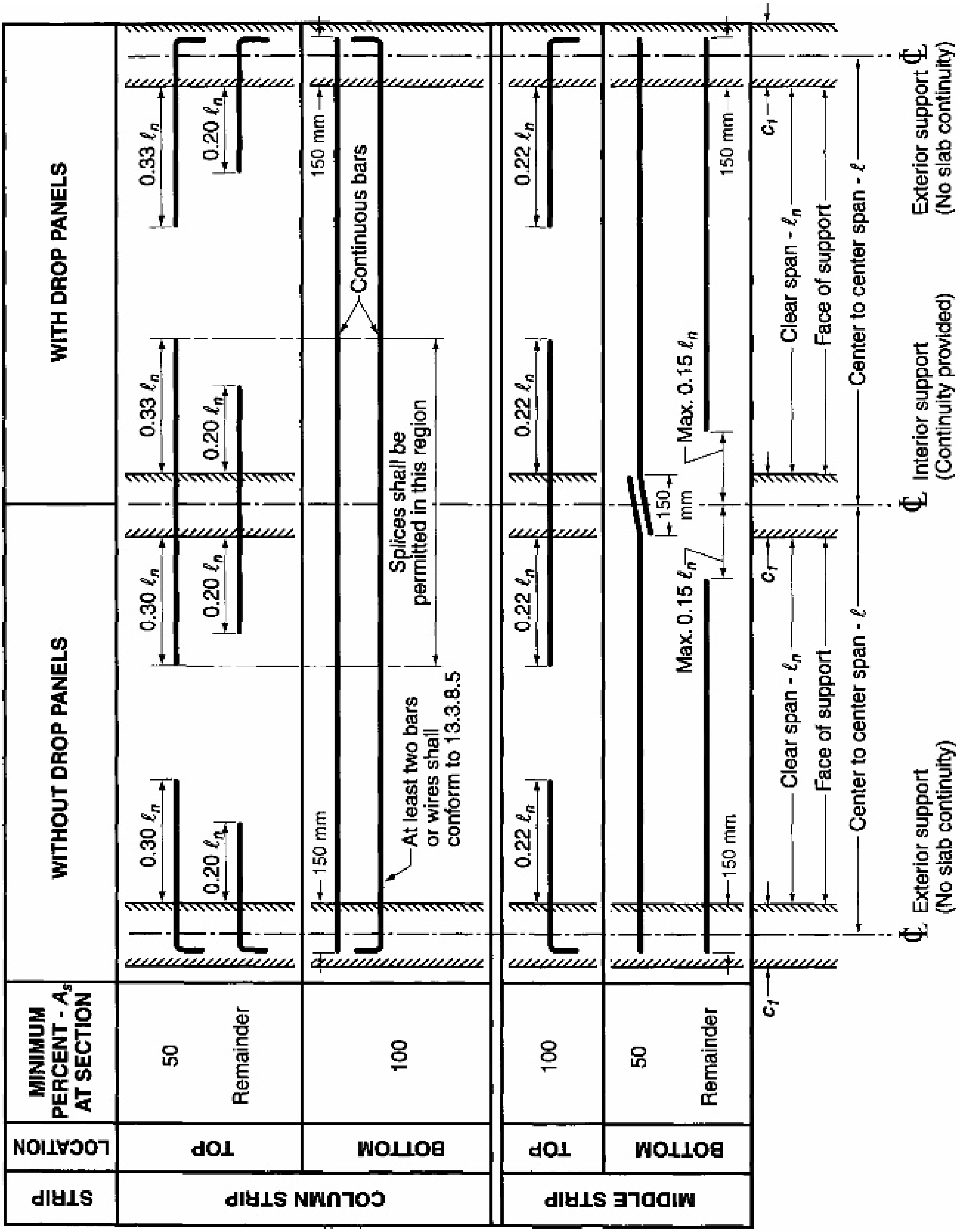


Fig. 13.3.8 - Minimum extensions for reinforcement in slabs without beams.
(See 12.11.1 for reinforcement extension into supports)

SECTION R13.4
OPENINGS IN SLAB SYSTEMS

See R11.12.5.

SECTION R13.5
DESIGN PROCEDURES

R13.5.1 This section permits a designer to base a design directly on fundamental principles of structural mechanics, provided it can be demonstrated explicitly that all safety and serviceability criteria are satisfied. The design of the slab may be achieved through the combined use of classic solutions based on a linearly elastic continuum, numerical solutions based on discrete elements, or yield-line analyses, including, in all cases, evaluation of the stress conditions around the supports in relation to shear and torsion as well as flexure. The designer should consider that the design of a slab system involves more than its analysis, and justify any deviations in physical dimensions of the slab from common practice on the basis of knowledge of the expected loads and the reliability of the calculated stresses and deformations of the structure.

R13.5.1.1 For gravity load analysis of two-way slab systems, two analysis methods are given in 13.6 and 13.7. The specific provisions of both design methods are limited in application to orthogonal frames subject to gravity loads only. Both methods apply to two-way slabs with beams as well as to flat slabs and flat plates. In both methods, the distribution of moments to the critical sections of the slab reflects the effects of reduced stiffness of elements due to cracking and support geometry.

R13.5.1.2 During the life of a structure, construction loads, ordinary occupancy loads, anticipated overloads, and volume changes will cause cracking of slabs. Cracking reduces stiffness of slab members, and increases lateral flexibility when lateral loads act on the structure. Cracking of slabs should be considered in stiffness assumptions so that drift caused by wind or earthquake is not grossly underestimated.

The designer may model the structure for lateral load analysis using any approach that is shown to satisfy equilibrium and geometric compatibility and to be in reasonable agreement with test data.[13.10,13.11] The selected approach should recognize effects of cracking as well as parameters such as ℓ_2/ℓ_1, c_1/ℓ_1, and c_2/c_1. Some of the available approaches are summarized in Reference 13.12, 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 quarter of the uncracked stiffness. For prestressed construction, 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.

R13.5.3 This section is concerned primarily with slab systems without beams. Tests and experience have shown that, unless special measures are taken to resist the torsional and shear stresses, all reinforcement resisting that part of the moment to be transferred to the column by flexure should be placed between lines that are one and one-half the slab or drop panel thickness, $1.5h$, on each side of the column. The calculated shear stresses in the slab around the column are required to conform to the requirements of 11.12.2. See R11.12.1.2 and R11.12.2.1 for more details on application of this section.

R13.5.3.3 Under certain conditions the designer is permitted to adjust the level of moment transferred by shear without revising member sizes. Tests indicate that some flexibility in distribution of unbalanced moments transferred by shear and flexure at both exterior and interior supports is possible. Interior, exterior, and corner supports refer to slab-column connections for which the critical perimeter for rectangular columns has 4, 3, or 2 sides, respectively.

At exterior supports, for unbalanced moments about an axis parallel to the edge, the portion of moment transferred by eccentricity of shear $\gamma_v M_u$ may be reduced provided that the factored shear at the support (excluding the shear produced by moment transfer) does not exceed 75 percent of the shear capacity ϕV_c as defined in 11.12.2.1 for edge columns or 50 percent for comer columns. Test[13.14,13.15] indicate that there is no significant interaction between shear and unbalanced moment at the exterior support in such cases. Note that as $\gamma_v M_u$ is decreased $\gamma_f M_u$ is increased.

Tests of interior supports indicate that some flexibility in distributing unbalanced moments transferred by shear and flexure is possible, but with more severe limitations than for exterior supports. For interior supports, the unbalanced moment transferred by flexure is permitted to be increased up to 25 percent provided that the factored shear (excluding the shear caused by the moment transfer) at the interior supports does not exceed 40 percent of the shear capacity ϕV_c as defined in 11.12.2.1.

Tests of slab-column connections indicate that a large degree of ductility is required because the interaction between shear and unbalanced moment is critical. When the factored shear is large, the column-slab joint cannot always develop all of the reinforcement provided in the effective width. The modifications for edge, corner, or interior slab-column connections in 13.5.3.3 are permitted only when the reinforcement ratio (within the effective width) required to develop the unbalanced moment $\gamma_f M_u$ does not exceed $0.375\rho_b$. The use of Eq. (13-1) without the modification permitted in 13.5.3.3 will generally indicate overstress conditions on the joint. The provisions of 13.5.3.3 are intended to improve ductile behavior of the column-slab joint. When a reversal of moments occurs at opposite faces of an interior support, both top and bottom reinforcement should be concentrated within the effective width. A ratio of top to bottom reinforcement of about 2 has been observed to be appropriate.

SECTION R13.6
DIRECT DESIGN METHOD

The direct design method consists of a set of rules for distributing moments to slab and beam sections to satisfy safety requirements and most serviceability requirements simultaneously. Three fundamental steps are involved as follows:

(1) Determination of the total factored static moment (see 13.6.2);

(2) Distribution of the total factored static moment to negative and positive sections (see 13.6.3);

(3) Distribution of the negative and positive factored moments to the column and middle strips and to the beams, if any (see 13.6.4 through 13.6.6). The distribution of moments to column and middle strips is also used in the equivalent frame method (see 13.7).

R13.6.1 **Limitations**

The direct design method was developed from consider- ations 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.

R13.6.1.1 The primary reason for the limitation in this section is the magnitude of the negative moments at the interior support in a structure with only two continuous spans. The rules given for the direct design method assume that the slab system at the first interior negative moment section is neither fixed against rotation nor discontinuous.

R13.6.1.2 If the ratio of the two spans (long spad/short span) of a panel exceeds two, the slab resists the moment in the shorter span essentially as a one-way slab.

R13.6.1.3 The limitation in this section is related to the possibility of developing negative moments beyond the point where negative moment reinforcement is terminated, as prescribed in Fig. 13.3.8.

R13.6.1.4 Columns can be offset within specified limits from a regular rectangular array. A cumulative total offset of 20 percent of the span is established as the upper limit.

R13.6.1.5 The direct design method is based on tests[13.16] for uniform gravity loads and resulting column reactions determined by statics. Lateral loads such as wind or seismic require a frame analysis. Inverted foundation mats designed as two-way slabs (see 15.10) involve application of known column loads. Therefore, even where the soil reaction is assumed to be uniform, a frame analysis should be performed.

R13.6.1.6 The elastic distribution of moments will deviate significantly from those assumed in the direct design method unless the requirements for stiffness are satisfied.

R13.6.1.7 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 13.6.7.

R13.6.1.8 The designer is permitted to use the direct design method even if the structure does not fit the limitations in this section, provided it can be shown by analysis that the particular limitation does not apply to that structure. For a slab system carrying a nonmovable load (such as a water reservoir in which the load on all panels is expected to be the same), the designer need not satisfy the live load limitation of 13.6.1.5.

R13.6.2 Total factored static moment for a span

R13.6.2.2 Eq. (13-3) follows directly from Nichol's derivation[13.17] with the simplifying assumption that the reactions are concentrated along the faces of the support perpendicular to the span considered. In general, the designer will find it expedient to calculate static moments for two adjacent half panels that include a column strip with a half middle strip along each side.

R13.6.2.4 The definition of ℓ_2 in this section is for computing the moment. However, ℓ_2 used in other sections shall be measured center to center of support.

R13.6.2.5 If a supporting member does not have a rectangular cross section or if the sides of the rectangle are not parallel to the spans, it is to be treated as a square support having the same area, as illustrated in Fig. R13.6.2.5.

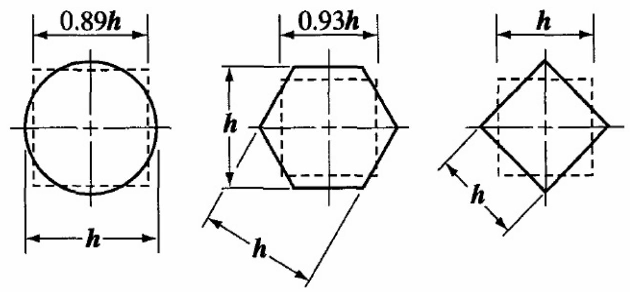


Fig. R13.6.2.5 - Examples of equivalent square section for supporting members

R13.6.3 Negative and positive factored moments

R13.6.3.3 The moment coefficients for an end span are based on the equivalent column stiffness expressions from References 13.18, 13.19, and 13.20. The coefficients for an unrestrained edge would be used, for example, if the slab were simply supported on a masonry or concrete wall.

Those for a fully restrained edge would apply if the slab were constructed integrally with a concrete wall having a flexural stiffness so large compared 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 capacity for most slab systems is governed by minimum reinforcement to control cracking. The final coefficients in the table have been adjusted so that the absolute sum of the positive and average moments equal M_o.

For two-way slab systems with beams between supports on all sides (two-way slabs), moment coefficients of column (2) 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.

R13.6.3.4 The differences in slab moment on either side of a column or other type of support should be accounted for in the design of the support. If an analysis is made to distribute unbalanced moments, flexural stiffness may be obtained on the basis of the gross concrete section of the members involved.

R13.6.3.5 Moments perpendicular to, and at the edge of, the slab structure should be transmitted to the supporting columns or walls. Torsional stresses caused by the moment assigned to the slab should be investigated.

R13.6.4, R13.6.5, and R13.6.6 Factored moments in column strips, beams, and middle strips

The rules given for assigning moments to the column strips, beams, and middle strips are based on studies[13.21] of moments in linearly elastic slabs with different beam stiffness tempered by the moment coefficients that have been used successfully.

R13.6.4.1 The negative moment factors may also be calculated as follows:

$$75 + 30 \cdot (\alpha_1 \ell_2 / \ell_1) \cdot (1 - \ell_2 / \ell_1);$$

$$\text{if } (\alpha_1 \ell_2 / \ell_1) > 1.0 \text{ use } 1.0.$$

For the purpose of establishing moments in the half column strip adjacent to an edge supported by a wall, ℓ_n in Eq. (13-3) may be assumed equal to ℓ_n of the parallel adjacent column to column span, and the wall may be considered as a beam having a moment of inertia I_b equal to infinity.

R13.6.4.2 The negative moment factors may also be calculated as follows:

$$100 - 10\beta_t + 12\beta_t (\alpha_1 \ell_2 / \ell_1) \cdot (1 - \ell_2 / \ell_1);$$

$$\text{if } \beta_t > 2.5, \text{ use } 2.5 \text{ if } (\alpha \ell_1 / \ell_2) > 1.0 \text{ use } 1.0.$$

The effect of the torsional stiffness parameter β_t is to assign all of the exterior negative factored moment to the column strip, and none to the middle strip, unless the beam torsional stiffness is high relative to the flexural stiffness of the supported slab. In the definition of β_t, the shear modulus has been taken as $E_{cb}/2$.

Where walls are used as supports along column lines, they can be regarded as very stiff beams with an $(\alpha_1 \ell_2 / \ell_1)$ value greater than one. Where the exterior support consists of a wall perpendicular to the direction in which moments are being determined, β_t may be taken as zero if the wall is of masonry without torsional resistance, and β_t may be taken as 2.5 for a concrete wall with great torsional resistance that is monolithic with the slab.

R13.6.4.4 The positive moment factors may also be calculated as follows:

$$60 + 30(\alpha_1 \ell_2 / \ell_1)(1.5 - \ell_2 / \ell_1);$$

$$\text{if } (\alpha_1 \ell_2 / \ell_1) > 1.0, \text{ use } 1.0.$$

R13.6.5 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 of these loads are normally included with w_u in Eq. (13-3). Linear loads applied directly to beams include partition walls over or along beam center lines and additional dead load of the projecting beam stem. Concentrated loads include posts above or hangers below the beams. For the purpose of assigning directly applied loads, only loads located within the width of the beam stem should be considered as directly applied to the beams. (The effective width of a beam as defined in 13.2.4 is solely for strength and relative stiffness calculations.) Line loads and concentrated loads located on the slab away from the beam stem require special consideration to determine their apportionment to slab and beams.

R13.6.8 Factored shear in slab systems with beams

The tributary area for computing shear on an interior beam is shown shaded in Fig. R13.6.8. If the stiffness for the beam $(\alpha_1 \ell_2 / \ell_1)$ is less than 1.0, the shear on the beam may be obtained by linear interpolation. In such cases, the beams framing into the column will not account for all of the shear force applied on the column. The remaining shear force will produce shear stresses in the slab around the column that should be checked in the same manner as for flat slabs, as required by 13.6.8.4. Sections 13.6.8.1 through 13.6.8.3 do not apply to the calculation of torsional moments on the beams. These moments should be based on the calculated flexural moments acting on the sides of the beam.

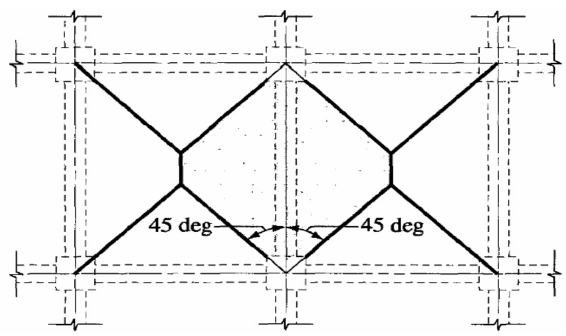


Fig. R13.6.8 - Tributary area for shear on an interior beam

R13.6.9 Factored moments in columns and walls

Eq. (13-4) 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.

Design and detailing of the reinforcement transferring the moment from the slab to the edge column is critical to both the performance and the safety of flat slabs or flat plates without edge beams or cantilever slabs. It is important that complete design details be shown on design drawings, such as concentration of reinforcement over the column by closer spacing or additional reinforcement.

SECTION R13.7
EQUIVALENT FRAME METHOD

The equivalent frame method involves the representation of the three-dimensional slab system by a series of two-dimensional frames that are then analyzed for loads acting in the plane of the frames. The negative and positive moments so determined at the critical design sections of the frame are distributed to the slab sections in accordance with 13.6.4 (column strips), 13.6.5 (beams), and 13.6.6 (middle strips). The equivalent frame method is based on studies reported in References 13.18, 13.19, and 13.20.

R13.7.2 Equivalent frame

Application of the equivalent frame to a regular structure is illustrated in Fig. R13.7.2. The three-dimensional building is divided into a series of two-dimensional frame bents (equivalent frames) centered on column or support centerlines with each frame extending the full height of the building. The width of each equivalent frame is bounded by the centerlines of the adjacent panels. The complete analysis of a slab system for a building consists of analyzing a series of equivalent (interior and exterior) frames spanning longitudinally and transversely through the building.

The equivalent frame comprises three parts: (1) the horizontal slab strip, including any beams spanning in the direction of the frame, (2) the columns or other vertical supporting members, extending above and below the slab, and (3) the elements of the structure that provide moment transfer between the horizontal and vertical members.

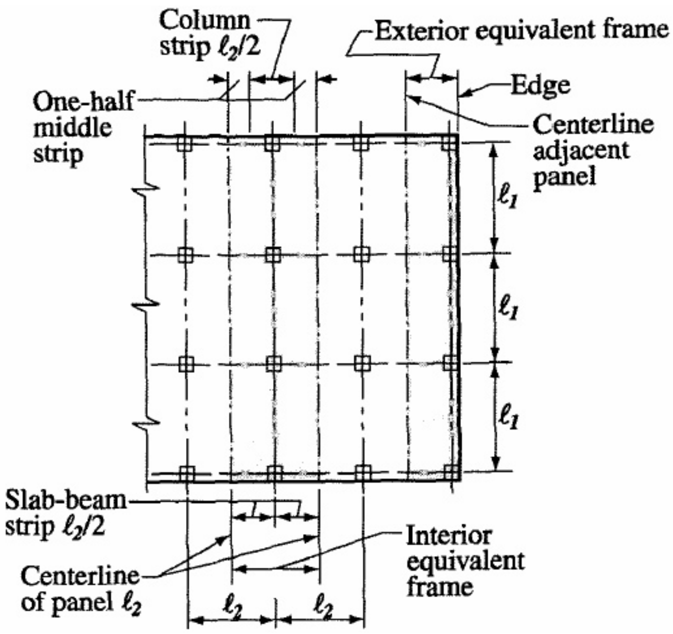


Fig. R13.7.2 - Definitions of equivalent frame

R13.7.3 Slab-beams

R13.7.3.3 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.

R13.7.4 **Columns**

Column stiffness is based on the length of the column from mid-depth of slab above to mid-depth of slab below. Column moment of inertia is computed on the basis of its cross section, taking into account the increase in stiffness provided by the capital, if any.

When slab-beams are analyzed separately for gravity loads, the concept of an equivalent column, combining the stiffness of the slab-beam and torsional member into a composite element, is used. The column flexibility is modified to account for the torsional flexibility of the slab-to-column connection that reduces its efficiency for transmission of moments. The equivalent column

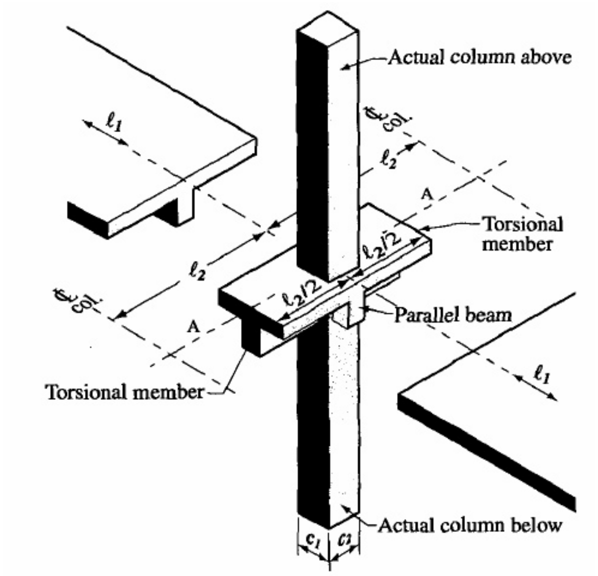


Fig. R13.7.4 - Equivalent column (column plus torsional members)

consists of the actual columns above and below the slab-beam, plus attached torsional members on each side of the columns extending to the centerline of the adjacent panels as shown in Fig. R13.7.4.

R13.7.5 **Torsional members**

Computation of the stiffness of the torsional member requires several simplifying assumptions. If no transverse-beam frames into the column, a portion of the slab equal to the width of the column or capital is assumed to be the torsional member. If a beam frames into the column, T-beam or L-beam action is assumed, with the flanges extending on each side of the beam a distance equal to the projection of the beam above or below the slab but not greater than four times the thickness of the slab. Furthermore, it is assumed that no torsional rotation occurs in the beam over the width of the support.

The member sections to be used for calculating the torsional stiffness are defined in 13.7.5.1. An approximate expression for the stiffness of the torsional member, based on the results of three-dimensional analysis of various slab configurations ((Refs 13.18, 13.19, and 13.20) is given below as:

Studies of three-dimensional analyses of various slab configurations suggest that a reasonable value of the torsional stiffness can be obtained by assuming a moment distribution along the torsional member that varies linearly from a maximum at the center of the column to zero at the middle of the panel. The assumed distribution of unit twisting moment along the column centerline is shown in Fig. R13.7.5.

An approximate expression for the stiffness of the torsional member, based on the results of three-dimensional analyses of various slab configurations (References 13.18, 13.19, and 13.20) is given below as

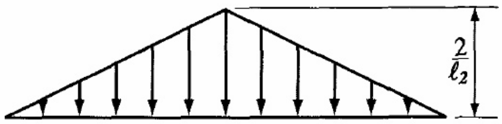


Fig. R13.7.5 - Distribution of unit twisting moment along column centerline AA shown in Fig. R13.7.4

$$K_t = \sum \frac{9E_{cs}C}{\ell_2\left(1-\dfrac{c_2}{\ell_2}\right)^3}$$

where an expression for C is given in 13.0.

R13.7.6 **Arrangement of live load**

The use of only three-quarters of the full factored live load of maximum moments is thus possible before failure occurs. This procedure, in effect, permits some local over-stress 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.

R13.7.7 **Factored moments**

R13.7.7.1-
R13.7.7.3 These SBC 304 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. Fig. R13.6.2.5 illustrates several equivalent rectangular supports for use in establishing faces of supports for design with nonrectangular supports.

R13.7.7.4 If two different methods are prescribed to obtain a particular answer, the SBC 304 should not require a value greater than the least acceptable value. Due to the long satisfactory experience with designs having total factored static moments not exceeding those given by Eq. (13-3), it is considered that these values are satisfactory for design when applicable limitations are met.

CHAPTER 14
WALLS

SECTION R14.0
NOTATION

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

SECTION R14.1
SCOPE

Chapter 14 applies generally to walls as vertical load carrying members. Cantilever retaining walls are designed according to the flexural design provisions of Chapter 10. Walls designed to resist shear forces, such as shearwalls, should be designed in accordance with Chapter 14 and 11.10 as applicable.

SECTION R14.2
GENERAL

Walls should be designed to resist all loads to which they are subjected, including eccentric axial loads and lateral forces. Design is to be carried out in accordance with 14.4 unless the wall meets the requirements of 14.5.1.

SECTION R14.3
MINIMUM REINFORCEMENT

The requirements of 14.3 apply to wall designed according to 14.4, 14.5, or 14.8. For walls resisting horizontal shear forces in the plane of the wall, reinforcement designed according to 11.10.9.2 and 11.10.9.4 may exceed the minimum reinforcement in 14.3.

SECTION R14.5
EMPIRICAL DESIGN METHOD

The empirical design method applies only to solid rectangular cross sections. All other shapes should be designed according to 14.4.

Eccentric loads and lateral forces are used to determine the total eccentricity of the factored axial load P_u. When the resultant load for all applicable load combinations falls within the middle third of the wall thickness (eccentricity not greater than $h/6$) at all sections along the length of the undeformed wall, the empirical design method may be used. The design is then carried out considering P_u as the concentric load. The factored axial load P_u should be less than or equal to the design axial load strength ϕP_{nw} computed by Eq. (14-l), $P_u \leq \phi P_{nw}$.

Eq. 14-1 reflects the general range of end conditions encountered in wall designs. Values of effective vertical length factors k are given for commonly occurring wall end conditions.

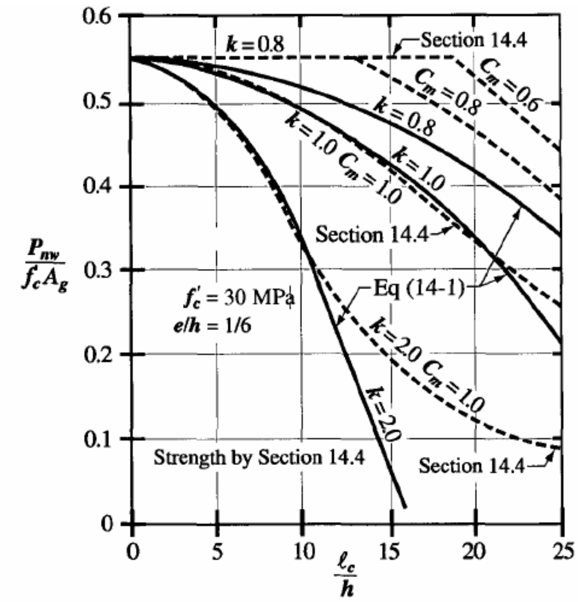


Fig. R14.5 - Empirical design of walls, Eq. (14-1) versus 14.4

The end condition "restrained against rotation" required for a k-factor of 0.8 implies attachment to a member having flexural stiffness EI/ℓ at least as large as that of the wall.The slenderness portion of Eq. (14-1) results in relatively comparable strengths by either 14.3 or 14.4 for members loaded at the middle third of the thickness with different braced and restrained end conditions. See Fig. R14.5.

The slenderness portion of Eq. (14-1) results in relatively comparable strengths by either 14.3 or 14.4 for members loaded at the middle third of the thickness with different braced and restrained end conditions. See Fig. R14.5.

R14.5.3 **Minimum thickness of walls designed by empirical design method**. The minimum thickness requirements need not be applied to walls designed according to 14.4.

SECTION R14.8
ALTERNATIVE DESIGN OF SLENDER WALLS

Section 14.8 is based on the corresponding requirements in Ref. 14.1 and experimental research.[14.2]

The procedure is presented as an alternative to the requirements of 10.10 for the out-of-plane design of precast wall panels, where the panels are restrained against overturning at the top.

Panels that have windows or other large openings are not considered to have constant cross section over the height of the panel. Such walls are to be designed taking into account the effects of openings.

Many aspects of the design of tilt-up walls and buildings are discussed in Refs. 14.3 and 14.4.

CHAPTER 15
FOOTINGS

SECTION R15.0
NOTATION

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

SECTION R15.1
SCOPE

While the provisions of Chapter 15 apply to isolated footings supporting a single column or wall, most of the provisions are generally applicable to combined footings and mats supporting several columns or walls or a combination thereof.[15.1, 15.2]

SECTION R15.2
LOADS AND REACTIONS

Footings are required to be proportioned to sustain the applied factored loads and induced reactions which 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 SBC 303. 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 such as D, L, W, and E in whatever combination that governs the design, as per SBC 301.

Only the computed end moments that exist at the base of a column (or pedestal) need to be transferred to the footing; the minimum moment requirement for slenderness considerations given in 10.12.3.2 need not be considered for transfer of forces and moments to footings.

In cases in which eccentric loads or moments are to be considered, the extreme soil pressure or pile reaction obtained from this loading should be within the permissible values. Similarly, the resultant reactions due to service loads combined with moments, shears, or both, caused by wind or earthquake loads should not exceed the increased values that may be permitted by SBC 301.

To proportion a footing or pile cap for strength, the contact soil pressure or pile reaction due to the applied factored loading (see 8.1.1) should be determined. For a single concentrically loaded spread footing, the soil reaction q_s due to the factored loading is $q_s = U / A_f$ where U is the factored concentric load to be resisted by the footing, and A_f is the base area of the footing as determined by the principles stated in 15.2.2 using the unfactored loads and the permissible soil pressure.

q_s is a calculated reaction to the factored loading used to produce the same required strength conditions regarding flexure, shear, and development of reinforcement in the footing or pile cap, as in any other member.

In the case of eccentric loading, load factors may cause eccentricities and reactions that are different from those obtained by unfactored loads.

SECTION R15.5
SHEAR IN FOOTINGS

R15.5.1 and
R15.5.2 The shear strength of footings are determined for the more severe condition of 11.12.1.1 or 11.12.1.2. The critical section for shear is measured from the face of supported member (column, pedestal, or wall), except for supported members on steel base plates.

Computation of shear requires that the soil reaction q_s be obtained from the factored loads and the design be in accordance with the appropriate equations of Chapter 11.

Where necessary, shear around individual piles may be investigated in accordance with 11.12.1.2. If shear perimeters overlap, the modified critical perimeter b_o should be taken as that portion of the smallest envelope of individual shear perimeter that will actually resist the critical shear for the group under consideration. One such situation is illustrated in Fig. R15.5.

R15.5.3 Pile caps supported on piles in more than one plane can be designed using three-dimensional strut-and-tie models satisfying Appendix A.[15.3] The effective concrete compressive strength is from A.3.2.2(b) because it is generally not feasible to provide confining reinforcement satisfying A.3.3.1 and A.3.3.2 in a pile cap.

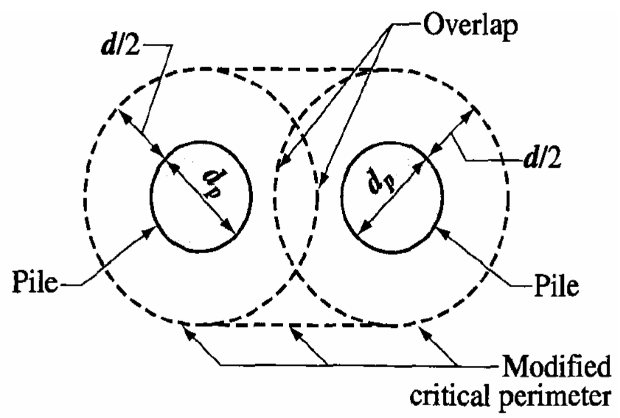


Fig. R15.5 - Modified critical perimeter for shear with overlapping critical perimeters

R15.5.4 When piles are located inside the critical sections d or $d/2$ from face of column, for one-way or two-way shear, respectively, an upper limit on the shear strength at a section adjacent to the face of the column should be considered CRSI Handbook [15.4] offers guidance for this situation.

SECTION R15.8
TRANSFER OF FORCE AT BASE OF COLUMN, WALL, OR REINFORCED PEDESTAL

Section 15.8 provides the specific requirements for force transfer from a column, wall, or pedestal (supported member) to a pedestal or footing (supporting member). Force transfer should be by bearing on concrete (compressive force only) and by reinforcement (tensile or compressive force). Reinforcement may consist of extended longitudinal bars, dowels, anchor bolts, or suitable mechanical connectors.

The requirements of 15.8.1 apply to both cast-in-place construction and precast construction. Additional requirements for cast-in-place construction are given in 15.8.2. Section 15.8.3 gives additional requirements for precast construction.

R15.8.1.1 Compressive force may be transmitted to a supporting pedestal or footing by bearing on concrete. For strength design, allowable bearing stress on the loaded area is equal to $0.85\phi f_c^{'}$, if the loaded area is equal to the area on which it is supported.

In the common case of a column bearing on a footing larger than the column, bearing strength should be checked at the base of the column and the top of the footing. Strength in the lower part of the column should be checked since the column reinforcement cannot be considered effective near the column base because the force in the reinforcement is not developed for some distance above the base, unless dowels are provided, or the column reinforcement is extended into the footing. The unit bearing stress on the column will normally be $0.85\phi f_c^{'}$. The permissible bearing strength on the footing may be increased in accordance with 10.17 and will usually be two times $0.85\phi f_c^{'}$. The compressive force that exceeds that developed by the permissible bearing strength at the base of the column or at the top of the footing should be carried by dowels or extended longitudinal bars.

R15.8.1.2 All tensile forces, whether created by uplift, moment, or other means, should be transferred to supporting pedestal or footing entirely by reinforcement or suitable mechanical connectors. Generally, mechanical connectors would be used only in precast construction.

R15.8.1.3 If computed moments are transferred from the column to the footing, the concrete in the compression zone of the column will be stressed to $0.85f_c^{'}$ under factored load conditions and, as a result, all the reinforcement will generally have to be doweled into the footing.

R15.8.1.4 The shear-friction method given in 11.7 may be used to check for transfer of lateral forces to supporting pedestal or footing. Shear keys may be used, provided that the reinforcement crossing the joint satisfies 15.8.2.1, 15.8.3.1, and the shear-friction requirements of 11.7. In precast construction, resistance to lateral forces may be provided by shear-friction, shear keys, or mechanical devices.

R15.8.2.1 and
R15.8.2.2 A minimum amount of reinforcement is required between all supported and supporting members to ensure ductile behavior. The SBC 304 does not require that all bars in a column be extended through and be anchored into a footing. However, reinforcement with an area of 0.005 times the column area or an equal area of properly spliced dowels is required to extend into the footing with proper anchorage. This reinforcement is required to provide a degree of structural integrity during the construction stage and during the life of the structure.

R15.8.2.3 Lap splices of Dia 40 mm and larger longitudinal bars in compression only to dowels from a footing are specifically permitted in 15.8.2.3. The dowel bars should be Dia 36 mm or smaller in size. The dowel lap splice length should meet the larger of the two criteria: (a) be able to transfer the bar stress in the Dia 40 mm and larger bars, and (b) fully develop the stress in the dowels as a splice.

This provision is an exception to 12.14.2.1, which prohibits lap splicing of Dia 40 mm and larger bars. This exception results from many years of successful experience with the lap splicing of these large column bars with footing dowels of the smaller size. The reason for the restriction on dowel bar size is recognition of the anchorage length problem of the large bars, and to allow use of the smaller size dowels. A similar exception is allowed for compression splices between different size bars in 12.16.2.

R15.8.3.1 and
R15.8.3.2 For cast-in-place columns, 15.8.2.1 requires a minimum area of reinforcement equal to $0.005A_g$ across the column-footing interface to provide some degree of structural been arbitrarily chosen, it is not necessary to include integrity. For precast columns this requirement is expressed in terms of an equivalent tensile force that should be transferred. Thus, across the joint, $A_s f_y = 1.5A_g$ [see 16.5.1.3(a)]. The minimum tensile strength required for precast wall-to-footing connection [see 16.5.1.3(b)] is somewhat less than that required for columns, since an overload would be distributed laterally and a sudden failure would be less likely. Since the tensile strength values of 16.5.1.3 have a strength reduction factor ϕ for these calculations.

SECTION R15.10
COMBINED FOOTINGS AND MATS

R15.10.1 Any reasonable assumption with respect to the distribution of soil pressure or pile reactions can be used as long as it is consistent with the type of structure and the properties of the soil, and conforms with established principles of soil mechanics (see 15.1). Similarly, as prescribed in 15.2.2 for isolated footings, the base area or pile arrangement of combined footings and mats should be determined using the unfactored forces, moments, or both, transmitted by the footing to the soil, considering permissible soil pressures and pile reactions.

Design methods using factored loads and strength reduction factors ϕ can be applied to combined footings or mats, regardless of the soil pressure distribution.

Detailed recommendations for design of combined footings and mats are reported by Ref. 15.1. See also Ref. 15.2.

CHAPTER 16
PRECAST CONCRETE

SECTION R16.0
NOTATION

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

SECTION R16.1
SCOPE

R16.1.1 See 2.1 for definition of precast concrete.

Design and construction requirements for precast concrete structural members differ in some respects from those for cast-in-place concrete structural members and these differences are addressed in this chapter. Where provisions for cast-in-place concrete applied to precast concrete, they have not been repeated. Similarly, items related to composite concrete in Chapter 17 and to prestressed concrete in Chapter 18 that apply to precast concrete are not restated.

More detailed recommendations concerning precast concrete are given in References 16.1 through 16.7. Tilt-up concrete construction is a form of precast concrete. It is recommended that Reference 16.8 be reviewed for tilt-up structures.

SECTION R16.2
GENERAL

R16.2.1 Stresses developed in precast members during the period from casting to final connection may be greater than the service load stresses. Handling procedures may cause undesirable deformations. 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 SBC 304 requirements.

R16.2.2 The structural behavior of precast members may differ substantially from that of similar members that are cast-in-place. Design of connections to minimize or transmit forces due to shrinkage, creep, temperature change, elastic deformation, differential settlement, wind, and earthquake require special consideration in precast construction.

R16.2.3 Design of precast members and connections is particularly sensitive to tolerances on the dimensions of individual members and on their location in the structure. To prevent misunderstanding, the tolerances used in design should be specified in the contract documents. The designer may specify the tolerance standard assumed in design. It is important to specify any deviations from accepted standards.

The tolerances required by 7.5 are considered to be a minimum acceptable standard for reinforcement in precast concrete. The designer should refer to publications of the Precast Prestressed Concrete Institute (PCI) (References 16.9, 16.10, 16.11) for guidance on industry established standard product and erection tolerances. Added guidance is given in Reference 16.12.

R16.2.4 The additional requirements may be included in either contract documents or shop drawings, depending on the assignment of responsibility for design.

SECTION R16.3
DISTRIBUTION OF FORCES AMONG MEMBERS

R16.3.1 Concentrated point and line loads can be distributed among members provided they have sufficient torsional stiffness and that shear can be transferred across joints. Torsionally stiff members such as hollow-core or solid slabs have more favorable load distribution properties than do torsionally flexible members such as double tees with thin flanges. The actual distribution of the load depends on many factors discussed in detail in References 16.13 through 16.19. Large openings can cause significant changes in distribution of forces.

R16.3.2 In-plane forces result primarily from diaphragm action in floors and roofs, causing tension or compression in the chords and shear in the body of the diaphragm. A continuous path of steel, steel reinforcement, or both, using lap splices, mechanical or welded splices, or mechanical connectors, should be provided to carry the tension, whereas the shear and compression may be carried by the net concrete section. A continuous path of steel through a connection includes bolts, weld plates, headed studs, or other steel devices. Tension forces in the connections are to be transferred to the primary reinforcement in the members.

In-plane forces in precast wall systems result primarily from diaphragm reactions and external lateral loads.

Connection details should provide for the forces and deformations due to shrinkage, creep, and thermal effects. Connection details may be selected to accommodate volume changes and rotations caused by temperature gradients and long-term deflections. When these effects are restrained, connections and members should be designed to provide adequate strength and ductility.

SECTION R16.4
MEMBER DESIGN

R16.4.1 For prestressed concrete members not wider than 4 m, 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 4 m 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.

R16.4.2 This minimum area of wall reinforcement, instead of the minimum values in 14.3, is recommended by Refs 16.4 and 16.20. The provisions for reduced minimum reinforcement and greater spacing recognize that precast wall panels have very little restraint at their edges during early stages of curing and develop less shrinkage stress than comparable cast-in-place walls.

SECTION R16.5
STRUCTURAL INTEGRITY

R16.5.1 The provisions of 7.13.3 apply to all precast concrete structures. Sections 16.5.1 and 16.5.2 give minimum requirements to satisfy 7.13.3. It is not intended that these minimum requirements override other applicable provisions of the SBC 304 for design of precast concrete structures.

The overall integrity of a structure can be substantially enhanced by minor changes in the amount, location, and detailing of member reinforcement and in the detailing of connection hardware.

R16.5.1.1 Individual members may be connected into 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.

R16.5.1.2 Diaphragms are typically provided as part of the lateral load resisting system. The ties prescribed in 16.5.1.2 are the minimum required to attach members to the floor or roof diaphragms. The tie force is equivalent to the service load value of 3.0 kN/m.

R16.5.1.3 Base connections and connections at horizontal joints in precast columns and wall panels, including shear walls, are designed to transfer all design forces and moments. The minimum tie requirements of 16.5.1.3 are not additive to these design requirements. Common practice is to place the wall ties symmetrically about the vertical centerline of the wall panel and within the outer quarters of the panel width, wherever possible.

R16.5.1.4 In the event of damage to a beam, it is important that displacement of its supporting members be minimized, so that other members will not lose their load-carrying capacity. This 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.

R16.5.2 The structural integrity minimum tie provisions for bearing wall structures, often called large panel structures, are intended to provide catenary hanger supports in case of loss of a bearing wall support, as shown by test.[16.21]

Forces induced by loading, temperature change, creep, and wind or seismic action may require a larger amount of tie force. It is intended that the general precast concrete provisions of 16.5.1 apply to bearing wall structures less than three stories in height.

Minimum ties in structures three or more stories in height, in accordance with 16.5.2.1, 16.5.2.2, 16.5.2.3, 16.5.2.4, and 16.5.2.5, are required for structural integrity (Fig. R16.5.2). These provisions are based on PCI's recommendations for design of precast concrete bearing wall buildings.[6.22] Tie capacity is based on yield strength.

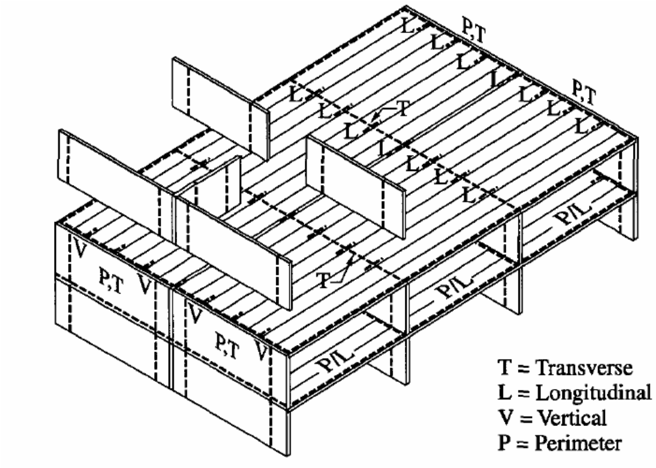


Fig. RI 6.5.2 - Typical arrangement of tensile ties in large panel structures

R16.5.2.1 Longitudinal ties may project from slabs and be lap spliced, welded, or mechanically connected, or they may be embedded in grout joints, with sufficient length and cover to develop the required force. Bond length for unstressed prestressing steel should be sufficient to develop the yield strength.[16.23] It is uncommon to have ties positioned in the walls reasonably close to the plane of the floor or roof system.

R16.5.2.3 Transverse ties may be uniformly spaced either encased in the panels or in a topping, or they may be concentrated at the transverse bearing walls.

R16.5.2.4 The perimeter tie requirements need not be additive with the longitudinal and transverse tie requirements.

SECTION R16.6
CONNECTION AND BEARING DESIGN

R16.6.1 The SBC 304 permits a variety of methods for connecting members. These are intended for transfer of forces both in-plane and perpendicular to the plane of the members.

R16.6.1.2 Various components in a connection (such as bolts, welds, plates, and inserts) have different properties that can affect the overall behavior of the connection.

R16.6.2.1 When tensile forces occur in the plane of the bearing, it may be desirable to reduce the allowable bearing stress, provide confinement reinforcement, or both. Guidelines are provided in Reference 16.4.

R16.6.2.2 This section differentiates between bearing length and length of the end of a precast member over the support (Fig. R16.6.2). Bearing pads distribute concentrated loads and reactions over the bearing area, and allow limited horizontal and rotational movements for stress relief. To prevent spalling under heavily loaded bearing areas, bearing pads should not extend to the edge of the support unless the edge is armored. Edges can be armored with anchored steel plates or angles. Section 11.9.7 gives requirements for bearing on brackets or corbels.

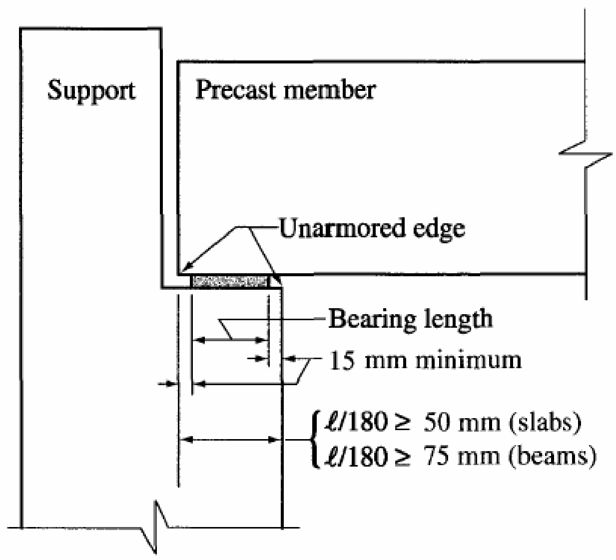


Fig. R16.6.2 - Bearing length on support

R16.6.2.3 It is unnecessary to develop positive bending moment reinforcement beyond the ends of the precast element if the system is statically determinate. Tolerances need to be considered to avoid bearing on plain concrete where reinforcement has been discontinued.

SECTION R16.7
ITEMS EMBEDDED AFTER CONCRETE PLACEMENT

R16.7.1 Section 16.7.1 is an exception to the provisions of 7.5.1. Many precast products are manufactured in such a way that it is difficult, if not impossible, to position reinforcement that protrudes from the concrete before the concrete is placed. 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.

SECTION R16.9
HANDLING

R16.9.1 The SBC 304 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 SBC 304. A precast member should not be rejected for minor cracking or spalling where strength and durability are not affected. Guidance on assessing cracks is given in PCI reports on fabrication and shipment cracks.[16.24, 16.25]

R16.9.2 All temporary erection connections, bracing, shoring as well as the sequencing of removal of these items are shown on contract or erection drawings.

SECTION R16.10
STRENGTH EVALUATION OF PRECAST CONSTRUCTION

The strength evaluation procedures of Chapter 20 are applicable to precast members.

CHAPTER 17
COMPOSITE CONCRETE FLEXURAL MEMBERS

SECTION R17.0
NOTATION

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

SECTION R17.1
SCOPE

R17.1.1 The scope of Chapter 17 is intended to include all types of composite concrete flexural members. In some cases with fully cast-in-place concrete, it may be necessary to design the interface of consecutive placements of concrete as required for composite members. Composite structural steel-concrete members are not covered in this chapter. Design provisions for such composite members are covered in Reference 17.1.

SECTION R17.2
GENERAL

R17.2.4 Tests have indicated that the strength of a composite member is the same whether or not the first element cast is shored during casting and curing of the second element.

R17.2.6 The extent of cracking is dependent on such factors as environment, aesthetics, and occupancy. In addition, composite action should not be impaired.

R17.2.7 The premature loading of precast elements can cause excessive creep and shrinkage deflections. This is especially so at early ages when the moisture content is high and the strength low.

The transfer of shear by direct bond is important if excessive deflection from slippage is to be prevented. A shear key is an added mechanical factor of safety but it does not operate until slippage occurs.

SECTION R17.3
SHORING

The provisions of 9.5.5 cover the requirements pertaining to deflections of shored and unshored members.

SECTION R17.5
HORIZONTAL SHEAR STRENGTH

R17.5.1 Full transfer of horizontal shear between segments of composite members should be ensured by honzontal shear strength at contact surfaces or properly anchored ties, or both.

R17.5.2 The nominal horizontal shear strengths V_{nh} apply when the design is based on the load factors and ϕ factors of Chapter 9.

Prestressed members used in composite construction may have variations in depth of tension reinforcement along member length due to draped or depressed tendons. Because of this variation, the definition of d used in Chapter 11 for determination of vertical shear strength is also appropriate when determining horizontal shear strength.

R17.5.2.3 The permitted horizontal shear strengths and the requirement of 5 mm amplitude for intentional roughness are based on tests discussed in References 17.2 through 17.4.

R17.5.3.1 The distribution of horizontal shear stresses along the contact surface in a composite member will reflect the distribution of shear along the member. Horizontal shear failure will initiate where the horizontal shear stress is a maximum and will spread to regions of lower stress. Because the slip at peak horizontal shear resistance is small for a concrete-to-concrete contact surface, longitudinal redistribution of horizontal shear resistance is very limited. The spacing of the ties along the contact surface should, therefore, be such as to provide horizontal shear resistance distributed approximately as the shear acting on the member is distributed.

R17.5.4 Proper anchorage of ties extending across interfaces is required to maintain contact of the interfaces.

SECTION R17.6
TIES FOR HORIZONTAL SHEAR

The minimum areas and maximum spacings are based on test data given in References 17.2 through 17.6.

CHAPTER 18
PRESTRESSED CONCRETE

SECTION R18.0
NOTATION

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

The factored prestressing force P_{su} is the product of the load factor (1.2 from Section 9.2.5) and the maximum prestressing force allowed. Under 18.5.1 this is usually overstressing to $0.94 f_{py}$ but not greater than $0.8 f_{pu}$, which is permitted for short periods of time.

$$\begin{aligned} P_{su} &= (1.2)(0.80) f_{pu} A_{ps} \\ &= 0.96 f_{pu} A_{ps} \end{aligned}$$

SECTION R18.1
SCOPE

R18.1.1 The provisions of Chapter 18 were developed primarily for structural members such as slabs, beams, and columns that are commonly used in buildings. Many of the provisions may be applied to other types of construction, such as, pressure vessels, pavements, pipes, and crossties. Application of the provisions is left to the judgment of the engineer in cases not specifically cited in the SBC 304.

R18.1.3 Some sections of the SBC 304 are excluded from use in the design of prestressed concrete for specific reasons. The following discussion provides explanation for such exclusions:

Section 7.6.5 of the SBC 304 is excluded from application to prestressed concrete because the requirements for bonded reinforcement and unbonded tendons for cast-in-place members are provided in 18.9 and 18.12, respectively.

The empirical provisions of 8.10.2, 8.10.3, and 8.10.4 for T-beams were developed for nonprestressed reinforced concrete, and if applied to prestressed concrete would exclude many standard prestressed products in satisfactory use today. Hence, proof by experience permits variations.

By excluding 8.10.2, 8.10.3, and 8.10.4, no special requirements for prestressed concrete T-beams appear in the SBC 304. Instead, the determination of an effective width of flange is left to the experience and judgment of the engineer. Where possible, the flange widths in 8.10.2, 8.10.3, and 8.10.4 should be used unless experience has proven that variations are safe and satisfactory. It is not necessarily conservative in elastic analysis and design considerations to use the maximum flange width as permitted in 8.10.2.

Sections 8.10.1 and 8.10.5 provide general requirements for T-beams that are also applicable to prestressed concrete members. The spacing limitations for slab reinforcement are based on flange thickness, which for tapered flanges can be taken as the average thickness.

In Section 8.11, the empirical limits established for nonprestressed reinforced concrete joist floors are based on successful past performance of joist construction using standard joist forming systems. See R8.11. For prestressed joist construction, experience and judgment should be used. The provisions of 8.11 may be used as a guide.

For prestressed concrete, the limitations on reinforcement given in 10.5, 10.9.1, and 10.9.2 are replaced by those in 18.8.3, 18.9, and 18.11.2.

Section 10.6 does not apply to prestressed members in its entirety. However, 10.6.4 and 10.6.7 are referenced in 18.4.4 pertaining to Class C prestressed flexural members.

In Chapter 13, the design of continuous prestressed concrete slabs requires recognition of secondary moments. Also, volume changes due to the prestressing force can create additional loads on the structure that are not adequately covered in Chapter 13. Because of these unique properties associated with prestressing, many of the design procedures of Chapter 13 are not appropriate for prestressed concrete structures and are replaced by the provisions of 18.12.

The requirements for wall design in 14.5 and 14.6 are largely empirical, utilizing considerations not intended to apply to prestressed concrete.

SECTION R18.2
GENERAL

R18.2.1 and
R18.2.2 The design investigation should include all stages that may be significant. The three major stages are: (1) jacking stage, or prestress transfer stage–when the tensile force in the prestressing steel is transferred to the concrete and stress levels may be high relative to concrete strength; (2) service load stage–after long-term volume changes have occurred; and (3) the factored load stage–when the strength of the member is checked. There may be other load stages that require investigation. For example, if the cracking load is significant, this load stage may require study, or the handling and transporting stage may be critical.

From the standpoint of satisfactory behavior, the two stages of most importance are those for service load and factored load.

Service load stage refers to the loads defined in the general building SBC 304 (without load factors), such as live load and dead load, while the factored load stage refers to loads multiplied by the appropriate load factors.

Section 18.3.2 provides assumptions that may be used for investigation at service loads and after transfer of the prestressing force.

R18.2.5 Section 18.2.5 refers to the type of post-tensioning where the prestressing steel makes intermittent contact with an oversize 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.

R18.2.6 In considering the area of the open ducts, the critical sections should include those that have coupler sheaths that may be of a larger size than the duct containing the prestressing steel. Also, in some instances, the trumpet or transition piece from the conduit to the anchorage may be of such a size as to create a critical section. If the effect of the open duct area on design is deemed negligible, section properties may be based on total area.

In post-tensioned members after grouting and in pretensioned members, section properties may be based on effective sections using transformed areas of bonded prestressing steel and nonprestressed reinforcement gross sections, or net sections.

SECTION R18.3 DESIGN ASSUMPTIONS

R18.3.3 This section defines three classes of behavior of prestressed flexural members. Class U members are assumed to behave as uncracked members. Class C members are assumed to behave as cracked members. The behavior of Class T members is assumed to be in transition between uncracked and cracked. The serviceability requirements for each class are summarized in Table R18.3.3. For comparison, Table R18.3.3 also shows corresponding requirements for nonprestressed members.

These classes apply to both bonded and unbonded prestressed flexural members, but prestressed two-way slab systems must be designed as Class U.

The precompressed tensile zone is that portion of the member cross section in which flexural tension occurs under dead and live loads. Prestressed concrete is usually designed so that the prestress force introduces compression into this zone, thus effectively reducing the magnitude of the tensile stress.

TABLE R18.3.3 — SERVICEABILITY DESIGN REQUIREMENTS

	Prestressed			Nonprestressed
	Class U	**Class T**	**Class C**	
Assumed behavior	**Uncracked**	**Transition between uncracked and cracked**	**Cracked**	**Cracked**
Section properties for stress calculation at service loads	Gross section 18.3.4	Gross section 18.3.4	Cracked section 18.3.4	No requirement
Allowable stress at transfer	18.4.1	18.4.1	18.4.1	No requirement
Allowable compressive stress based on uncracked section properties	18.4.2	18.4.2	No requirement	No requirement
Tensile stress at service loads 18.3.3	$\leq 0.7\sqrt{f_c'}$	$0.7\sqrt{f_c'} < f_t \leq \sqrt{f_c'}$	No requirement	No requirement
Deflection calculation basis	9.5.4.1 Gross section	9.5.4.2 Cracked section, bilinear	9.5.4.2 Cracked section, bilinear	9.5.2, 9.5.3 Cracked effective moment of inertia
Crack control	No requirement	No requirement	10.6.4 Modified by 18.4.4.1	10.6.4
Computation Δf_{ps} or f_s for crack control	---	---	Cracked section analysis	$M/(A_s$ *x lever arm*$)$ or $0.6 f_y$
Side skin reinforcement	No requirement	No requirement	10.6.7	10.6.7

R18.3.4 A method for computing stresses in a cracked section is given in Reference 18.1.

R18.3.5 Reference 18.2 provides information on computing deflections of cracked members.

SECTION R18.4
SERVICEABILITY REQUIREMENTS – FLEXURAL MEMBERS

Permissible stresses in concrete address serviceability. Permissible stresses do not ensure adequate structural strength, which should be checked in conformance with other SBC 304 requirements.

R18.4.1 The concrete stresses at this stage are caused by the force in the prestressing steel at transfer reduced by the losses due to elastic shortening of the concrete, relaxation of the prestressing steel, seating at transfer, and the stresses due to the weight of the member. Generally, shrinkage and creep effects are not included at this stage. These stresses apply to both pretensioned and post-tensioned concrete with proper modifications of the losses at transfer.

R18.4.l(b) and (c) The tension stress limits of $(1/4)\sqrt{f'_{ci}}$ and $(1/2)\sqrt{f'_{ci}}$ refer to tensile stress at locations other than the precompressed tensile zone. Where tensile stresses exceed the permissible values, the total force in the tensile stress zone may be calculated and reinforcement proportioned on the basis of this force at a stress of $0.6 f_y$, but not more than 200 MPa. 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.

R18.4.2(a) and (b) The compression stress limit of $0.45 f'_c$ 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.

Designs with transient live loads that are large compared to sustain live and dead loads have been penalized by the previous single compression stress limit. Therefore, the stress limit of $0.60 f'_c$, 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 the sustained live and dead loads are a large percentage of total service load, the $0.45 f'_c$ limit of 18.4.2(a) may control. On the other hand, when a large portion of the total service load consists of a transient or temporary service live load, the increased stress limit of 18.4.2(b) may apply.

The compression limit of $0.45 f'_c$ for prestress plus sustained loads will continue to control the long-term behavior of prestressed members.

R18.4.3 This section provides a mechanism whereby development of new products, materials, and techniques in prestressed concrete construction need not be inhibited by SBC 304 limits on stress. Approvals for the design should be in accordance with 1.4 of the SBC 304.

R18.4.4 For conditions of corrosive environments, defined as an environment in which chemical attack (such as seawater, corrosive industrial atmosphere, or sewer gas) is encountered, cover greater than that required by 7.7.2 should be used, and tension stresses in the concrete reduced to eliminate possible cracking at service loads. The engineer should use judgment to determine the amount of increased cover and whether reduced tension stresses are required.

R18.4.4.1 Only tension steel nearest the tension face need be considered in selecting the value of c_c used in computing spacing requirements. To account for prestressing steel, such as strand, having bond characteristics less effective than deformed reinforcement, a 2/3 effectiveness factor is used.

For post-tensioned members designed as cracked members, it will usually be advantageous to provide crack control by the use of deformed reinforcement, for which the provisions of 10.6 may be used directly. Bonded reinforcement required by other provisions of this SBC 304 may also be used as crack control reinforcement.

R18.4.4.2 It is conservative to take the decompression stress f_{dc} equal to the effective prestress f_{se}.

R18.4.4.4 The steel area of reinforcement, bonded tendons, or a combination of both may be used to satisfy this requirement.

SECTION R18.5
PERMISSIBLE STRESSES IN PRESTRESSING STEEL

The SBC 304 does not distinguish between temporary and effective prestressing steel stresses. Only one limit on prestressing steel stress is provided because the initial prestressing steel stress (immediately after transfer) can prevail for a considerable time, even after the structure has been put into service. This stress, therefore, should have an adequate safety factor under service conditions and cannot be considered as a temporary stress. Any subsequent decrease in prestressing steel stress due to losses can only improve conditions and no limit on such stress decrease is provided in the SBC 304.

R18.5.1 For higher yield strength of low-relaxation wire and strand meeting the requirements of ASTM A 421M and A 416M, it is more appropriate to specify permissible stresses in terms of specified minimum ASTM yield strength rather than specified minimum ASTM tensile strength. For the low-relaxation wire and strands, with f_{py} equal to $0.90 f_{pu}$, the $0.94 f_{py}$ and $0.82 f_{py}$ limits are equivalent to $0.85 f_{pu}$ and $0.74 f_{pu}$, respectively. The higher yield strength of the low-relaxation prestressing steel does not change the effectiveness of tendon anchorage devices; thus, the permissible stress at post-tensioning anchorage devices and couplers is not increased above the previously permitted value of $0.70 f_{pu}$. For ordinary prestressing steel (wire, strands, and bars) with f_{py} equal to $0.85 f_{pu}$ the $0.94 f_{py}$ and $0.82 f_{py}$ limits are equivalent to $0.80 f_{pu}$, and $0.70 f_{pu}$, respectively. For bar prestressing steel with f_{py} equal to $0.80 f_{pu}$, the same limits are equivalent to $0.75 f_{pu}$ and $0.66 f_{pu}$, respectively.

Designers should be concerned with setting a limit on final stress when the structure is subject to corrosive conditions or repeated loadings.

SECTION R18.6 LOSS OF PRESTRESS

R18.6.1 For an explanation of how to compute prestress losses, see Ref. 18.3 through 18.6. Reasonably accurate estimates of prestress losses can be calculated in accordance with the recommendations in Reference 18.6, which include consideration of initial stress level ($0.70 f_{pu}$ 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, since the former can result in excessive camber and horizontal movement.

R18.6.2 **Friction loss in post-tensioning tendons.** The coefficients tabulated in Table R18.6.2 give a range that generally can be expected. Due to the many types of prestressing steel ducts and sheathing available, these values can only serve as a guide. Where rigid conduit is used, the wobble coefficient K can be considered as zero. For large diameter prestressing steel in semirigid type conduit, the wobble factor can also be considered zero. Values of the coefficients to be used for the particular types of prestressing steel and particular types of ducts should be obtained from the manufacturers of the tendons. An unrealistically low evaluation of the friction loss can lead to improper camber of the member and inadequate prestress, Overestimation of the friction may result in extra prestressing force. 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 tendon stressing should be adjusted to give only that prestressing force in the critical portions of the structure required by the design.

TABLE R18.6.2-FRICTION COEFFICIENTS FOR POST-TENSIONEDTENDONS FOR USE IN EQ. (18-1) OR (18-2)

			Wobble coefficient, K	Curvature coefficient, μ
Grouted tendons in metal sheathing		Wire tendons	0.0010-0.0015	0.15-0.25
		High-strength bars	0.0001-0.0006	0.08-0.30
		7-wire strand	0.0005-0.0020	0.15-0.25
Unbonded tendons	Mastic coated	Wire tendons	0.0010-0.0020	0.05-0.15
		7-wire strand	0.0010-0.0020	0.05-0.15
	Pregreased	Wire tendons	0.0003-0.0020	0.05-0.15
		7-wire strand	0.0003-0.0020	0.05-0.15

R18.6.2.3 When the safety or serviceability of the structure may be involved, the acceptable range of prestressing steel jacking forces or other limiting requirements should either be given or approved by the structural engineer in conformance with the permissible stresses of 18.4 and 18.5.

SECTION R18.7
FLEXURAL STRENGTH

R18.7.1 Design moment strength of prestressed flexural members may be computed using strength equations similar to those for nonprestressed concrete members. 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 cross sections other than rectangular, the design moment strength ϕM_n is computed by an analysis based on stress and strain compatibility, using the stress-strain properties of the prestressing steel and the assumptions given in 10.2.

R18.7.2 Eq. (18-3) may underestimate the strength of beams with high percentages of reinforcement and, for more accurate evaluations of their strength, the strain compatibility and equilibrium method should be used. Use of Eq. (18-3) is appropriate when all of the prestressed reinforcement is in the tension zone. When part of the prestressed reinforcement is in the compression zone, a strain compatibility and equilibrium method should be used.

By inclusion of the ω' term, Eq. (18-3) reflects the increased value of f_{ps} obtained when compression reinforcement is provided in a beam with a large reinforcement index. When the term $\left[\rho_p f_{pu} / f_c' + (d/d_p)(\omega - \omega')\right]$ in Eq. (18-3) is small, the neutral axis depth is small, the compressive reinforcement does not develop its yield strength, and Eq. (18-3) becomes unconservative. This is the reason why the term $\left[\rho_p f_{pu} / f_c' + (d/d_p)(\omega - \omega')\right]$ in Eq. (18-3) may not be taken less than 0.17 if compression reinforcement is taken into account when computing f_{ps}. If the compression reinforcement is neglected when using Eq. (18-3), d' is taken as zero, then the term $\left[\rho_p f_{pu} / f_c' + (d/d_p)\omega\right]$ may be less than 0.17 and an increased and correct value of f_{ps} is obtained.

When d' is large, the strain in compression reinforcement can be considerably less than its yield strain. In such a case, the compression reinforcement does not influence f_{ps} as favorably as implied by Eq. (18-3). For this reason, the applicability of Eq. (18-3) is limited to beams in which d' is less than or equal to $0.15 d_p$.

The term $\left(\rho_p f_{pu} / f_c' + (d/d_p)(\omega - \omega')\right)$ in Eq. (18-3)

may also be written $\left(\rho_p f_{pu}/f_c' + A_s f_y / (b d_p f_c') - A_s' f_y' / (b d_p f_c')\right)$.

This form may be more convenient, such as when there is no unprestressed tension reinforcement.

Eq. (18-5) reflects results of tests on members with unbonded tendons and span-to-depth ratios greater than 35

Eq. (18.5) 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).[18.7] These tests also indicate that Eq. (18-4), overestimates the amount of stress increase in such members. Although these same tests indicate that the moment strength of those shallow members designed using Eq. (18-4) meets the factored load strength requirements, this reflects the effect of the SBC 304 requirements for minimum bonded reinforcement, as well as the limitation on concrete tensile stress that often controls the amount of prestressing force provided.

SECTION R18.8
LIMITS FOR REINFORCEMENT OF FLEXURAL MEMBERS

R18.8.1 The net tensile strain limits for compression- and tension-controlled sections given in 10.3.3 and 10.3.4 apply to prestressed sections.

The net tensile strain limits for tension-controlled sections given in 10.3.4 may also be stated in terms of ω_p. The net tensile strain limit of 0.005 corresponds to $\omega_p = 0.32\beta_1$ for pre- stressed rectangular sections.

R18.8.2 This provision is a precaution against abrupt flexural failure developing immediately after cracking. A flexural member designed according to SBC 304 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 were reached shortly after cracking, the warning deflection would not occur.

R18.8.3 Some bonded steel is required to be placed near the tension face of prestressed flexural members. The purpose of this bonded steel is to control cracking under full service loads or overloads.

SECTION R18.9
MINIMUM BONDED REINFORCEMENT

R18.9.1 Some bonded reinforcement is required by the SBC 304 in members prestressed with unbonded tendons to ensure flexural performance at ultimate member strength, rather than as a tied arch, and to limit crack width and spacing at service load when concrete tensile stresses exceed the modulus of rupture. Providing the minimum bonded reinforcement as stipulated in 18.9helps 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 21.2.1.1. The minimum bonded reinforcement areas required by Eq. (18-6) and (18-8) are absolute minimum areas independent of grade of steel or design yield strength.

R18.9.2 The minimum amount of bonded reinforcement for members other than two-way flat slab systems is based on research comparing the behavior of bonded and unbonded post-tensioned beams.[18.8] Based on this research, it is advisable to apply the provisions of 18.9.2 also to one-way slab systems.

R18.9.3 The minimum amount of bonded reinforcement in two-way flat slab systems is based on reports of References: 18.3 & 18.9.Limited research available for two-way flat slabs with drop panels 18.16 indicates that behavior of these particular systems is similar to the behavior of flat plates.

R18.9.3.1 For usual loads and span lengths, flat plate tests summarized in Reference: 18.3 indicate satisfactory performance without bonded reinforcement in the areas described in 18.9.3.1.

R18.9.3.2 In positive moment areas, where the concrete tensile stresses are between $(1/6)\sqrt{f_c'}$ and $(1/2)\sqrt{f_c'}$ a minimum bonded reinforcement area proportioned according to Eq. (18-7) is required. The tensile force N, is computed at service load on the basis of an uncracked, homogeneous section.

R18.9.3.3 Research on unbonded post-tensioned two way flat slab systems shows that bonded reinforcement in negative moment regions, 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 (References: 18.1, 18.3, 18.9, 18.10). To account for different adjacent tributary spans, Eq. (18-8) is given on the basis of the equivalent frame as defined in 13.7.2 and pictured in Fig. R13.7.2. For rectangular slab panels, Eq. (18-8) is conservatively based upon the larger of the cross-sectional areas of the two intersecting equivalent frame slab-beam strips at the column. This ensures that the minimum percentage of steel recommended by research is provided in both directions. Concentration of this reinforcement in the top of the slab directly over and immediately adjacent to the column is important. Research also shows that where low tensile stresses occur at service loads, satisfactory behavior has been achieved at factored loads without bonded reinforcement. However, the SBC 304 requires minimum bonded reinforcement regardless of service load stress levels to help ensure flexural continuity and ductility, and to limit crack widths and spacing due to overload, temperature, or shrinkage. Research on post-tensioned flat plate-to-column connections is reported in References 18.11, 18.12, 18.13, 18.14, and 18.15.

R18.9.4 Bonded reinforcement should be adequately anchored to develop factored load forces. The requirements of Chapter 12 will ensure that bonded reinforcement required for flexural strength under factored loads in accordance with 18.7.3, or for tensile stress conditions at service load in accordance with 18.9.3.2, will be adequately anchored to develop tension or compression forces. The minimum lengths apply for bonded reinforcement required by 18.9.2 or 18.9.3.3, but not required for flexural strength in accordance with 18.7.3. Research[18.1] on continuous spans shows that these minimum lengths provide adequate behavior under service load and factored load conditions.

SECTION R18.10
STATICALLY INDETERMINATE STRUCTURES

R18.10.3 For statically indeterminate structures, the moments due to reactions induced by prestressing forces, 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. However, the elastic deformations caused by a nonconcordant tendon 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 SBC 304 requires that secondary moments be included in determining design moments.

To determine the moments used in design, the order of calculation should be: (a) determine moments due to dead and live load; (b) modify by algebraic addition of secondary moments; (c) redistribute as permitted. A positive secondary moment at the support caused by a tendon transformed downward from a concordant profile will 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.

R18.10.4 **Redistribution of negative moments in continuous prestressed flexural members.** The provisions for redistribution of negative moments given in 8.4 apply equally to prestressed members. See Ref. 9.16 for a comparison of research results.

For the moment redistribution principles of 18.10.4 to be applicable to beams with unbonded tendons, it is necessary that such beams contain sufficient bonded reinforcement to ensure they will act as beams after cracking and not as a series of tied arches. The minimum bonded reinforcement requirements of 18.9 will serve this purpose.

SECTION R18.11 COMPRESSION MEMBERS-COMBINED FLEXURE AND AXIAL LOADS

R18.11.2 **Limits for reinforcement of prestressed compression members**

R18.11.2.3 The minimum amounts of reinforcement in 14.3 need not apply to prestressed concrete walls, provided the average prestress is 1.5 MPa or greater and a structural analysis is performed to show adequate strength and stability with lower amounts of reinforcement.

SECTION R18.12 SLAB SYSTEMS

R18.12.1 Use of the equivalent frame method of analysis (see 13.7) or more precise analysis procedures is required for determination of both service and factored moments and shears for prestressed slab systems. The equivalent frame method of analysis has been shown by tests of large structural models to satisfactorily predict factored moments and shears in prestressed slab systems. (See References 18.11, 18.12, 18.13, 18.17, 18.18, and 18.19). The referenced research also shows that analysis using prismatic sections or other approximations of stiffness may provide erroneous results on the unsafe side. Section 13.7.7.4 is excluded from application to prestressed slab systems because it relates to reinforced slabs designed by the direct design method, and because moment redistribution for prestressed slabs is covered in 18.10.4. Section 13.7.7.5 does not apply to prestressed slab systems because the distribution of moments between column strips and middle strips required by 13.7.7.5 is based on tests for nonprestressed concrete slabs. Simplified methods of analysis using average coefficients do not apply to prestressed concrete slab systems.

R18.12.2 Tests indicate that the moment and shear strength of prestressed slabs is controlled by total prestessing steel strength and by the amount and location of nonprestressed reinforcement, rather than by tendon distribution. (See References 18.11, 18.12, 18.13, 18.17, 18.18, and 18.19).

R18.12.3 For prestressed flat slabs continuous over two or more spans in each direction, the span-thickness ratio generally should not exceed 42 for floors and 48 for roofs; these limits may be increased to 48 and 52, respectively, if calculations verify that both short- and long-term deflection, camber, and vibration frequency and amplitude are not objectionable.

Short- and long-term deflection and camber should be computed and checked against the requirements of serviceability of the structure.

The maximum length of a slab between construction joints is generally limited to 30 to 45 m to minimize the effects of slab shortening, and to avoid excessive loss of prestress due to friction.

R18.12.4 This section provides specific guidance concerning tendon distribution that will permit the use of banded tendon distributions in one direction. This method of tendon distribution has been shown to provide satisfactory performance by structural research.

SECTION R18.13
POST-TENSIONED TENDON ANCHORAGE ZONES

R18.13.1 **Anchorage zone**. Based on the Principle of Saint-Venant, the extent of the anchorage zone may be estimated as approximately equal to the largest dimension of the cross section. Local zone and general zone are shown in Fig. R18.13.l(a). When anchorage devices located away from the end of the member are tensioned, large tensile stresses exist locally behind and ahead of the device. These tensile stresses are induced by incompatibility of deformations ahead of [as shown in Fig. R.18.13.l(b)] and behind the anchorage device. The entire shaded region should be considered, as shown in Fig. R18.13.l(b).

R18.13.2 **Local zone**. The local zone resists the very high local stresses introduced by the anchorage device and transfers them to the remainder of the anchorage zone. The behavior of the local zone is strongly influenced by the specific characteristics of the anchorage device and its confining reinforcement, and less influenced by the geometry and loading of the overall structure. Local-zone design sometimes cannot be completed until specific anchorage devices are determined at the shop drawing stage. When special anchorage devices are used, the anchorage device supplier should furnish the test information to show the device is satisfactory under AASHTO "Standard Specifications for Highway Bridges",[18.2] 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.

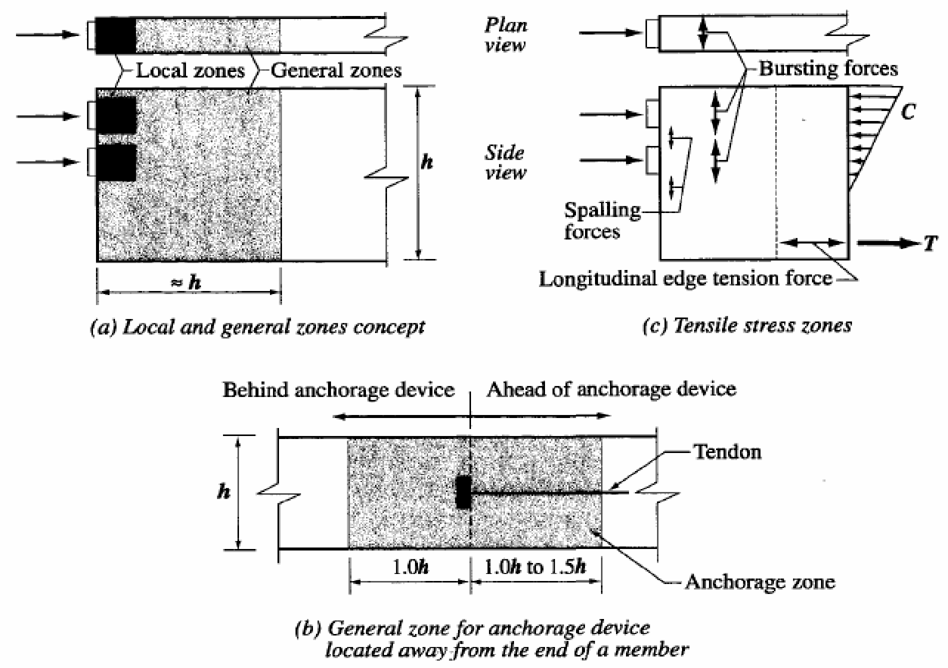


Fig.R18.13.1 - Anchorage zone

R18.13.3 **General zone.** Within the general zone the usual assumption of beam theory that plane sections remain plane is not valid.

Design should consider all regions of tensile stresses that can be caused by the tendon anchorage device, including bursting, spalling, and edge tension as shown in Fig. R18.13.l(c). Also, the compressive stresses immediately ahead [as shown in

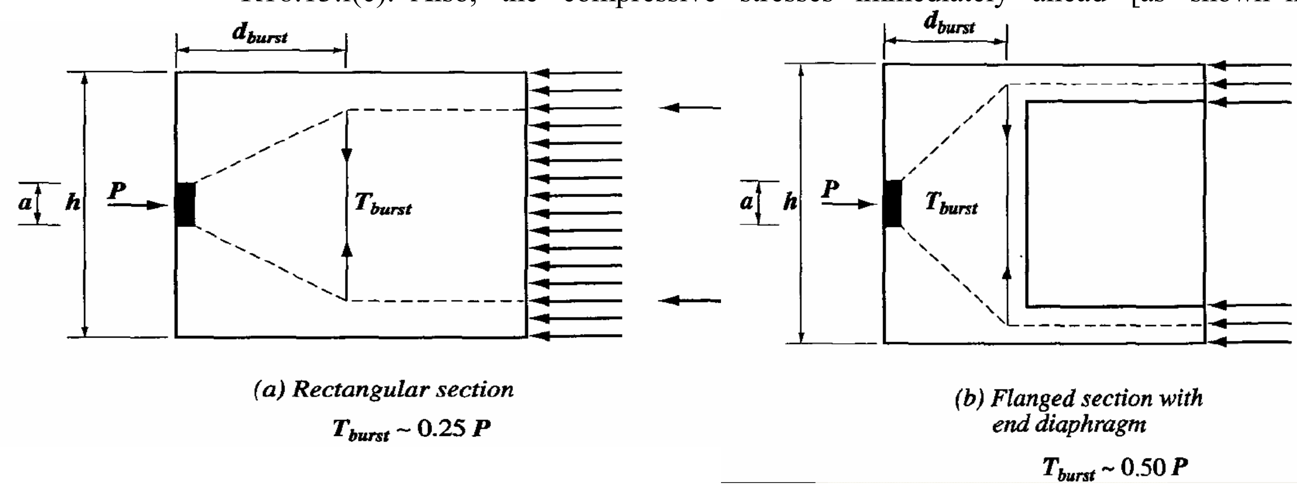


Fig.R18.13.3 - Effect of cross section change

Fig. R18.13.l(b)] of the local zone should be checked. Sometimes reinforcement requirements cannot be determined until specific tendon and anchorage device layouts are determined at the shop-drawing stage. Design and approval responsibilities should be clearly assigned in the project drawings and specifications.

Abrupt changes in section can cause substantial deviation in force paths. These deviations can greatly increase tension forces as shown in Fig. R18.13.3.

R18.13.4 **Nominal material strengths.** Some inelastic deformation of concrete is expected because anchorage zone design is based on a strength approach. The low value for the nominal compressive strength for unconfined concrete reflects this possibility. For well-confined concrete, the effective compressive strength could be increased (See Reference 18.23). The value for nominal tensile strength of bonded prestressing steel is limited to the yield strength of the prestressing steel because Eq. (18-3) may not apply to these nonflexural applications. The value for unbonded prestressing steel is based on the values of 18.7.2 (b) and (c), but is somewhat limited for these short-length, nonflexural applications. Test results given in Reference 18.23 indicate that the compressive stress introduced by auxiliary prestressing applied perpendicular to the axis of the main tendons is effective in increasing the anchorage zone capacity. The inclusion of the λ factor for lightweight concrete reflects its lower tensile strength, which is an indirect factor in limiting compressive stresses, as well as the wide scatter and brittleness exhibited in some lightweight concrete anchorage zone tests.

The designer is required to specify concrete strength at the time of stressing in the project drawings and specifications. To limit early shrinkage cracking, monostrand tendons are sometimes stressed at concrete strengths less than 20 MPa. In such cases, either oversized monostrand anchorages are used, or the strands are stressed in stages, often to levels 1/3 to 1/2 the final prestressing force.

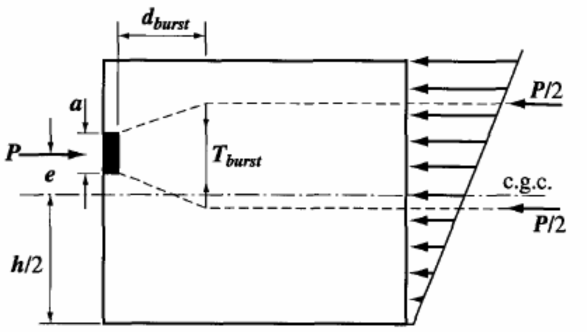


Fig. R18.13.5 - Strut-and-tie model example

R18.13.5 **Design methods.** The list of design methods in 18.13.5.1 include those procedures for which fairly specific guidelines have been given in References 18.20 and 18.21. These procedures have been shown to be conservative predictors of strength when compared to test results.[18.23] The use of strut-and-tie model is especially helpful for general zone design.[18.23] In many applications, where substantial or massive concrete regions surround the anchorages, simplified equations can be used except in the cases noted in 18.13.5.2.

For many cases, simplified equations based on References 18.20 and 18.21 can be used. Values for the magnitude of the bursting force, T_{burst}, and for its centroidal distance from the major bearing surface of the anchorage, d_{burst} may be estimated from Eq. (R18-1) and (R18-2), respectively. The terms of Eq. (R18-1) and (R18-2) are shown in Fig. R18.13.5 for a prestressing force with small eccentricity. In the applications of Eq. (R18-1) and (R18-2), the specified stressing sequence should be considered if more than one tendon is present.

$$T_{burst} = 0.25\sum P_{su}\left(1-\frac{a}{h}\right) \quad \text{(R18-1)}$$

$$d_{burst} = 0.5(h-2e) \quad \text{(R18-2)}$$

where:

$\sum P_{su}$ = the sum of the total factored prestressing force for the stressing arrangement considered, N;

a = the depth of anchorage device or single group of closely spaced devices in the direction considered, mm;

e = the eccentricity (always taken as positive) of the anchorage device or group of closely spaced devices with respect to the centroid of the cross section, mm;

h = the depth of the cross section in the direction considered, mm.

Anchorage devices should be treated as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage device in the direction considered.

The spalling force for tendons for which the centroid lies within the kern of the section may be estimated as 2 percent of the total factored prestressing force, except for multiple anchorage devices with center-to-center spacing greater than 0.4 times the depth of the section. For large spacings and for cases where the centroid of the tendons is located outside the kern, a detailed analysis is required. In addition, in the post-tensioning of thin sections, or flanged sections, or irregular sections, or when the tendons have appreciable curvature within the general zone, more general procedures such as those of AASHTO Articles 9.21.4 and 9.21.5 will be required. Detailed recommendations for design principles that apply to all design methods are given in Article 9.21.3.4 of Reference 18.20.

R18.13.5.3 The sequence of anchorage device stressing can have a significant effect on the general zone stresses. Therefore, it is important to consider not only the final stage of a stressing sequence with all tendons stressed, but also intermediate stages during construction. The most critical bursting forces caused by each of the sequentially post-tensioned tendon combinations, as well as that of the entire group of tendons, should be taken into account.

R18.13.5.4 The provision for three-dimensional effects was included to alert the designer to effects perpendicular to the main plane of the member, such as bursting forces in the thin direction of webs or slabs. In many cases these effects can be determined independently for each direction, but some applications require a fully three-dimensional analysis (for example diaphragms for the anchorage of external tendons).

R18.13.5.5 Where anchorages are located away from the end of a member, local tensile stresses are generated behind these anchorages [see Fig. R18.13.l(b)] due to compatibility requirements for deformations ahead of and behind the anchorages. Bonded tie-back reinforcement is required in the immediate vicinity of the anchorage to limit the extent of cracking behind the anchorage. The requirement $0.35P_{su}$ was developed using 25 percent of the unfactored prestressing force being resisted by reinforcement at $0.60f_y$.

SECTION R18.14
DESIGN OF ANCHORAGE ZONES FOR MONOSTRAND OR SINGLE 16 MM DIAMETER BAR TENDONS

R18.14.2 **General-zone design for slab tendons.** For monostrand slab tendons, the general-zone minimum reinforcement requirements are based on the recommendations of the ACI-ASCE Committee 423[18.22], which shows typical details. The horizontal bars parallel to the edge required by 18.14.2.2 should be continuous where possible.

The tests on which the recommendations of Ref. 18.24 were based were limited to anchorage devices for 12.5 mm diameter, 1860 MPa strand, unbonded tendons in normal-weight concrete. Thus, for larger strand anchorage devices and for all use in lightweight concrete slabs, the Ref. 18.22 recommended that the amount and spacing of reinforcement should be conservatively adjusted to provide for the larger anchorage force and smaller splitting tensile strength of lightweight concrete.[18.22]

Both Refs. 18.21 and 18.22 recommend that hairpin bars also be furnished for anchorages located within 300 mm of slab comers to resist edge tension forces. The words “ahead of" in 18.14.2.3 have the meaning shown in Fig. R18.13.1.

In those cases where multistrand anchorage devices are used for slab tendons, 18.15 is applicable.

The bursting reinforcement perpendicular to the plane of the slab required by 18.14.2.3 for groups of relatively closely spaced tendons should also be provided in the case of widely spaced tendons if an anchorage device failure could cause more than local damage.

R18.14.3 **General-zone design for groups of monostrand tendons in beams and girders.** Groups of monostrand tendons with individual monostrand anchorage devices are often used in beams and girders. Anchorage devices can be treated as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage device in the direction considered. If a beam or girder has a single anchorage device or a single group of closely spaced anchorage devices, the use of simplified equations such as those given in R18.13.5 is allowed, unless 18.13.5.2 governs. More complex conditions can be designed using strut-and-tie models. Detailed recommendations for use of such models are given in References 18.22 and 18.23 as well as in R18.13.5.

SECTION R18.15
DESIGN OF ANCHORAGE ZONES FOR MULTISTRAND TENDONS

R18.15.1 **Local zone design.** See R18.13.2.

R18.15.2 **Use of special anchorage devices**. Skin reinforcement is reinforcement placed near the outer faces in the anchorage zone to limit local crack width and spacing. Reinforcement in the general zone for other actions (flexure, shear, shrinkage, temperature and similar) may be used in satisfying the supplementary skin reinforcement requirement. Determination of the supplementary skin reinforcement depends on the anchorage device hardware used and frequently cannot be determined until the shop-drawing stage.

SECTION R18.16
CORROSION PROTECTION FOR UNBONDED TENDONS

R18.16.1 Suitable material for corrosion protection of unbonded prestressting steel should have the properties identified in Section 5.1 of Reference 18.23.

R18.16.2 Typically, sheathing is a continuous, seamless, high-density polythylene material that is extruded directly onto the coated prestressing steel.

SECTION R18.17
POST-TENSIONING DUCTS

R18.17.4 Water in ducts may cause distress to the surrounding concrete upon freezing. When strands are present, ponded water in ducts should also be avoided. A corrosion inhibitor should be used to provide temporary corrosion protection if prestressing steel is exposed to prolonged periods of moisture in the ducts before grouting.[18.24]

SECTION R18.18
GROUT FOR BONDED TENDONS

Proper grout and grouting procedures are critical to post-tensioned construction.[18.25, 18.26] Grout provides bond. between the prestressing steel and the duct, and provides corrosion protection to the prestressing steel

R18.18.2 The limitations on admixtures in 3.6 apply to grout. Substances known to be harmful to tendons, grout, or concrete are chlorides, fluorides, sulfites, and nitrates. Aluminum powder or other expansive admixtures, when approved, should produce an unconfined expansion of 5 to 10 percent. Neat cement grout is used in almost all building construction. Use of finely graded sand in the grout should only be considered with large ducts having large void areas.

R18.18.3 **Selection of grout proportions.** Grout proportioned in accordance with these provisions will generally lead to 7 day compressive strength on standard 50 mm cubes in excess of 18 MPa and 28 day strengths of about 30 MPa. The handling and placing properties of grout are usually given more consideration than strength when designing grout mixtures.

R18.18.4 **Mixing and pumping grout**. In an ambient temperature of 2^{o}C, grout with an initial minimum temperature of 15^{o}C may require as much as 5 days to reach 6 MPa. A minimum grout temperature of 15^{o}C is suggested because it is consistent with the recommended minimum temperature for concrete placed at an ambient temperature of 2^{o}C. Quickset grouts, when approved, may require shorter periods of protection and the recommendations of the suppliers should be followed. Test cubes should be cured under temperature and moisture conditions as close as possible to those of the grout in the member. Grout temperatures in excess of 30^{o}C will lead to difficulties in pumping.

SECTION R18.20
APPLICATION AND MEASUREMENT OF PRESTRESSING FORCE

R18.20.1 Elongation measurements for prestressed elements should be in accordance with the procedures outlined in the "Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products," published by the Precast Prestressed Concrete Institute.[18.27]

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 prestressing steel in post-tensioning applications may be affected to varying degrees by placing tolerances and small irregularities in tendon profile due to concrete placement. The friction coefficients between the pre-stressing steel and the duct are also subject to variation.

R18.20.4 This provision applies to all prestressed concrete members. For cast-in-place post-tensioned slab systems, a member should be that portion considered as an element in the design, such as the joist and effective slab width in one-way joist systems, or the column strip or middle strip in two-way flat plate systems.

SECTION R18.21
POST-TENSIONING ANCHORAGES AND COUPLERS

R18.21.1 The prestressing steel 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 prestressing steel 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 when testing to failure. Tendon assemblies should conform to the 2 percent elongation requirements in Ref 18.28 and industry recommendation.[18.14] Anchorages and couplers for bonded tendons that develop less than 100 percent of the specified breaking strength of the prestressing steel 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 prestressing steel strength. This bond length may be calculated by the results of tests of bond characteristics of untensioned prestressing strand [18.29] or by bond tests on other prestressing steel materials, as appropriate.

R18.21.3 For discussion on fatigue loading, see Reference 18.30.

For detailed recommendations on tests for static and cyclic loading conditions for tendons and anchorage fittings of unbonded tendons, see Section 4.1.3 of Reference 18.9, and Section 15.2.2 of Reference 18.28.

R18.21.4 For recommendations regarding protection see Sections 4.2 and 4.3 of Reference 18.9, and also Reference 18.23.

SECTION R18.22
EXTERNAL POST-TENSIONING

External attachment of tendons is a versatile method of providing additional strength, or improving serviceability, or both, in existing structures. It is well suited to repair or upgrade existing structures and permits a wide variety of tendon arrangements.

Additional information on external post-tensioning is given in Reference 18.31.

R18.22.3 External tendons are often attached to the concrete member at various locations between anchorages (such as midspan, quarter points, or third points) for desired load balancing effects, for tendon alignment, or to address tendon vibration concerns. Consideration should be given to the effects caused by the tendon profile shifting in relationship to the concrete centroid as the member deforms under effects of post-tensioning and applied load.

R18.22.4 Permanent corrosion protection can be achieved by a variety of methods. The corrosion protection provided should be suitable to the environment in which the tendons are located. Some conditions will require that the prestressing steel be protected by concrete cover or by cement grout in polyethylene or metal tubing; other conditions will permit the protection provided by coatings such as paint or grease. Corrosion protection methods should meet the fire protection requirements of the general building SBC 304, unless the installation of external post-tensioning is to only improve serviceability.

CHAPTER 19
SHELLS AND FOLDED PLATE MEMBERS

SECTION R19.0
NOTATION

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

SECTION R19.1
SCOPE AND DEFINITIONS

The SBC 304 and commentary provide information on the design, analysis, and construction of concrete thin shells and folded plates. Additional information may be found in Refs. 19.1 and 19.2.

Since Chapter 19 applies to concrete thin shells and folded plates of all shapes, extensive discussion of their design, analysis, and construction in the commentary is not possible. Additional information can be obtained from the references. Performance of shells and folded plates requires special attention to detail.[19.3]

R19.1.1 Discussion of the application of thin shells in special structures such as cooling towers and circular prestressed concrete tanks may be found in Refs. 19.4 and 19.5.

R19.1.3 Common types of thin shells are domes (surfaces of revolution),[19.6,19.7] cylindrical shell,[19.7] barrel vault,[19.8] conoids,[19.8] elliptical paraboloid,[19.8] hyperbolic paraboloid,[19.9] and groined vaults.[19.9]

R19.1.4 Folded plates may be prismatic,[19.6,19.7] nonprismatic[19.7] or faceted. The first two types consist generally of planar thin slabs joined along their longitudinal edges to form a beam-like structure spanning between supports. Faceted folded plates are made up of triangular or polygonal planar thin slabs joined along their edges to form three-dimensional spatial structures.

R19.1.5 Ribbed shell[19.8,19.9] generally have been used for larger spans where the increased thickness of the curved slab alone becomes excessive or uneconomical. Ribbed shells are also used because of the construction techniques employed and to enhance the aesthetic impact of the completed structure.

R19.1.6 Most thin shell structures require ribs or edge beams at their boundaries to carry the shell boundary forces, to assist in transmitting them to the supporting structure, and to accommodate the increased amount of reinforcement in these areas.

R19.1.7 Elastic analysis of thin shells and folded plates can be performed using any method of structural analysis based on assumptions that provide suitable approximations to the three-dimensional behavior of the structure. The method should determine the internal forces and displacements needed in the design of the shell proper, the rib or edge members, and the supporting structure.

Equilibrium of internal forces and external loads and compatibility of deformations should be satisfied.

Methods of elastic analysis based on classical shell theory, simplified mathematical or analytical models, or numerical solutions using finite element[19.10] finite difference[19.8] or numerical integration technique[19.8,19.11] 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 the extent of personal or documented experience regarding the reliability of the given method of analysis in predicting the behavior of the specific type of shell[19.8] or folded plate.[19.7]

R19.1.8 Inelastic analysis of thin shells and folded plates can be performed using a refined method of analysis based on the specific nonlinear material properties, nonlinear behavior due to the cracking of concrete, and time-dependent effects such as creep, shrinkage, temperature, and load history. These effects are incorporated in order to trace the response and crack propagation of a reinforced concrete shell through the elastic, inelastic, and ultimate ranges. Such analyses usually require incremental loading and iterative procedures to converge on solutions that satisfy both equilibrium and strain compatibility.[19.12,19.13]

SECTION R19.2
ANALYSIS AND DESIGN

R19.2.1 For types of shell structures where experience, tests, and analyses have shown that the structure can sustain reasonable overloads without undergoing brittle ailure, elastic analysis is an acceptable procedure. The designer may assume that reinforced concrete is ideally elastic, homogeneous, and isotropic, having identical properties in all directions. An analysis should be performed for the shell considering service load conditions. The analysis of shells of unusual size, shape, or complexity should consider behavior through the elastic, cracking, and inelastic stages.

R19.2.2 Several inelastic analysis procedures contain possible solution methods.[19.12,19.13]

R19.2.4 Experimental analysis of elastic model[19.14] has been used as a substitute for an analytical solution of a complex shell structure. Experimental analysis of reinforced micro concrete models through the elastic, cracking, inelastic, and ultimate stages should be considered for important shells of unusual size, shape, 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.

R19.2.5 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 proved that safe designs have resulted from their use. Such methods include beam-type analysis for barrel shells and folded plates having large ratios of span to either width or radius of curvature, simple membrane analysis for shells of revolution, and others in which the equations of equilibrium are satisfied, while the strain compatibility equations are not.

R19.2.6 If the shell is prestressed, the analysis should 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 should be given to the resulting force components. The effects of post-tensioning of shell supporting members should be taken into account.

R19.2.7 The thin shell's thickness and reinforcement are required to be proportioned to satisfy the strength provisions of this SBC 304, and to resist internal forces obtained from an analysis, an experimental model 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 19.2.8, or by the SBC 304 minimum thickness requirements.

R19.2.8 Thin shells, like other structures that experience in-plane membrane compressive forces, are subject to buckling when the applied load reaches a critical value. Because of the surface-like geometry of shells, the problem of calculating buckling load is complex. If one of the principal membrane forces is tensile, the shell is less likely to buckle than if both principal membrane forces are compressive. The kinds of membrane forces that develop in a shell depend on its initial shape and the manner in which the shell is supported and loaded. In some types of shells, post-buckling behavior should be considered in determining safety against instability.[19.2]

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 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 recommendation of Ref. 19.2. Some recommendations for buckling design of domes used in industrial applications are given in Refs.19.5 and 19.15.

R19.2.10 The stresses and strains in the shell slab used for design are those determined by analysis (elastic or inelastic) multiplied by appropriate load factors. Because of detrimental effects of membrane cracking, the computed tensile strain in the reinforcement under factored loads should be limited.

R19.2.11 When principal tensile stress produces membrane cracking in the shell, experiments indicate the attainable compressive strength in the direction parallel to the cracks is reduced.[19.16,19.17]

SECTION R19.4
SHELL REINFORCEMENT

R19.4.1 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 membrane crack width and spacing due to shrinkage, temperature, and service load conditions is a major design consideration.

R19.4.2 The requirement of ensuring strength in all directions is based on safety considerations. Any method that ensures sufficient strength consistent with equilibrium is 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 does not 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 are objectionable, the computation of reinforcement may have to be based on a more refined approach[19.16,19.18,19.19] 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 tensile resisting forces in reinforcement and compressive resisting forces in concrete.

The alternative method to calculate orthogonal reinforcement is the shear-friction method. It is based on the assumption that shear integrity of a shell should be maintained at factored loads. It is not necessary to calculate principal stresses if the alternative approach is used.

R19.4.3 Minimum membrane reinforcement corresponding to slab shrinkage and temperature reinforcement are to be provided in at least two approximately orthogonal directions even if the calculated membrane forces are compressive in one or more directions.

R19.4.5 The requirement that the tensile reinforcement yields before the concrete crushes anywhere is consistent with 10.3.3. Such crushing can also occur in regions near supports and for some shells where the principal membrane forces are approximately equal and opposite in sign.

R19.4.6 Generally, for all shells, and particularly in regions of substantial tension, the orientation of reinforcement should approximate 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.

R19.4.7 When the directions of reinforcement deviate significantly (more than 10 deg) from the directions of the principal membrane forces, higher strains in the shell occur to develop the capacity of reinforcement. This might lead to the development of unacceptable 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 Ref. 19.20. 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 closer spacing of smaller diameter bars.

R19.4.8 The practice of concentrating tensile reinforcement in the regions of maximum tensile stress has led to a number of successful and economical designs, primarily for long folded plates, long 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.

R19.4.9 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.[19.21] The sign of 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.

R19.4.10 The value of ϕ to be used is that prescribed in 9.3.2.1 for axial tension.

R19.4.11 and
R19.4.12 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 19.4.11 and 19.4.12 require extra reinforcement length to maintain the minimum lengths on curved surfaces.

SECTION R19.5
CONSTRUCTION

R19.5.1 When early removal of forms is necessary, the magnitude of the modulus of elasticity at the time of proposed form removal should be investigated to ensure safety of the shell with respect to buckling, and to restrict deflections.[19.3,19.22] The value of the modulus of elasticity E_c should be obtained from a flexural test of field-cured specimens. It is not sufficient to determine the modulus from the formula in 8.5.1, even if f_c' is determined for the field-cured specimen.

R19.5.2 In some types of shells, small local deviations from the theoretical geometry of the shell can cause relatively large changes in local stresses and in overall safety against instability. These changes can result in local cracking and yielding that may make the structure unsafe or can greatly affect the critical load producing instability. The effect of such deviations should be evaluated and any necessary remedial actions should be taken. Special attention is needed when using air supported form systems.[19.23]

CHAPTER 20
STRENGTH EVALUATION OF EXISTING STRUCTURES

SECTION R20.0
NOTATION

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

SECTION R20.1
STRENGTH EVALUATION – GENERAL

Chapter 20 does not cover load testing for the approval of new design or construction methods. (See 16.10 for recommendations on strength evaluation of precast concrete members.) Provisions of Chapter 20 may be used to evaluate whether a structure or a portion of a structure satisfies the safety requirements of the SBC. 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 building 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 SBC 304. In such cases, Chapter 20 provides guidance for investigating the safety of the structure.

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.

R20.1.2 Strength considerations related to axial load, flexure, and combined axial load and flexure are well understood. There are reliable theories relating strength and short-term displacement to load in terms of dimensional and material data for the structure.

To determine the strength of the structure by analysis, calculations should be based on data gathered on the actual dimensions of the structure, properties of the materials in place, and all pertinent details. Requirements for data collection are in 20.2.

R20.1.3 If the shear or bond strength of an element is critical in relation to the doubt expressed about safety, a test may be the most efficient solution to eliminate or confirm the doubt. A test may also be appropriate if it is not 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, support the results of the load test by analysis.

R20.1.4 For a deteriorating structure, the acceptance provided by the load test may not be assumed to be without limits in terms of time. In such cases, a periodic inspection program is useful. A program that involves physical tests and periodic inspection can justify a longer period in service. Another option for maintaining the structure in service, while the periodic inspection program continues, is to limit the live load to a level determined to be appropriate.

The length of the specified time period should be based on consideration of (a) the nature of the problem, (b) environmental and load effects, (c) service history of the structure, and (d) scope of the periodic inspection program. At the end of a specified time period, further strength evaluation is required if the structure is to remain in service.

With the agreement of all concerned parties, special procedures may be devised for periodic testing that do not necessarily conform to the loading and acceptance criteria specified in Chapter 20 of SBC 304.

SECTION R20.2 DETERMINATION OF REQUIRED DIMENSIONS AND MATERIAL PROPERTIES

This section applies if it is decided to make an analytical evaluation (see 20.1.2).

R20.2.1 Critical sections are where each type of stress calculated for the load in question reaches its maximum value.

R20.2.2 For individual elements, amount, size, arrangement, and location should be determined at the critical sections for reinforcement or tendons, or both, designed to resist applied load. Nondestructive investigation methods are acceptable. In large structures, determination of these data for approximately 5 percent of the reinforcement or tendons in critical regions may suffice if these measurements confirm the data provided in the construction drawings.

R20.2.3 The number of tests may depend on the size of the structure and the sensitivity of structural safety to concrete strength. In cases where the potential problem involves flexure only, investigation of concrete strength can be minimal for a lightly reinforced section ($\rho f_y / f_c' \leq 0.15$ for rectangular section).

R20.2.4 The number of tests required depends on the uniformity of the material and is best determined by the engineer for the specific application.

SECTION R20.3 LOAD TEST PROCEDURE

R20.3.1 **Load arrangement**. It is important to apply the load at locations so that its effects on the suspected defect are a maximum and the probability of unloaded members sharing the applied load is a minimum. In cases where it is shown by analysis that adjoining unloaded elements will help carry some of the load, the load should be placed to develop effects consistent with the intent of the load factor.

R20.3.2 **Load intensity.** The required load intensity follows previous load test practice. The live load L may be reduced as permitted by SBC 301 governing safety considerations for the structure. The live load should be increased to compensate for resistance provided by unloaded portions of the structure in questions. The increase in live load is determined from analysis of the loading conditions in relation to the selected pass/fail criterion for the test.

The test load intensity remained the same. It is considered appropriate for designs using the load combinations and strength reduction factors of Chapter 9.

SECTION R20.4
LOADING CRITERIA

R20.4.2 Inspecting the structure after each load increment is advisable.

R20.4.3 Arching refers to the tendency for the load to be transmitted nonuniformly to the flexural element being tested. For example, if a slab is loaded by a uniform arrangement of bricks with the bricks in contact, arching would results in reduction of the load on the slab near the midspan of the slab.

SECTION R20.5
ACCEPTANCE CRITERIA

R20.5.1 A general acceptance criterion for the behavior of a structure under the test load is that it does not show evidence of failure. Evidence of failure includes cracking, spalling, or deflection of such magnitude and extent that the observed result is obviously excessive and incompatible with the safety requirements of the structure. No simple rules have been developed for application to all types of structures and conditions. If sufficient damage has occurred so that the structure is considered to have failed that test, retesting is not permitted since it is considered that damaged members should not be put into service even at a lower load rating.

Local spalling or flaking of the compressed concrete in flexural elements related to casting imperfections need not indicate overall structural distress. Crack widths are good indicators of the state of the structure and should be observed to help determine whether the structure is satisfactory. However, exact prediction or measurement of crack widths in reinforced concrete elements is not likely to be achieved under field conditions. Establish criteria before the test, relative to the types of cracks anticipated; where the cracks will be measured; how they will be measured; and approximate limits or criteria to evaluate new cracks or limits for the changes in crack width.

R20.5.2 The deflection limits and the retest option follow past practice. If the structure shows no evidence of failure, recovery of deflection after removal of the test load is used to determine whether the strength of the structure is satisfactory. In the case of a very stiff structure, however, the errors in measurements under field conditions may be of the same order as the actual deflections and recovery. To avoid penalizing a satisfactory structure in such a case, recovery measurements are waived if the maximum deflection is less than $\ell_t^2/(20{,}000h)$. The residual deflection $\Delta_{r\max}$ is the difference between the initial and final (after load removal) deflections for the load test or the repeat load test.

R20.5.3 Forces are transmitted across a shear crack plane by a combination of aggregate interlock at the interface of the crack that is enhanced by clamping action of transverse stirrup reinforcing and by dowel action of stirrups crossing the crack. As crack lengths increase to approach a horizontal projected length equal to the depth of the member and concurrently widen to the extent that aggregate interlock cannot occur, and as transverse stirrups if present begin to yield or display loss of anchorage so as to threaten their integrity, the member is assumed to be approaching imminent shear failure.

R20.5.4 The intent of 20.5.4 is to make the professionals in charge of the test pay attention to the structural implication of observed inclined cracks that may lead to brittle collapse in members without transverse reinforcement.

R20.5.5 Cracking along the axis of the reinforcement in anchorage zones may be related to high stresses associated with the transfer of forces between the reinforcement and the concrete. These cracks may be indicators of pending brittle failure of the element if they are associated with the main reinforcement. It is important that their causes and consequences be evaluated.

SECTION R20.6
PROVISION FOR LOWER LOAD RATING

Except for load tested members that have failed under a test (see 20.5), the building official may permit the use of a structure or member at a lower load rating that is judged to be safe and appropriate on the basis of the test results.

CHAPTER 21
SPECIAL PROVISIONS FOR SEISMIC DESIGN

SECTION R21.0
NOTATION

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

SECTION R21.1
DEFINITION

The design displacement is an index of the maximum lateral displacement expected in design for the design-basis earthquake. In References 21.1 through 21.4, the design-basis earthquake has approximately a 90 percent probability of nonexceedance in 50 years. In those documents, the design displacement is calculated using static or dynamic linear elastic analysis under SBC 304 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.

The provisions of 21.6 are intended to result in a special moment frame constructed using precast concrete having minimum strength and toughness equivalent to that for a special moment frame of cast-in-place concrete.

The provisions of 21.13 are intended to result in an intermediate precast structural wall having minimum strength and toughness equivalent to that for an ordinary reinforced concrete structural wall of cast-in-place concrete. A precast concrete wall satisfying only the requirements of Chapters 1 through 18 and not the additional requirements of 21.13 or 21.8 is considered to have ductility and structural integrity less than that for an intermediate precast structural wall.

The provisions of 21.8 are intended to result in a special precast structural wall having minimum strength and toughness equivalent to that for a special reinforced concrete structural wall of cast-in-place concrete.

SECTION R21.2
GENERAL REQUIREMENTS

R21.2.1 **Scope**. Chapter 21 contains provisions considered to be the minimum requirements for a cast-in-place or precast concrete structure capable of sustaining a series of oscillations into the inelastic range of response without critical deterioration in strength. The integrity of the structure in the inelastic range of response should be maintained because the design forces defined in references 21.1, 21.5 and 21.6 provisions are considered less than those corresponding to linear response at the anticipated earthquake intensity.[21.1,21.6-21.8]

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[21.8]. Thus, the use of design forces representing earthquake effects such as those in Reference 21.2 requires that the lateral-force resisting system retain a substantial portion of its strength into the inelastic range under displacement reversals.

The provisions of Chapter 21 relate detailing requirements to type of structural framing, earthquake risk level at the site, level of inelastic deformation intended in structural design, and use and occupancy of the structure. Earthquake risk levels traditionally have been classified as low, moderate, and high. The seismic risk level of a region or the seismic performance or design category of a structure is regulated by provisions of SBC 301.

The design and detailing requirements should be compatible with the level of energy dissipation (or toughness) assumed in the computation of the design seismic loads. The terms ordinary, intermediate, and special are specifically used to facilitate this compatibility. The degree of required toughness, and therefore the level of required detailing, increases for structures progressing from ordinary through intermediate to special categories. It is essential that structures in higher seismic zones or assigned to higher seismic performance or design categories possess a higher degree of toughness. It is permitted, however, to design for higher toughness in the lower seismic zones or design categories and take advantage of the lower design force levels.

The provisions of Chapters 1 through 18 are intended to provide adequate toughness for structures in regions of low seismic risk, or assigned to ordinary categories. Therefore, it is not required to apply the provisions of Chapter 21 to lateral-force resisting systems consisting of ordinary structural walls.

Chapter 2 1 requires special details for reinforced concrete structures in regions of moderate seismic risk, or assigned to intermediate seismic performance or design categories. These requirements are contained in 21.2.1.3, 21.12, and 21.13. Although new provisions are provided in 21.13 for design of intermediate precast structural walls, general building codes that address seismic performance or design categories currently do not include intermediate structural walls.

Structures in regions of high seismic risk, or assigned to high seismic performance or design categories, may be subjected to strong ground shaking. Structures designed using seismic forces based upon response modification factors for special moment frames or special reinforced concrete structural walls are likely to experience multiple cycles of lateral displacements well beyond the point where reinforcement yields should the design earthquake ground shaking occur. The provisions of 2 1.2 through 2 1.11 have been developed to provide the structure with adequate toughness for this special response.

The requirements of Chapter 21 as they apply to various components of structures in regions of intermediate or high seismic risk, or assigned to intermediate or high seismic performance or design categories, are summarized in Table R21.2.1

The special proportioning and detailing requirements in Chapter 21 are based predominantly on field and laboratory experience with monolithic reinforced concrete building structures and precast concrete building structures designed and detailed to behave like monolithic building structures. Extrapolation of these requirements to other types of cast-in-place or precast concrete structures should be based on evidence provided by field experience, tests, or analysis. ACI T1.1-01, "Acceptance Criteria for Moment Frames Based on Structural Testing," can be used in conjunction with Chapter 21 to demonstrate that the strength and toughness of a proposed frame system equals or exceeds that provided by a comparable monolithic concrete system.

The toughness requirements in 21.2.1.5 refer to the concern for the structural integrity of the entire lateral-force resisting system at lateral displacements anticipated for ground motions corresponding to the design earthquake. Depending on the energy-dissipation characteristics of the structural system used, such displacements may be larger than for a monolithic reinforced concrete structure.

TABLE R21.2.1- SECTIONS OF CHAPTER 21 TO BE SATISFIED*

Component resisting earthquake effect, unless otherwise noted	Level of seismic risk or assigned seismic performance or design categories (as defined in SBC 304 section)	
	Intermediate (21.2.1.3)	High (21.2.1.4)
Frame members	21.12	21.2, 21.3, 21.4, 21.5
Structural walls and coupling beams	None	21.2, 21.7
Precast structural walls	21.13	21.2, 21.8
Structural diaphragms and trusses	None	21.2, 21.9
Foundations	None	21.2, 21.10
Frame members not proportioned to resist forces induced by earthquake motions	None	21.11

*In addition to requirements of Chapters 1 through 18 for structures at intermediate seismic risk (21.2.1.3), and for Chapters 1 through 17 for structures at high seismic risk (21.2.1.4).

R21.2.2 **Analysis and proportioning of structural members**. It is assumed that the distribution of required strength to the various components of a lateral-force resisting system will be guided by the analysis of a linearly elastic model of the system acted upon by the factored forces required by the governing SBC 304. 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 admits nonlinear response, it is necessary to investigate the stability of the lateral-force resisting system as well as its interaction with other structural and nonstructural members at displacements larger than those indicated by linear analysis. To handle this without having to resort to nonlinear response analysis, one option is to multiply by a factor of at least two the displacements from linear analysis by using the factored lateral forces, unless the governing SBC 304 specifies the factors to be used as in References 21.1 and

21.2. For lateral displacement calculations, assuming all the horizontal structural members to be fully cracked is likely to lead to better estimates of the possible drift than using uncracked stiffness for all members.

The main concern of Chapter 21 is the safety of the structure. The intent of 21.2.2.1 and 21.2.2.2 is to draw attention to the influence of nonstructural members on structural response and to hazards from falling objects.

Section 21.2.2.3 alerts the designer that the base of the structure as defined in analysis may not necessarily correspond to the foundation or ground level.

In selecting member sizes for earthquake-resistant structures, it is important to consider problems related to congestion of reinforcement. The designer should ensure that all reinforcement can be assembled and placed and that concrete can be cast and consolidated properly. Use of upper limits of reinforcement ratios permitted is likely to lead to insurmountable construction problems especially at frame joints.

R21.2.4 **Concrete in members resisting earthquake-induced forces.** Requirements of this section refer to concrete quality in frames, trusses, or walls proportioned to resist earthquake-induced forces. The maximum design compressive strength of lightweight aggregate concrete to be used in structural design calculations is limited to 35 MPa, primarily because of paucity of experimental and field data on the behavior of members made with lightweight aggregate concrete subjected to displacement reversals in the nonlinear range. If convincing evidence is developed for a specific application, the limit on maximum compressive strength of lightweight aggregate concrete may be increased to a level justified by the evidence.

R21.2.5 **Reinforcement in members resisting earthquake-induced forces**. Use of longitudinal reinforcement with strength substantially higher than that assumed in design will lead to higher shear and bond stresses at the time of development of yield moments. These conditions may lead to brittle failures in shear or bond and should be avoided even if such failures may occur at higher loads than those anticipated in design. Therefore, a ceiling is placed on the actual yield strength of the steel [see 21.2.5(a)].

The requirement for an ultimate tensile strength larger than the yield strength of the reinforcement [21.2.5(b)] is based on the assumption that the capability of a structural member to develop inelastic rotation capacity is a function of the length of the yield region along the axis of the member. In interpreting experimental results, the length of the yield region has been related to the relative magnitudes of ultimate and yield moments.[21.10] According to this interpretation, the larger the ratio of ultimate to yield moment, the longer the yield region. Chapter 21 requires that the ratio of 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.

R21.2.6 **Mechanical splices**. In a structure undergoing inelastic deformations during an earthquake, the tensile stresses in reinforcement may approach the tensile strength of the reinforcement. The requirements for Type 2 mechanical splices are intended to avoid a splice failure when the reinforcement is subjected to expected stress levels in yielding regions. Type 1 splices are not required to satisfy the more stringent requirements for Type 2 splices, and may not be capable of resisting the stress levels expected in yielding regions. The locations of Type 1 splices are restricted because tensile stresses in rein- forcement in yielding regions can exceed the strength requirements of 3.5.2 and 12.14.3.3.

Recommended detailing practice would preclude the use of splices in regions of potential yield in members resisting earthquake effects. If use of mechanical splices in regions of potential yielding cannot be avoided, the designer should have documentation on the actual strength characteristics of the bars to be spliced, on the force-deformation characteristics of the spliced bar, and on the ability of the Type 2 splice to be used to meet the specified performance requirements.

R21.2.7 Welded splices

R21.2.7.1 Welding of reinforcement should be according to ANSI/AWS D1.4 as required in Chapter 3. The locations of welded splices are restricted because reinforcement tension stresses in yielding regions can exceed the strength requirements of 12.14.3.4.

R21.2.7.2 Welding of crossing reinforcing bars can lead to local embrittlement of the steel. If welding of crossing bars is used to facilitate fabrication or placement of reinforcement, it should be done only on bars added for such purposes. The prohibition of welding crossing reinforcing bars does not apply to bars that are welded with welding operations under continuous, competent control as in the manufacture of welded wire fabric.

SECTION R21.3
FLEXURAL MEMBERS OF SPECIAL MOMENT FRAMES

R21.3.1 **Scope**. This section refers to beams of special moment frames resisting lateral loads induced by earthquake motions. Any frame member subjected to a factored axial compressive force exceeding $\left(A_g f_c^{'}/10\right)$ is to be proportioned and detailed as described in 21.4.

Experimental evidence[21.11] indicates that, under reversals of displacement into the nonlinear range, behavior of continuous members having length-to-depth ratios of less than four is significantly different from the behavior of relatively slender members. Design rules derived from experience with relatively slender members do not apply directly to members with length-to-depth ratios less than four, especially with respect to shear strength.

Geometric constraints indicated in 21.3.1.3 and 21.3.1.4 were derived from practice with reinforced concrete frames resisting earthquake-induced forces[21.12].

R21.3.2 **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) fails to describe the conditions in a flexural member subjected to reversals of displacements well into the inelastic range. Thus, there is little rationale for continuing to refer to balanced conditions in earthquake-resistant design of reinforced concrete structures.

R21.3.2.1 The limiting reinforcement ratio of 0.02 is based primarily on considerations of steel congestion and, indirectly, on limiting shear stresses in girders of typical proportions. The requirement of at least two bars, top and bottom, refers again to construction rather than behavioral requirements.

R21.3.2.3 Lap splices of reinforcement are prohibited at regions where flexural yielding is anticipated because such splices are not reliable under conditions of cyclic loading into the inelastic range. Transverse reinforcement for lap splices at any location is mandatory because of the likelihood of loss of shell concrete.

R21.3.3 **Transverse reinforcement.** Transverse reinforcement is required primarily to confine the concrete and maintain lateral support for the reinforcing bars in regions where yielding is expected. Examples of hoops suitable for flexural members of frames are shown in Fig. R21.3.3.

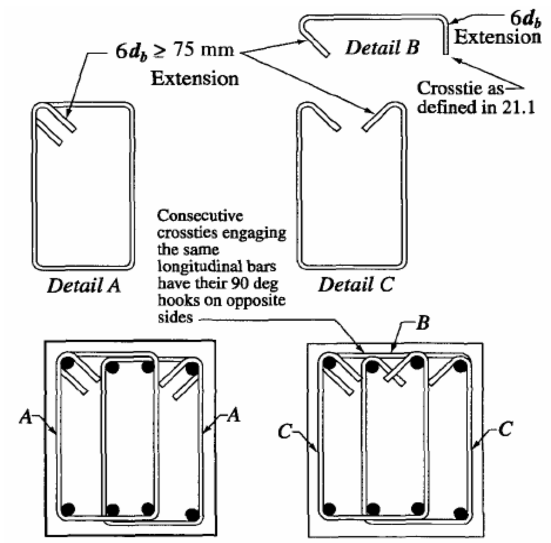


Fig. R21.3.3 - Examples of overlapping hoops

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 21.3.3.5.

R21.3.4 Shear strength requirements

R21.3.4.1 Design forces. In determining the equivalent lateral forces representing earthquake effects for the type of frames considered, it is assumed that frame members will dissipate energy in the nonlinear range of response. Unless a frame member possesses a strength that is a multiple on the order of 3 or 4 of the design forces, it should be assumed that it will yield in the event of a major earthquake. The design shear force should be a good approximation of the maximum shear that may develop in a member. Therefore, required shear strength for frame members is related to flexural strengths of the designed member rather than to factored shear forces indicated by lateral load analysis. The conditions described by 21.3.4.1 are illustrated in Fig. R21.3.4.

Because the actual yield strength of the longitudinal reinforcement may exceed the specified yield strength and because strain hardening of the reinforcement is likely to take place at a joint subjected to large rotations, required shear strengths are determined using a stress of at least $1.25f_y$ in the longitudinal reinforcement.

R21.3.4.2 Transverse reinforcement. Experimental studies[21.13,21.14] of reinforced concrete members subjected to cyclic loading have demonstrated that more shear reinforcement is required to ensure a flexural failure if the member is subjected to alternating nonlinear displacements than if the member is loaded in only one direction: the necessary increase of shear reinforcement being higher in the case of no axial load. This observation is reflected in the SBC 304 (see 21.3.4.2) by eliminating the term representing the contribution of concrete to shear strength. The added conservatism on shear is deemed necessary in locations where potential flexural hinging may occur. 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 of 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.

Notes on Fig. R21.3.4

1. Direction of shear force V_e depends on relative magnitudes of gravity loads and shear generated by end moments.
2. End moments M_{pr} based on steel tensile stress of $1.25 f_y$, where f_y is specified yield strength. (Both end moments should be considered in both directions, clockwise and counter-clockwise).
3. End moment M_{pr} for columns need no be greater than moments generated by the M_{pr} of the beams framing into the beam-column joints. V_e should not be less than that required by analysis of the structure.

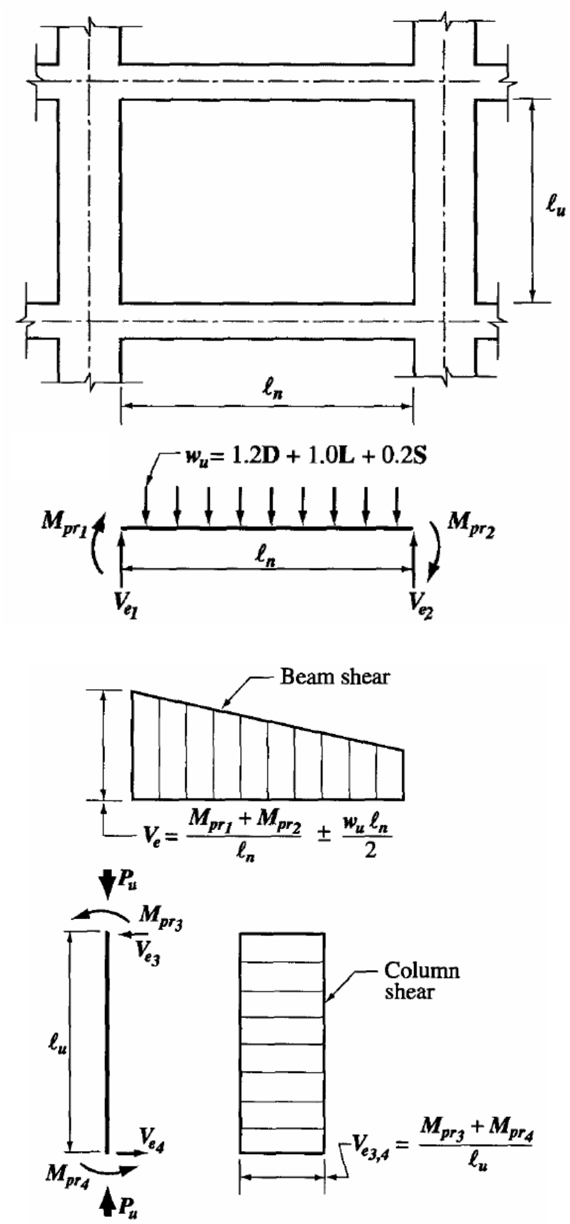


Fig. R21.3.4 - Design shears for girders and columns

SECTION R21.4
SPECIAL MOMENT FRAME MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

R21.4.1 **Scope.** Section 21.4.1 is intended primarily for columns of special moment frames. Frame members, other than columns, that do not satisfy 21.3.1 are to be proportioned and detailed according to this section.
The geometric constraints in 21.4.1.1 and 21.4.1.2 follow from previous practice.[21.12]

R21.4.2 **Minimum flexural strength of columns.** The intent of 21.4.2.2 is to reduce the likelihood of yielding in columns that are considered as part of the lateral force resisting system. If columns are not stronger than beams framing into a joint, there is likelihood of inelastic action. In the worst case of weak columns, flexural yielding can occur at both ends of all columns in a given story, resulting in a column failure mechanism that can lead to collapse.

In 21.4.2.2, the nominal strengths of the girders and columns are calculated at the joint faces, and those strengths are compared directly using Eq. (21-1).

When determining the nominal flexural strength of a girder section in negative bending (top in tension), longitudinal reinforcement contained within an effective flange width of a top slab that acts monolithically with the girder increases the girder strength. Research[21.15] on beam-column subassemblies under lateral loading indicates that using the effective flange widths defined in 8.10 gives reasonable estimates of girder negative bending strengths of interior connections at interstory displacement levels approaching 2 percent of story height. This effective width is conservative where the slab terminates in a weak spandrel.

If 21.4.2.2 cannot be satisfied at a joint, any positive contribution of the column or columns involved to the lateral strength and stiffness of the structure is to be ignored. Negative contributions of the column or columns should not be ignored. For example, ignoring the stiffness of the columns ought not to 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 provisions of SBC 301.

R21.4.3 **Longitudinal reinforcement.** The lower limit of the reinforcement ratio is to control time-dependent deformations and to have the yield moment exceed the cracking moment. The upper limit of the section reflects concern for steel congestion, load transfer from floor elements to column especially in low-rise construction, and the development of high shear stresses.

Spalling of the shell concrete, which is likely to occur near the ends of the column in frames of typical configuration, makes lap splices in 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. Special transverse reinforcement is required along the lap-splice length because of the uncertainty in moment distributions along the height and the need for confinement of lap splices subjected to stress reversals.[21.16]

R21.4.4 **Transverse reinforcement.** Requirements of this section are concerned with confining the concrete and providing lateral support to the longitudinal reinforcement.

The effect of helical (spiral) reinforcement and adequately configured rectangular hoop reinforcement on strength and ductility of columns is well established[21.17] While analytical procedures exist for calculation of strength and ductility capacity of columns under axial and moment reversals[21.18],the axial load and deformation demands required during earthquake loading are not known with sufficient accuracy to justify calculation of required transverse reinforcement as a function of design earthquake demands. Instead, Eq. (10-5) and (21-3) are required, with the intent that spalling of shell concrete will not result in a loss of axial load strength of the column. Eq. (21-2) and (21-4) govern for large-diameter columns, and are intended to ensure adequate flexural curvature capacity in yielding regions.

Fig. R21.4.4 shows an example of transverse reinforcement provided by one hoop and three crossties. Crossties with a 90 deg hook are not as effective as either crossties with 135 deg hooks or hoops in providing confinement. Tests show that if crosstie ends with 90 deg hooks are alternated, confinement will be sufficient.

Sections 21.4.4.2 and 21.4.4.3 are interrelated requirements for configuration of rectangular hoop reinforcement. The requirement that spacing not exceed one-quarter of the minimum member dimension is to obtain adequate concrete confinement. The requirement that spacing not exceed six bar diameters is intended to restrain longitudinal reinforcement buckling after spalling. The 100 mm spacing is for concrete confinement; 21.4.4.2 permits this limit to be relaxed to a maximum of 150 mm if the spacing of crossties or legs of overlapping hoops is limited to 200 mm.

The unreinforced shell may spall as the column deforms to resist earthquake effects. Separation of portions of the shell from the core caused by local spalling creates a falling hazard. The additional reinforcement is required to reduce the risk of portions of the shell falling away from the column.

Section 21.4.4.4 stipulates a minimum length over which to provide closely-spaced transverse reinforcement at the member ends, where flexural yielding normally occurs. Research results indicate that the length should be increased by 50 percent or more in locations, such as the base of the building, where axial loads and flexural demands may be especially high.[21.19]

Columns supporting discontinued stiff members, such as walls or trusses, may develop considerable inelastic response. Therefore, it is required that these columns have special transverse reinforcement throughout their length. This covers all columns beneath the level at which the stiff member has been discontinued, unless the factored forces corresponding to earthquake effect are low (see 21.4.4.5).

Field observations have shown significant damage to columns in the unconfined region near the midheight. The requirements of 21.4.4.6 are to ensure a relatively uniform toughness of the column along its length.

R21.4.4.6 The provisions of 21.4.4.6 provide reasonable protection and ductility to the midheight of columns between transverse reinforcement. Observations after earthquakes have shown significant damage to columns in the nonconfined region, and the minimum ties or spirals required should provide a more uniform toughness of the column along its length.

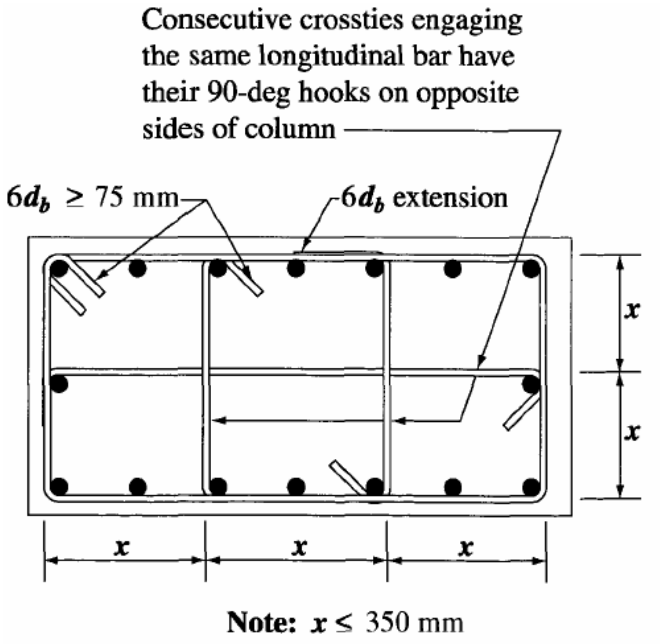


Fig. R21.4.4 - Example of transverse reinforcement in columns

R21.4.5 **Shear strength requirements**

R21.4.5.1 **Design forces.** The provisions of 21.3.4.1 also apply to members subjected to axial loads (for example, columns). Above the ground floor the moment at a joint may be limited by the flexural strength of the beams framing into the joint. Where beams frame into opposite sides of a joint, the combined strength may be the sum of the negative moment strength of the beam on one side of the joint and the positive moment strength of the beam on the other side of the joint. Moment strengths are to be determined using a strength reduction factor of 1.0 and reinforcing steel stress equal to at least $1.25 f_y$. 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. R21.3.4 may be computed from the flexural member strengths at the beam-column joints.

SECTION R21.5
JOINTS OF SPECIAL MOMENT FRAMES

R21.5.1 **General requirements.** Development of inelastic rotations at the faces of joints of reinforced concrete frames is associated with strains in the flexural reinforcement well in excess of the yield strain. Consequently, joint shear force generated by the flexural reinforcement is calculated for a stress of $1.45 f_y$ in the reinforcement (see 21.5.1.1). A detailed explanation of the reasons for the possible development of stresses in excess of the yield strength in girder tensile reinforcement is provided in Reference 21.10.

R21.5.1.4 Research [21.20-21.24] has shown that straight beam bars may slip within the beam-column joint during a series of large moment reversals. The bond stresses on these straight bars may be very large. To substantially reduce slip during the formation of adjacent beam hinging, it would be necessary to have a ratio of column dimension to bar diameter of approximately 1/32, which would result in very large joints. On reviewing the available tests, the limit of 1/25 of the column depth in the direction of loading for the maximum size of beam bars for normalweight concrete and a limit of 1/30 for lightweight concrete were chosen. Due to the lack of specific data, the modification for lightweight concrete used a factor of 1.2. These limits provide reasonable control on the amount of potential slip of the beam bars in a beam-column joint, considering the number of anticipated inelastic excursions of the building frames during a major earthquake. A thorough treatment of this topic is given in Reference 21.25.

R21.5.2 **Transverse reinforcement.** No matter how low the calculated shear force in a joint of a frame resisting earthquake-induced forces, confining reinforcement (see 21.4.4) should be provided through the joint around the column reinforcement (see 21.5.2.1). In 21.5.2.2, confining reinforcement may be reduced if horizontal members frame into all four sides of the joint. A maximum limit on spacing to these areas is based on available data (References 21.26 through 21.29).

Section 21.5.2.3 refers to a joint where the width of the girder exceeds the corresponding column dimension. In that case, girder reinforcement not confined by the column reinforcement should be provided lateral support either by a girder framing into the same joint or by transverse reinforcement.

R21.5.3 **Shear strength.** The requirements in Chapter 21 for proportioning joints are based on Reference 21.10 in that behavioral phenomena within the joint are interpreted in terms of a nominal shear strength of the joint. Because tests of joints[2120] and deep beams[21.11] indicated that shear strength was not as sensitive to joint (shear) reinforcement as implied by the expression developed by Reference 21.30 for beams and adopted to apply to joints by Reference 21.10. This SBC 304 set the strength of the joint as a function of only the compressive strength of the concrete (see 21.5.3) and to require a minimum amount of transverse reinforcement in the joint (see 21.5.2). The effective area of joint A_j is illustrated in Fig. R21.5.3.

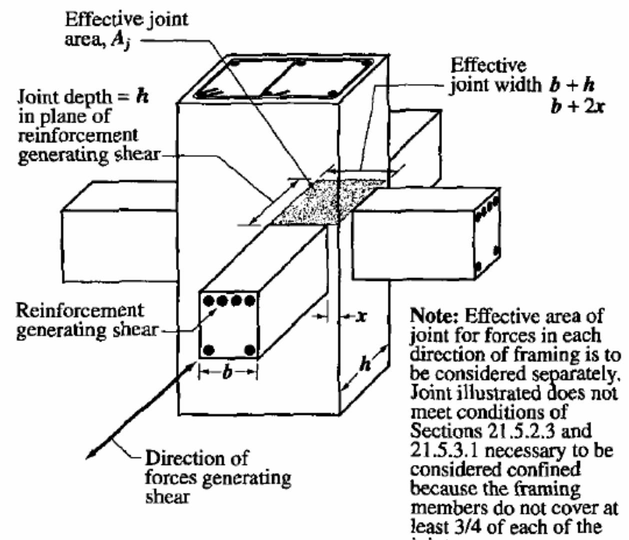


Fig. R21.5.3 - Effective joint area

In no case is A_j greater than the column cross-sectional area.

The three levels of shear strength required by 21.5.3.1 are based on the recommendation of Reference 21.10.

R21.5.4 Development length of bars in tension. Minimum development length for deformed bars with standard hooks embedded in normalweight concrete is determined using Eq. (21-6). Eq. (21-6) is based on the requirements of 12.5. Because Chapter 21 stipulates that the hook is to be embedded in confined concrete, the coefficients 0.7 (for concrete cover) and 0.8 (for ties) have been incorporated in the constant used in Eq. (21-6). The development length that would be derived directly from 12.5 is increased to reflect the effect of load reversals.

The development length in tension for a reinforcing bar with a standard hook is defined as the distance, parallel to the bar, from the critical section (where the bar is to be developed) to a tangent drawn to the outside edge of the hook. The tangent is to be drawn perpendicular to the axis of the bar (see Fig. R12.5).

Factors such as the actual stress in the reinforcement being more than the yield force and the effective development length not necessarily starting at the face of the joint were implicitly considered in the development of the expression for basic development length that has been used as the basis for Eq. (21-6).

For lightweight aggregate concrete, the length required by Eq. (21-6) is to be increased by 25 percent to compensate for variability of bond characteristics of reinforcing bars in various types of lightweight aggregate concrete.

Section 21.5.4.2 specifies the minimum development length for straight bars as a multiple of the length indicated by 21.5.4.1. Section 21.5.4.2(b) refers to top bars. If the required straight embedment length of a reinforcing bar extends beyond the confined volume of concrete (as defined in 21.3.3,21.4.4, or 21.5.2), the required development length is increased on the premise that the limiting bond stress outside the confined region is less than that inside.

or $$\ell_{dm} = 1.6(\ell_d - \ell_{dc}) + \ell_{dc}$$

where $$\ell_{dm} = 1.6\ell_d - 0.6\ell_{dc}$$

ℓ_{dm} = required development length if bar is not entirely embedded in confined concrete;

ℓ_d = required development length for straight bar embedded in confined concrete (see 21.5.4.3);

ℓ_{dc} = length of bar embedded in confined concrete

Lack of reference to Dia 40 mm bars and larger in 21.5.4 is due to the paucity of information on anchorage of such bars subjected to load reversals simulating earthquake effects.

SECTION R21.6 SPECIAL MOMENT FRAMES CONSTRUCTED USING PRECAST CONCRETE

The detailing provisions in 21.6.1 and 21.6.2 are intended to produce frames that respond to design displacements essentially like monolithic special moment frames.

Precast frame systems composed of concrete elements with ductile connections are expected to experience flexural yielding in connection regions. Reinforcement in ductile connections can be made continuous by using Type 2 mechanical splices or any other technique that provides development in tension or compression of at least 125 percent of the specified yield strength f_y of bars and the specified tensile strength of bars.[21.31,21.32,21.33,21.34] Requirements for mechanical splices are in addition to those in 21.2.6 and are intended to avoid strain concentrations over a short length of reinforcement adjacent to a splice device. Additional requirements for shear strength are provided in 21.6.1 to prevent sliding on connection faces. Precast frames composed of elements with ductile connections may be designed to promote yielding at locations not adjacent to the joints. Therefore, design shear, V_e as computed according to 21.3.4.1 or 21.4.5.1, may be conservative.

Precast concrete frame systems composed of elements joined using strong connections are intended to experience flexural yielding outside the connections. Strong connections include the length of the coupler hardware as shown in Fig. R21.6.2. Capacity-design techniques are used in 21.6.2(b) to ensure the strong connection remains elastic following formation of plastic hinges. Additional column requirements are provided to avoid hinging and strength deterioration of column-to-column connections.

Strain concentrations have been observed to cause brittle fracture of reinforcing bars at the face of mechanical splices in laboratory tests of precast beam-column connection.[21.35] Designers should carefully select locations of strong connections or take other measures, such as debonding of reinforcing bars in highly stressed regions, to avoid strain concentrations that can result in premature fracture of reinforcement.

R21.6.3 Precast frame systems not satisfying the prescriptive requirements of Chapter 21 have been demonstrated in experimental studies to provide satisfactory seismic performance characteristic.[21.36,21.37] ACI TI.1 defines a protocol for establishing a design procedure, validated by analysis and laboratory tests, for such frames. The design procedure should identify the load path or mechanism by which the frame resists gravity and earthquake effects. The tests should be configured to test critical behaviors, and the measured quantities should establish upper-bound acceptance values for components of the load path, which may be in terms of limiting stresses, forces, strains, or other quantities. The design procedure used for the structure should not deviate from that used to design the test specimens, and acceptance values should not exceed values that were demonstrated by the tests to be acceptable. Materials and components used in the structure should be similar to those used in the tests. Deviations may be acceptable if the engineer can demonstrate that those deviations do not adversely affect the behavior of the framing system.

SECTION R21.7
SPECIAL REINFORCED CONCRETE STRUCTURAL WALLS AND COUPLING BEAMS

R21.7.1 **Scope.** This section contains requirements for the dimensions and details of special reinforced concrete structural walls and coupling beams. Provisions for diaphragms are in 21.9.

R21.7.2 **Reinforcement**. Minimum reinforcement requirements (see 21.7.2.1) follow from preceding SBC 304s. The uniform distribution requirement of the shear reinforcement is related to the intent to control the width of inclined cracks. The requirement for two layers of reinforcement in walls carrying substantial design shears (see 21.7.2.2) is based on the observation that, under ordinary construction conditions, the probability of maintaining a single layer of reinforcement near the middle of the wall section is quite low. Furthermore, presence of reinforcement close to the surface tends to inhibit fragmentation of the concrete in the event of severe cracking during an earthquake.

Because the actual forces in longitudinal reinforcing bars of stiff members may exceed the calculated forces, it is required (see 21.7.2.3) that all continuous reinforcement be developed fully.

R21.7.3 **Design forces**. Design shears for structural walls are obtained from lateral load analysis with the appropriate load factors. However, the designer should consider the possibility of yielding in components of such structures, as in the portion of a wall between two window openings, in which case the actual shear may be in excess of the shear indicated by lateral load analysis based on factored design forces.

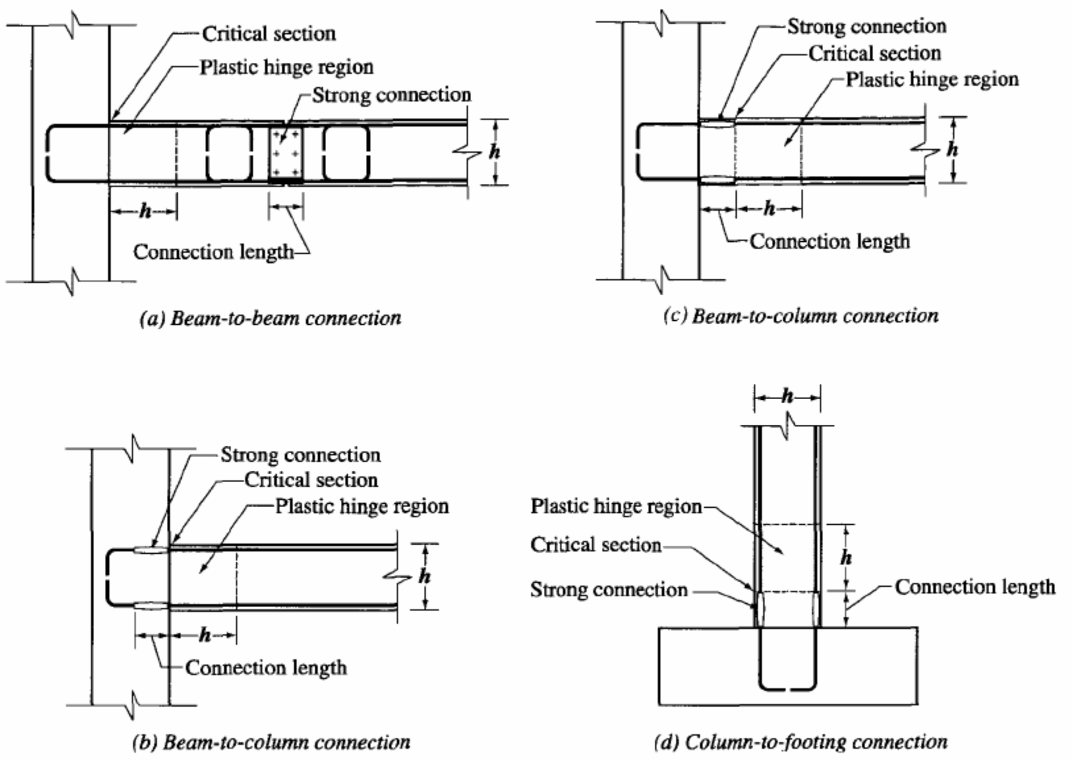


Fig. R21.6.2 - Strong connection examples

R21.7.4 **Shear strength.** Eq. (21-7) recognizes the higher shear strength of walls with high shear-to-moment ratios.[21.10,21.30,21.38] The nominal shear strength is given in terms of the net area of the section resisting shear. For a rectangular section without openings, the term A_{cv} refers to the gross area of the cross section rather than to the product of the width and the effective depth. The definition of A_{cv} in Eq. (21-7) facilitates design calculations for walls with uniformly distributed reinforcement and walls with openings.

A wall segment refers to a part of a wall bounded by openings or by an opening and an edge. Traditionally, a vertical wall segment bounded by two window openings has been referred to as a pier.

The ratio h_w / ℓ_w may refer to overall dimensions of a wall, or of a segment of the wall bounded by two openings, or an opening and an edge. The intent of 21.7.4.2 is to make certain that any segment of a wall is not assigned a unit strength larger than that for the whole wall. However, a wall segment with a ratio of h_w / ℓ_w higher than that of the entire wall should be proportioned for the unit strength associated with the ratio h_w / ℓ_w based on the dimensions for that segment.

To restrain the inclined cracks effectively, reinforcement included in ρ_n and ρ_v should be appropriately distributed along the length and height of the wall (see 21.7.4.3). Chord reinforcement provided near wall edges in concentrated amounts for resisting bending moment is not to be included in determining ρ_n and ρ_v within practical limits, shear reinforcement distribution should be uniform and at a small spacing.

If the factored shear force at a given level in a structure is resisted by several walls or several piers of a perforated wall, the average unit shear strength assumed for the total available cross-sectional area is limited to $2/3\sqrt{f_c'}$ with the additional requirement that the unit shear strength assigned to any single pier does not exceed $5/6\sqrt{f_c'}$. The upper limit of strength to be assigned to any one member is imposed to limit the degree of redistribution of shear force.

“Horizontal wall segments” in 21.7.4.5 refers to wall sections between two vertically aligned openings (see Fig. R21.7.4.5). It is, in effect, a pier rotated through 90 deg. A horizontal wall segment is also referred to as a coupling beam when the openings are aligned vertically over the building height.

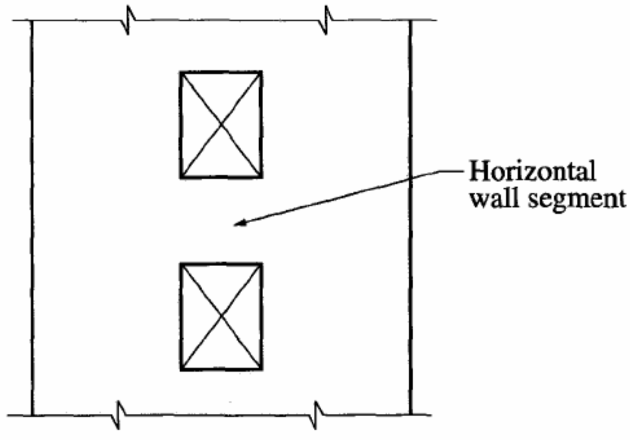


Fig. R21.7.4.5 - Wall with openings

R21.7.5 Design for flexure and axial loads

R21.7.5.1 Flexural strength of a wall or wall segmentis determined according to procedures commonly used for columns. Strength should be determined considering the applied axial and lateral forces. Reinforcement concentrated in boundary elements and distributed in flanges and webs should be included in the strength computations based on a strain compatibility analysis. The foundation supporting the wall should be designed to develop the wall boundary and web forces. For walls with openings, the influence of the opening or openings on flexural and shear strengths is to be considered and a load path around the opening or openings should be verified. Capacity design concepts and strut-and-tie models may be useful for this purpose.[21.39]

R21.7.5.2 Where wall sections intersect to form L-, T-, C-, or other cross-sectional shapes, the influence of the flange on the behavior of the wall should be considered by selecting appropriate flange widths. Tests [21.40] show that effective flange width increases with increasing drift level and the effectiveness of a flange in compression differs from that for a flange in tension. The value used for the effective compression flange width has little impact on the strength and deformation capacity of the wall; therefore, to simplify design, a single value of effective flange width based on an estimate of the effective tension flange width is used in both tension and compression. [21.40]

R21.7.6 Boundary elements of special reinforced concrete structural walls

R21.7.6.1 Two design approaches for evaluating detailing requirements at wall boundaries are included in 21.7.6.1. Section 21.7.6.2 allows the use of displacement-based design of walls, in which the structural details are determined directly on the basis of the expected lateral displacements of the wall. Requirements of 21.7.6.4 and 21.7.6.5 apply to structural walls designed by either 21.7.6.2 or 21.7.6.3.

R21.7.6.2 Section 21.7.6.2 is based on the assumption that inelastic response of the wall is dominated by flexural action at a critical, yielding section. The wall should be proportioned so that the critical section occurs where intended.

Eq. (21-8) follows from a displacement-based approach.[21.41,21.42] The approach assumes that special boundary elements are required to confine the concrete where the strain at the extreme compression fiber of the wall exceeds a critical value when the wall is displaced to the design displacement. The horizontal dimension of the special boundary element is intended to extend at least over the length where the compression strain exceeds the critical value. The height of the special boundary element is based on upper bound estimates of plastic hinge length and extends beyond the zone over which concrete spalling is likely to occur. The lower limit of 0.007 on the quantity δ_u / h_w requires moderate wall deformation capacity for stiff buildings.

The neutral axis depth c in Eq. (21-8) is the depth calculated according to 10.2, except the nonlinear strain requirements of 10.2.2 need not apply, corresponding to development of nominal flexural strength of the wall when displaced in the same direction as δ_u. The axial load is the factored axial load that is consistent with the design load combination that produces the displacement δ_u.

R21.7.6.3 By this procedure, the wall is considered to be acted on by gravity loads Wand 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.

R21.7.6.4 The value of $c/2$ in 21.7.6.4(a) is to provide a minimum length of the special boundary element. Where flanges are heavily stressed in compression, the web-to-flange interface is likely to be heavily stressed and may sustain local crushing failure unless special boundary element reinforcement extends into the web. Eq. (21-3) does not apply to walls.

Because horizontal reinforcement is likely to act as web reinforcement in walls requiring boundary elements, it should be fully anchored in boundary elements that act as flanges (21.7.6.4). Achievement of this anchorage is difficult when large transverse cracks occur in the boundary elements. Therefore, standard 90 deg hooks or mechanical anchorage schemes are recommended instead of straight bar development.

R21.7.6.5 Cyclic load reversals may lead to buckling of boundary longitudinal reinforcement even in cases where the demands on the boundary of the wall do not require special boundary elements. For walls with moderate amounts of boundary longitudinal reinforcement, ties are required to inhibit buckling. The longitudinal reinforcement ratio is intended to include only the reinforcement at the wall boundary as indicated in Fig. R21.7.6.5. A larger spacing of ties relative to 21.7.6.4(c) is allowed due to the lower deformation demands on the walls.

The addition of hooks or U-stirrups at the ends of horizontal wall reinforcement provides anchorage so that the reinforcement will be effective in resisting shear forces. It will also tend to inhibit the buckling of the vertical edge reinforcement. In walls with low in-plane shear, the development of horizontal reinforcement is not necessary.

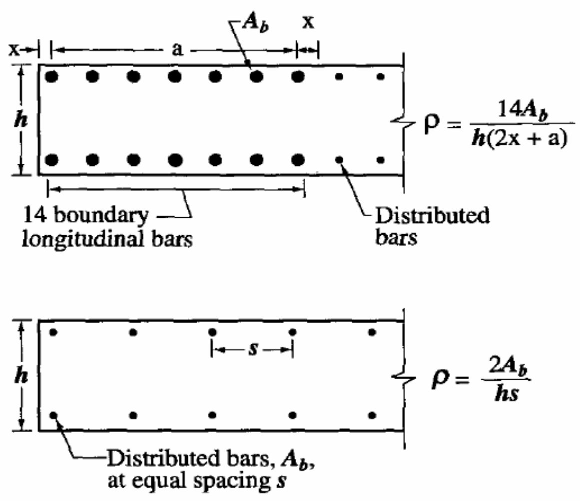


Fig. R21.7.6.5 - Longitudinal reinforcement ratios for typical wall boundary conditions

R21.7.7 **Coupling beams**. Coupling beams connecting structural walls can provide stiffness and energy dissipation. In many cases, geometric limits result in coupling beams that are deep in relation to their clear span. Deep coupling beams may be controlled by shear and may be susceptible to strength and stiffness deterioration under earthquake loading. Test results[21.43,21.44] 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 $\ell_n / h < 4$.

Each diagonal element consists of a cage of longitudinal and transverse reinforcement as shown in Fig. R21.7.7. The cage contains at least four longitudinal bars and confines a concrete core. The requirement on side dimensions of the cage and its core is to provide adequate toughness and stability to the cross section when the bars are loaded beyond yielding. The minimum dimensions and required reinforcement clearances may control the wall width.

When coupling beams are not used as part of the lateral force resisting system, the requirements for diagonal reinforcement may be waived. Nonprestressed coupling beams are permitted at locations where damage to these beams does not impair vertical load carrying capacity or egress ofthe structure, or integrity of the nonstructural components and their connections to the structure.

Tests in Reference 21.44 demonstrated that beams reinforced as described in Section 21.7.7 have adequate ductility at shear forces exceeding $(5/6)\sqrt{f_c'}b_w d$. Consequently, the use of a limit of $(5/6)\sqrt{f_c'}A_{cp}$ provides an acceptable upper limit.

When the diagonally oriented reinforcement is used, additional reinforcement in 21.7.7.4(f) is to contain the concrete outside the diagonal cores if the concrete is damaged byearthquake loading (Fig. R21.7.7).

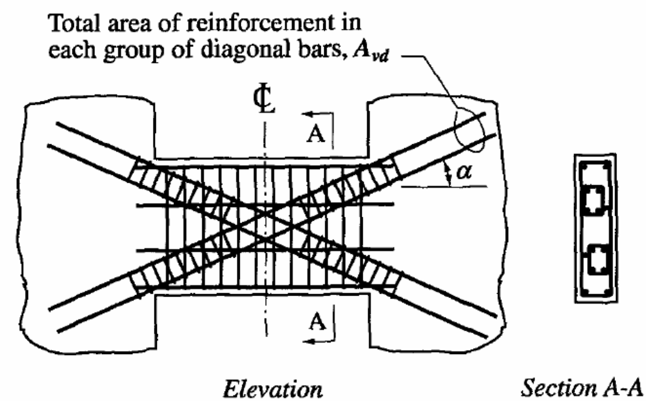


Fig. R21.7.7 - Coupling beam with diagonally oriented reinforcement

SECTION R21.9
STRUCTUREAL DIAPHRAGMS AND TRUSSES

R21.9.1 **Scope.** Diaphragms as used in building construction are structural elements (such as a floor or roof) that provide some or all of the following functions:

(a) Support for building elements (such as walls, partitions, and cladding) resisting horizontal forces but not acting as part of the building vertical lateral-force-resisting system;

(b) Transfer of lateral forces from the point of application to the building vertical lateral-force-resisting system;

(c) Connection of various components of the building vertical lateral-force-resisting system with appropriate strength, stiffness, and toughness so the building responds as intended in the design[21.45]

R21.9.2 **Cast-in-place composite-topping slab diaphragms.** A bonded topping slab is required so that the floor or roof system can provide restraint against slab buckling. Reinforcement is required to ensure the continuity of the shear transfer across precast joints. The connection requirements are introduced to promote a complete system with necessary shear transfers.

R21.9.3 **Cast-in-place topping slab diaphragms.** Composite action between the topping slab and the precast floor elements is not required, provided that the topping slab is designed to resist the design seismic forces.

R21.9.4 **Minimum thickness of diaphragms.** The minimum thickness of concrete diaphragms reflects current practice in joist and waffle systems and composite top ping slabs on precast floor and roof systems. Thicker slabs are required when the topping slab does not act compositely with the precast system to resist the design seismic forces.

R21.9.5 **Reinforcement.** Minimum reinforcement ratios for diaphragms correspond to the required amount of temperature and shrinkage reinforcement (7.12). The maximum spacing for web reinforcement is intended to control the width of inclined cracks. Minimum average prestress requirements (7.12.3) are considered to be adequate to limit the crack widths in post-tensioned floor systems; therefore, the maximum spacing requirements do not apply to these systems.

The minimum spacing requirement for welded wire fabric in topping slabs on precast floor systems (see 21.9.5.1) is to avoid fracture of the distributed reinforcement during an earthquake. Cracks in the topping slab open immediately above the boundary between the flanges of adjacent precast members, and the wires crossing those cracks are restrained by the transverse wire. [21.46] 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 250 mm for the transverse wires is required in 21.9.5.1 to reduce the likelihood of fracture of the wires crossing the critical cracks during a design earthquake. The minimum spacing requirements do not apply to diaphragms reinforced with individual bars, because strains are distributed over a longer length.

Compressive stress calculated for the factored forces on a 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 is assumed to indicate that integrity of the entire structure is dependent on the ability of that member to resist substantial compressive force under severe cyclic loading. Therefore, transverse reinforcement in 21.4.4 is required in such members to provide confinement for the concrete and the reinforcement (21.9.5.3).

The dimensions of typical structural diaphragms often preclude the use of transverse reinforcement along the chords. Reducing the calculated compressive stress by reducing the span of the diaphragm is considered to be a solution.

R21.9.7 **Shear strength.** The shear strength requirements for monolithic diaphragms, Eq. (21-10) in 21.9.7.1, are the same as those for slender structural walls. The term A_{cv} refers to the thickness times the width of the diaphragm. This corresponds to the gross area of the effective deep beam that forms the diaphragm. The shear reinforcement should be placed perpendicular to the span of the diaphragm.

The shear strength requirements for topping slab diaphragms are based on a shear friction model, and the contribution of the concrete to the nominal shear strength is not included in Eq. (21-9) for topping slabs placed over precast floor elements. Following typical construction practice, the topping slabs are roughened immediately above the boundary between the flanges of adjacent precast floor members to direct the paths of shrinkage cracks. As a result, critical sections of the diaphragm are cracked under service loads, and the contribution of the concrete to the shear capacity of the diaphragm may have already been reduced before the design earthquake occurs.

R21.9.8 **Boundary elements of structural diaphragms**. For structural diaphragms, the design moments are assumed to be resisted entirely by chord forces acting at opposite edges of the diaphragm. Reinforcement located at the edges of collectors should be fully developed for its yield strength. Adequate confinement of lap splices is also required. If chord reinforcement is located within a wall, the joint between the diaphragm and the wall should be provided with adequate shear strength to transfer the shear forces.

Section 21.9.8.3 is intended to reduce the possibility of chord buckling in the vicinity of splices and anchorage zones.

SECTION R21.10 FOUNDATIONS

R21.10.1 **Scope**. Requirements for foundations supporting buildings assigned to high seismic performance or design categories, represent a consensus of a minimum level of good practice in designing and detailing concrete foundations including piles, drilled piers, and caissons. It is desirable that inelastic response in strong ground shaking occurs above the foundations, as repairs to foundations can be extremely difficult and expensive.

R21.10.2 **Footings, foundation mats, and pile caps**

R21.10.2.2 Tests [21.47] have demonstrated that flexural members terminating in a footing, slab or beam (a T-joint) should have their hooks turned inwards toward the axis of the member for the joint to be able to resist the flexure in the member forming the stem of the T.

R21.10.2.3 Columns or boundary members supported close to the edge of the foundation, as often occurs near property lines, should be detailed to prevent an edge failure of the footing, pile cap, or mat.

R21.10.2.4 The purpose of 21.10.2.4 is to alert the designer to provide top reinforcement as well as other required reinforcement.

R21.10.2.5 In region of high seismicity, it is desirable to reinforce foundation, and basement wall.

R21.10.3 **Grade beams and slabs on grade.** For seismic conditions, slabs on grade (soil-supported slabs) are often part of the lateral-force-resisting system and should be designed in accordance with this SBC 304 as well as other appropriate standards or guidelines. See 1.1.6.

R21.10.3.2 Grade beams between pile caps or footings can be separate beams beneath the slab on grade or can be a thickened portion of the slab on grade. The cross-sectional limitation and minimum tie requirements provide reasonable proportions.

R21.10.3.3 Grade beams resisting seismic flexural stresses from column moments should have reinforcing details similar to the beams of the frame above the foundation.

R21.10.3.4 Slabs on grade often act as a diaphragm to hold the 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 grade should be adequately reinforced and detailed. The design drawings should clearly state that these slabs on grade are structural members so as to prohibit saw cutting of the slab.

R21.10.4 **Piles, piers, and caissons.** Adequate performance of piles and caissons for seismic loadings requires that these provisions be met in addition to other applicable standards or guidelines. See R1.1.5.

R21.10.4.2 A load path is necessary at pile caps to transfer tension forces from the reinforcing bars in the column or boundary member through the pile cap to the reinforcement of the pile or caisson.

R21.10.4.3 Grouted dowels in a blockout in the top of a precast concrete pile need to be developed, and testing is a practical means of demonstrating capacity. Alternatively, reinforcing bars can be cast in the upper portion of the pile, exposed by chipping of concrete and mechanically connected or welded to an extension.

R21.10.4.4 During earthquakes, piles can be subjected to extremely high flexural demands at points of discontinuity, especially just below the pile cap and near the base of a soft or loose soil deposit. The requirement for confinement reinforcement at the top of the pile is based on numerous failures observed at this location in recent earthquakes. Transverse reinforcement is required in this region to provide ductile performance. The designer should also consider possible inelastic action in the pile at abrupt changes in soil deposits, such as changes from soft to firm or loose to dense soil layers. Where precast piles are to be used, the potential for the pile tip to be driven to an elevation different than that specified in the drawings needs to be considered when detailing the pile. If the pile reaches refusal at a shallower depth, a longer length of pile will need to be cut off. If this possibility is not forseen, the length of transverse reinforcement required by 21.10.4.4 may not be available after the excess pile length is cut off.

R21.10.4.7 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.

SECTION R21.11
FRAME MEMBERS NOT PROPORTIONED TO RESIST FORCES INDUCED BY EARTHQUAKE MOTIONS

The detailing requirements for members that are part of the lateral-force resisting system assume that the members may undergo deformations that exceed the yield limit of the member without significant loss of strength. Members that are not part of the designated lateral-force-resisting system are not required to meet all the detailing requirements of members that are relied on to resist lateral forces. They should, however, be able to resist the gravity loads at lateral displacements corresponding to the design level prescribed by the governing SBC 304 for earthquake-resistant design. The design displacement is defined in 21.1.

Section 21.1 recognizes that actual displacements resulting from earthquake forces may be larger than the displacements calculated using the design forces and commonly used analysis models. Section 21.11.1 defines a nominal displacement for the purpose of prescribing detailing requirements. This section is consistent with the strength design approach of Ref. 21.2. Actual displacements may exceed the value of 21.11.1.

Section 21.11.2 prescribes detailing requirements intended to provide a system capable of sustaining gravity loads under moderate excursions into the inelastic range. Section 21.11.3 prescribes detailing requirements intended to provide a system capable of sustaining gravity loads under larger displacements.

Models used to determine design deflections of buildings should be chosen to produce results that conservatively bound the values expected during the design earthquake considering vertical, horizontal, and diaphragm systems as appropriate.

For gravity load factors, see R9.2.

The poor performance of some buildings with precast concrete gravity systems during the Northridge Earthquake was attributed to several factors addressed in 21.1 1.4. Columns should contain ties over their entire height, frame members not proportioned to resist earthquake forces should be tied together, and longer bearing lengths should be used to maintain integrity of the gravity system during shaking. The 50 mm increase in bearing length is based on an assumed 4 percent story drift ratio and 1.3 m beam depth, and is considered to be conservative for the ground motions expected in high seismic zones. In addition to the provisions of 21.11.4, precast frame members assumed not to contribute to lateral resistance should also satisfy 21.1 1.1 through 21.1 1.3.

SECTION R21.12
REQUIREMENTS FOR INTERMEDIATE MOMENT FRAMES

The objective of the requirements in 21.12.3 is to reduce the risk of failure in shear during an earthquake. The designer is given two options by which to determine the factored shear force.

According to option (a) of 21.12.3, the factored shear force is determined from the nominal moment strength of the member and the gravity load on it. Examples for a beam and a column are illustrated in Fig. R21.12.3.

To determine the maximum beam shear, it is assumed that its nominal moment strengths $\phi = 1.0$ are developed simultaneously at both ends of its clear span. A_s indicated in Fig. R21.12.3, the shear associated with this condition $[(M_{nl} + M_{nr}) / \ell_n]$ added algebraically to the effect of the factored gravity loads indicates the design shear for of the beam. For this example, both the dead load w_D and the live load w_L have been assumed to be uniformly distributed.

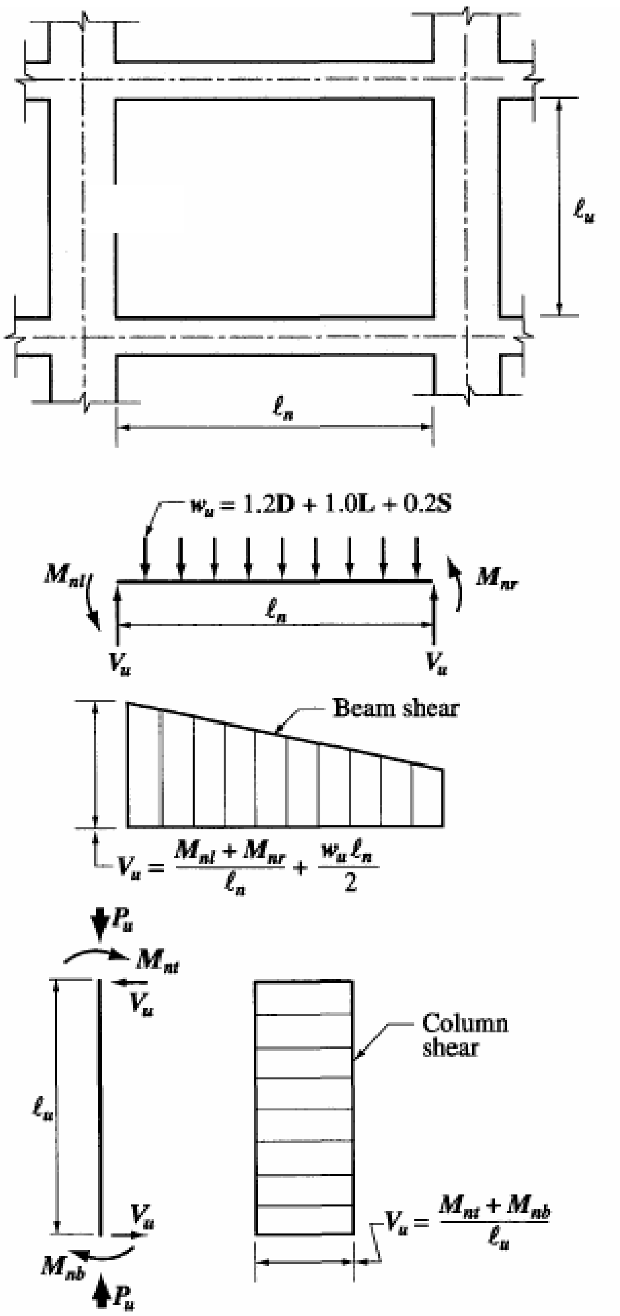


Fig. R21.12.3 - Design shears for frames in regions of moderate seismic risk (see 21.12)

Determination of the design shear for a column is also illustrated for a particular example in Fig. R21.12.3. The factored design axial load, P_u, should be chosen to develop the largest moment strength of the column.

In all applications of option (a) of 21.12.3, shears are required to be calculated for moment, acting clockwise and counter-clockwise. Fig. R21.12.3 demonstrates only one of the two conditions that are to be considered for every member. Option (b) bases V_u on the load combination including the earthquake effect, E, which should be doubled. For example, the load combination defined by Eq. (9-5) would be:

$$U = 1.2D + 2.0E + 1.0L$$

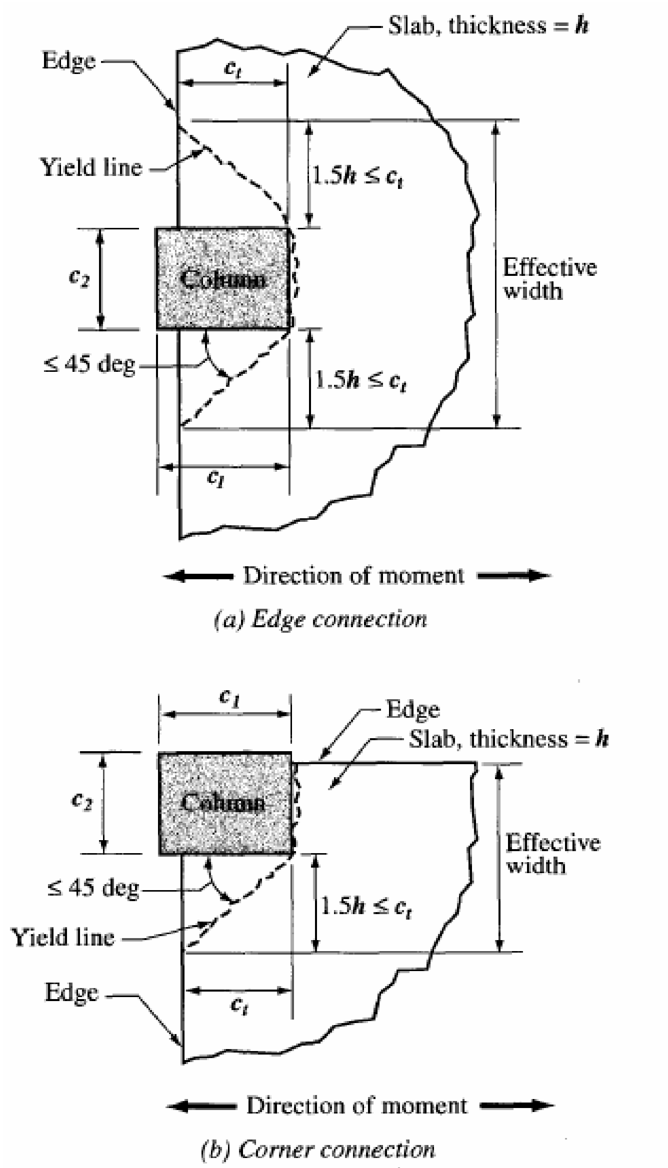
where E is the value specified by the governing SBC 304. Section 21.12.4 contains requirements for providing beams with a threshold level of toughness. Transverse reinforcement at the ends of the beam shall be hoops. In most cases, stirrups required by 21.12.3 for design shear force will be more than those required by 21.12.4. Requirements of 21.12.5 serve the same purpose for columns.

Section 21.12.6 applies to two-way slabs without beams, such as flat plates.

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_s refers, for a given design load combination with E acting in one horizontal direction, to that portion of the factored slab moment that is balanced by the supporting members at a joint. It is not necessarily equal to the total design moment at support for a load combination including earthquake effect. In accordance with 13.5.3.2, only a fraction $(\gamma_f M_s)$ of the moment M_s is assigned to the slab effective width. For edge and corner connections, flexural reinforcement perpendicular to the edge is not considered fully effective unless it is placed within the effective slab width.[21.48,49] See Fig. 21.12.6.1.

Application of the provisions of 21.12.6 are illustrated in Fig. R21.12.6.2 and Fig. R21.12.6.3.



Fig, R21.12.6.1 - Effective width for reinforcement placement in edge and corner connections

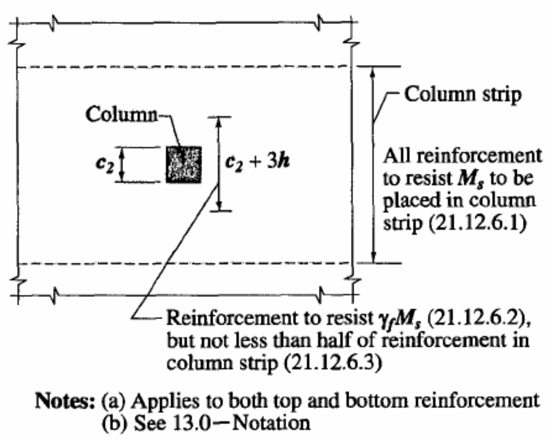


Fig. R21.12.6.2 - Location of reinforcement in slabs

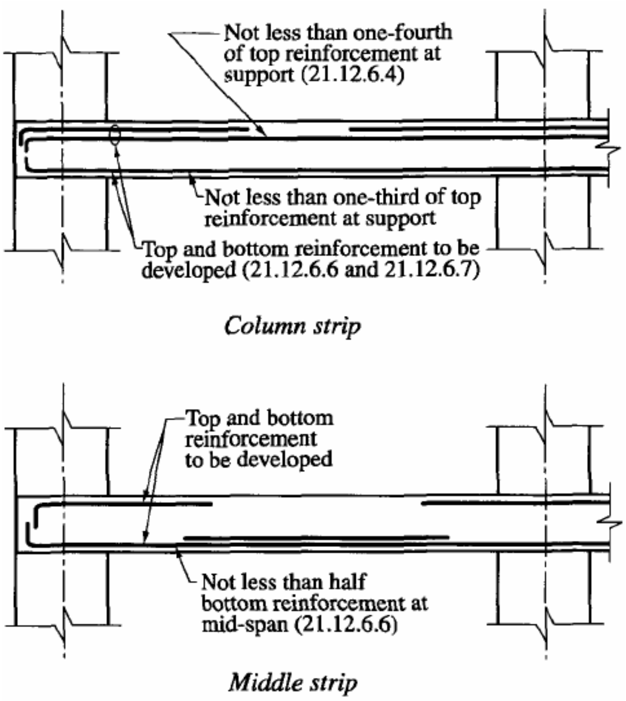


Fig. R21.12.6.3 - Arrangement of reinforcement in slabs

R21.12.6.8 The requirements apply to two-way slabs that are part of the primary lateral force-resisting system. Slab-column connections in laboratory tests [21.49] exhibited reduced lateral displacement ductility when the shear at the column connection exceeded the recommended limit.

SECTION R21.13
INTERMEDIATE PRECAST STRUCTURAL WALLS

Connections between precast wall panels or between wall panels and the foundation are required to resist forces induced by earthquake motions and to provide for yielding in the vicinity of connections. When Type 2 mechanical splices are used to directly connect primary reinforcement, the probable strength of the splice should be at least 1-1/2 times the specified yield strength of the reinforcement.

COMMENTARY REFERENCES

REFERENCES, CHAPTER 1

1.1 ACI Committee 307, "Standard Practice for the Design and Construction of Cast-in-Place Reinforced Concrete Chimneys (ACI307-98)," American Concrete Institute, Farmington Hills, MI, 1998,32 pp. Also *ACI Manual of Concrete Practice.*

1.2 ACI Committee 313, "Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials (ACI3 13-97)," American Concrete Institute, Farmington Hills, MI, 1997, 22 pp. Also *ACI Manual of Concrete Practice.*

1.3 ACI Committee 350, "Environmental Engineering Concrete Structures (ACI 350R-89)," American Concrete Institute, Farming- ton Hills, MI, 1989,20pp. Also *ACI Manual of Concrete Practice.*

1.4 ACI Committee 349, "Code Requirements for Nuclear Safety Related Concrete Structures (ACI349-97)," American Concrete Institute, Farmington Hills, MI, 1997, 129 pp., plus 1997 Supplement. Also *ACI Manual of Concrete Practice.*

1.5 ACI-ASME Committee 359, "Code for Concrete Reactor Vessels and Containments (ACI 359-92)," American Concrete Institute, Farmington Hills, MI, 1992.

1.6 ACI Committee 543, "Recommendations for Design, Manufacture, and Installation of Concrete Piles, (ACI 543R-74) (Reapproved 1980)," ACI JOURNAL , *Proceedings* V. 71, No. 10, Oct.1974, pp. 477-492.

1.7 ACI Committee 336, "Design and Construction of Drilled Piers (ACI 336.3R-93)," American Concrete Institute, Farmington Hills, MI, 1993,30 pp. Also *ACI Manual of Concrete Practice.*

1.8 "Recommended Practice for Design, Manufacture and Installation of Pre-stressed Concrete Piling," *PCI Journal,* V. 38, No. 2, Mar.-Apr. 1993, pp. 14-41.

1.9 ACI Committee 311, "Guide for Concrete Inspection (ACI31 1.4R-93," American Concrete Institute, Farmington Hills, MI,1995, 11 pp. Also *ACI Manual* of *Concrete Practice.*

1.10 ACI Committee 3 11, *ACI Manual of Concrete Inspection,*SP-2, 8th Edition, American Concrete Institute, Farmington Hills, MI, 1992,200 pp.

REFERENCES, CHAPTER 2

2.1 ACI Committee 116, "Cement and Concrete Terminology (ACI 116R-90)," American Concrete Institute, Farmington Hills, MI, 1990,58 pp. Also *ACI Manual of Concrete Practice.*

REFERENCES, CHAPTER 3

3.1 ACI Committee 214, "Recommended Practice for Evaluation of Strength Test Results of Concrete (ACI 214-77) (Reapproved 1989)," (ANSVACI 214-77), American Concrete Institute, Farmington Hills, MI, 1977, 14 pp. Also *ACI Manual of Concrete Practice.*

3.2 ACI Committee 223, "Standard Practice for the Use of Shrinkage-Compensating Concrete (ACI 223-98)," American Concrete Institute, Farmington Hills, MI, 29 pp. Also *ACI Manual of Concrete Practice.*

REFERENCES, CHAPTER 4

4.1 ASTM C 1012-89,"Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution," *ASTM Book of Standards,* Part 04.01, ASTM, West Conshohocken, PA, 5 pp.

4.2 ACI Committee 201, "Guide to Durable Concrete (ACI201.2R-92)," American Concrete Institute, Farmington Hills, MI,1992,39 pp. Also *ACI Manual of Concrete Practice.*

4.3 ACI Committee 222, "Corrosion of Metals in Concrete (ACI222R-96)," American Concrete Institute, Farmington Hills, MI,1996, 30 pp. Also *ACI Manual of Concrete Practice.*

4.4 Ozyildirim, *C.,* and Halstead, W., "Resistance to Chloride Ion Penetration of Concretes Containing Fly Ash, Silica Fume, or Slag,. *Permeability of Concrete,* SP-108, American Concrete Institute, Farmington Hills, MI, 1988, pp. 35-61.

REFERENCES, CHAPTER 5

5.1 ACI Committee 21 1, "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete (ACI21 1.1-98)," American Concrete Institute, Farmington Hills, MI, 1998, 38 pp. Also *ACI Manual of Concrete Practice.*

5.2 ACI Committee 21 1, "Standard Practice for Selecting Proportions for Structural Lightweight Concrete (ACI 21 1.2-91)," American Concrete Institute, Farmington Hills, M1, 1991, 18 pp. Also, *ACI Manual of Concrete Practice.*

5.3 Municipality of Riyadh, "Quality of ready-mixed concrete production in Riyadh, Internal Report. Quality scheme for ready-mixed concrete plants, Riyadh, 2005.

5.4 Al-Negheimish, A and Alhozaimy, A., "Assessment of production control by ready-mixed concrete plants in Riyadh", submitted to Journal of King Saud University, Engineering Sciences, 2007.

5.5 ASTM C 1077-92, "Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation," ASTM, West Conshohocken, PA, *5* pp.

5.6 ASTM D 3665-99, "Standard Practice for Random Sampling of Construction Materials," ASTM, West Conshohocken, PA, 5 pp.

5.7 Bloem, D. L., "Concrete Strength Measurement-Cores vs. Cylinders," *Proceedings,* ASTM, V. 65, 1965, pp. 668-696.

5.8 Bloem, Delmar L., "Concrete Strength in Structures," ACI JOURNAL, *Proceedings* V. 65, No. 3, Mar. 1968, pp. 176-187.

5.9 Malhotra, V. M., *Testing Hardened Concrete: Nondestructive Methods,* ACI Monograph No. 9, American Concrete Institute. Iowa State University Press, Farmington Hills, MI, 1976, 188 pp.

5.10 Malhotra, V. M., "Contract Strength Requirements-Cores Versus In Situ Evaluation," ACI JOURNAL, Proceedings V. 74, No. 4, Apr. 1977, pp. 163-172.

5.11 Bartlett, M.F., and Macgregor, J.G., "Effect of Moisture Condition on Concrete Core Strengths", *ACI Materials Journal,* V. 91, No. 3, May-June, 1994, pp. 227-236.

5.12 ACI Committee 304, "Guide for Measuring, Mixing, Transporting, and Placing Concrete (ACI304R-89)", American Concrete Institute, Farmington Hills, MI, 1989, 49 pp. Also *ACI Manual of Concrete Practice.*

5.13 Newlon, H., Jr., and 0201, A., "Delayed Expansion of Concrete Delivered by Pumping through Aluminum Pipe Line", *Concrete Case Study* No. 20; Virginia Highway Research Council, Oct.1969, 39 pp.

5.14 ACI Committee 309, "Guide for Consolidation of Concrete (ACI 309R-96)", American Concrete Institute, Farmington Hills, MI, 1996, 40 pp. Also *ACI Manual of Concrete Practice.*

5.15 ACI Committee 308, "Standard Practice for Curing Concrete (ACI 308-92)", American Concrete Institute, Farmington Hills, MI, 1992, 11 pp. Also *ACI Manual of Concrete Practice.*

5.16 ACI Committee 305, "Hot Weather Concreting (ACI 305R- 91)", American Concrete Institute, Farmington Hills, MI, 1991, 17 pp. Also *ACI Manual of Concrete Practice.*

REFERENCES, CHAPTER 6

6.1 ACI Committee 347, "Guide to Formwork for Concrete (ACI 347R-94), " American Concrete Institute, Farmington Hills, MI, 1994, 33 pp. Also *ACI Manual of Concrete Practice.*

6.2 Hurd, M. K., and ACI Committee 347, *Formwork for Concrete,* SP-4, 5th Edition, American Concrete Institute, Farmington Hills, MI, 1989, 475 pp.

6.3 Liu, X. L.; Lee, H. M.; and Chen, W. F., "Shoring and Re-shoring of High-Rise Buildings", *Concrete International,* V. 10, No. 1, Jan. 1989, pp. 64-68.

6.4 ASTM C 873-99, "Standard Test Method for Compressive Strength of Concrete Cylinders Cast-in-Place in Cylindrical Molds", ASTM, West Conshohocken, PA, 4 pp.

6.5 ASTM C 803K 803M-97, "Test Method for Penetration Resistance of Hardened Concrete", ASTM, West Conshohocken, PA, 4 pp.

6.6 ASTM C 900, "Standard Test Method for Pullout Strength of Hardened Concrete", ASTM, West Conshohocken, PA, 5 pp.

6.7 ASTM C 1074-87, "Estimating Concrete Strength by the Maturity Method", ASTM, West Conshohocken, PA.

REFERENCES, CHAPTER 7

7.1 ACI Committee 315, *ACI Detailing Manual-I994,* SP-66, American Concrete Institute, Farmington Hills, MI, 1994, 244 pp. Also "Details and Detailing of Concrete Reinforcement (ACI 315-92)," and "Manual of Engineering and Placing Drawings for Reinforced Structures (ACI 3 15R-94)." Also *ACI Manual of Concrete Practice.*

7.2 Black, William C., "Field Corrections to Partially Embedded Reinforcing Bars," ACI JOURNAL , *Proceedings* V. 70, No. 10, Oct. 1973, pp. 690-691.

7.3 Stecich, J.; Hanson, J. M.; and Rice, P. F.; "Bending and Straightening of Grade 60 Reinforcing Bars," *Concrete International: Design & Construction,* V. 6, No. 8, Aug. 1984, pp. 14-23.

7.4 Sason, A. S. "Evaluation of Degree of Rusting on Prestressed Concrete Strand," *PCI Journal,* V. 37, No. 3, May-June 1992,pp. 25-30.

7.5 ACI Committee 117, "Standard Tolerances for Concrete Construction and Materials (ACI 117-90)," American Concrete Institute, Farmington Hills, MI, 22 pp. Also *ACI Manual of Concrete Practice.*

7.6 *PCI Design Handbook: Precast and Prestressed Concrete,* 4th Edition, Pre-cast/Pre-stressed Concrete Institute, Chicago, 1992, 580 pp.

7.7 ACI Committee 408, "Bond Stress-The State of the Art," ACI JOURNAL, *Proceedings* V. 63, No. 11, Nov. 1966, pp. 1161-1188.

7.8 Deatherage, J. H., Burdette, E. G. and Chew, C. K., "Development Length and Lateral Spacing Requirements of Prestressing Strand for Pre-stressed Concrete Bridge Girders," *PCI Journal,* V.39, No. 1, Jan.-Feb. 1994, pp. 70-83.

7.9 Russell, B. W., and Bums, N. H. “Measured Transfer Lengths of 0.5 and 0.6 in. Strands in Pre-tensioned Concrete,” *PCI Journal,* V. 41, No. 5, Sept.-Oct. 1996, pp. 44-65.

7.10 ACI Committee 362, “Design of Parking Structures (ACI 362.1R-97),” American Concrete Institute, Farmington Hills, MI, 1997,40 pp.

7.11 Hanson, N. W., and Conner, H. W., “Seismic Resistance of Reinforced Concrete Beam-Column Joints,” *Proceedings,* ASCE, V. 93, ST5, Oct. 1967, pp. 533-560.

7.12 ACI-ASCE Committee 352, “Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI352R-91),” American Concrete Institute, Fannington Hills, MI, 1991, 18 pp. Also *ACI Manual of Concrete Practice.*

7.13 Pfister, J. F., “Influence of Ties on the Behavior of Reinforced Concrete Columns,” ACIJOURNAL, *Proceedings* V. 61, No. *5,* May 1964, pp. 521-537.

7.14 Gilbert, R. I., “Shrinkage Cracking in Fully Restrained Concrete Members,” *ACI Structural Journal,* V. 89, No. 2, Mar.-Apr.1992, pp. 141-149.

7.15 "Design and typical Details of Connections for Precast and Pre-stresses Concrete", MNL-123-88, Precast/Prestressed Concrete Institute, Chicago, 1988, 270 pp.

7.16 PCI Building Code Committee, “Proposed Design Requirements for Precast Concrete,” *PCI Journal,* V. 31, No. 6, Nov.-Dec.1986, pp. 32-47.

REFERENCES, CHAPTER 8

8.1 Fintel, M.; Ghosh, S. K.; and Iyengar, H., *Column Shortening in Tall Buildings-Prediction and Compensation,* EB 108D, Portland Cement Association, Skokie, Ill., 1986, 34 pp.

8.2 Cohn, M. Z., “Rotational Compatibility in the Limit Design of Reinforced Concrete Continuous Beams,” *Flexural Mechanic of Reinforced Concrete,* SP-12, American Concrete/Institute American Society of Civil Engineers, Farmington Hills, MI, 1965, pp.359-382.

8.3 Mattock, A. H., “Redistribution of Design Bending Moments in Reinforced Concrete Continuous Beams,” *Proceedings,* Institution of Civil Engineers (London), V. 13, 1959, pp. 35-46.

8.4 Mast, R.F., “Unified Design Provision for Reinforced and Pre-stressed Concrete Flexural and Compression Members,” *ACI Structural Journal,* V. 89, No. 2, Mar.-Apr., 1992, pp. 185-199.

8.5 Pauw, Adrian, “Static Modulus of Elasticity of Concrete as Affected by Density,” ACI JOURNAL, *Proceedings* V. 57, No. 6, Dec.1960, pp. 679-687.

8.6 ASTM C 469-94, “Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression,” ASTM, West Conshohocken, PA.

8.7 “Handbook of Frame Constants,” Portland Cement Association, Skokie, IL, 1972,34 pp.

8.8 “Continuity in Concrete Building Frames,” Portland Cement Association, Skokie, IL, 1959,56pp.

REFERENCES, CHAPTER 9

9.1 MacGregor, J. G., “Safety and Limit States Design for Reinforced Concrete,” *Canadian Journal of Civil Engineering,* V. 3, No. 4, Dec. 1976, pp. 484-513.

9.2 Winter, G., “Safety and Serviceability Provisions in the ACI Building Code,” *Concrete Design: US. and European Practices,* SP-59, American Concrete Institute, Farmington Hills, MI, 1979,pp. 35-49.

9.3 *Deflections* of *Concrete Structures,* SP-43, American Concrete Institute, Farmington Hills, MI, 1974,637 pp.

9.4 ACI Committee 213, "Guide for Structural Lightweight Aggregate Concrete (ACI 2 13R-87)," American Concrete Institute, Farmington Hills, MI, 1987, 27 pp. Also *ACI Manual of Concrete Practice.*

9.5 Branson, D. E., "Instantaneous and Time-Dependent Deflections on Simple and Continuous Reinforced Concrete Beams," *HPR Report* No. 7, Part 1, Alabama Highway Department, Bureau of Public Roads, Aug. 1965, pp. 1-78.

9.6 ACI Committee 435, "Deflections of Reinforced Concrete Flexural Members (ACI 435.2R-66) (Reapproved 1989)," *ACI JOURNAL, Proceedings* V. 63, No. 6, June 1966, pp. 637-674. Also *ACI Manual of Concrete Practice.*

9.7 Subcommittee 1, ACI Committee 435, "Allowable Deflections (ACI 435.3R-68) (Reapproved 1989)," *ACI JOURNAL*, *Proceedings* V. 65, No. 6, June 1968, pp. 433-444. Also *ACI Manual of Concrete Practice.*

9.8 Subcommittee 2, ACI Committee 209, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures (ACI 209R-92)," *Designing for the Effects of Creep, Shrinkage, and Temperature in Concrete Structures,* SP-27, American Concrete Institute, Farmington Hills, MI, 1971, pp. 51-93.

9.9 ACI Committee 435, "Deflections of Continuous Concrete Beams (ACI 435.5R-73)(Reapproved 1989)," American Concrete Institute, Farmington Hills, MI, 1973, 7 pp. Also *ACI Manual of Concrete Practice.*

9.10 ACI Committee 435, "Proposed Revisions by Committee 435 to ACI Building Code and Commentary, Provisions on Deflections," *ACI JOURNAL ,Proceedings* V. 75, No. 6, June 1978, pp. 229-238.

9.11 Branson, D. E., "Compression Steel Effect on Long-Time Deflections," *ACI JOURNAL, Proceedings* V. 68, No. 8, Aug. 1971, pp. 555-559.

9.12 Branson, D. E., *Deformation of Concrete Structures,* McGraw-Hill Book Co., New York, 1977,546 pp.

9.13 *PCI Design Handbook -Precast and Pre-stressed Concrete,* 5th Edition, Pre-cast\Pre-stressed Concrete Institute, Chicago, IL, 1998, pp. 4-68 to 4-72.

9.14 Mast, R.F., "Analysis of Cracked Pre-stressed Concrete Sections: A Practical Approach," *PCI Journal,* V. 43, No. 4, July-Aug., 1998, pp. 80-91.

9.15 Shaikh, A. F., and Branson, D. E., "Non-Tensioned Steel in Pre-stressed Concrete Beams," *Journal of the Pre-stressed Concrete Institute,* V. 15, No. 1, Feb. 1970, pp. 14-36.

9.16 Branson, D. E., discussion of "Proposed Revision of ACI 318-63: Building Code Requirements for Reinforced Concrete,'' by ACI Committee 318, *ACI JOURNAL*, *Proceedings* V. 67, No. 9, Sept. 1970, pp. 692-695.

9.17 Subcommittee *5,* ACI Committee 435, "Deflections of Pre-stressed Concrete Members (ACI 435.1R-63)(Reapproved 1989);' *ACI JOURNAL, Proceedings* V. 60, No. 12, Dec. 1963, pp. 1697-1728. Also *ACI Manual of Concrete Practice,* Part 4.

9.18 Branson, D. E.; Meyers, B. L.; and Knpanarayanan, K. M., "Time-Dependent Deformation of Non-composite and Composite Pre-stressed Concrete Structures," *Symposium on Concrete Deformation,* Highway Research Record 324, Highway Research Board,1970, pp. 15-43.

9.19 Ghali, A., and Favre, R., *Concrete Structures: Stresses and Deformations,* Chapman and Hall, New York, 1986,348 pp.

REFERENCES, CHAPTER 10

10.1 Nedderman, H., "Flexural Stress Distribution in Extra High Strength Concrete," MS thesis, University of Texas at Arlington,1973.

10.2 Karr, P. H.; Hanson, N. W; and Capell, H. T.; "Stress- train Characteristics of High Strength Concrete", *Douglas McHenry International Symposium on Concrete and Concrete Structures,* SP-*55,* American Concrete Institute, Farmington Hills, MI, 1978, pp.161-1*85.*

10.3 Mattock, A. H.; Kriz, L. B.; and Hognestad, E., "Rectangular Concrete Stress Distribution in Ultimate Strength Design," *ACI JOURNAL, Proceedings* V. 57, No. 8, Feb. 1961, pp. 875-928.

10.4 *ACI Design Handbook, Vol. 2-Colurnns,* SP-l7A(90), American Concrete Institute, Farmington Hills, MI, 1990, pp. 161-163 and 207-221.

10.5 *CRSI Handbook,* 7th Edition, Concrete Reinforcing Steel Institute, Schaumberg, IL, 1992, 840 pp.

10.6 Bresler, B., "Design Criteria for Reinforced Concrete Columns under Axial Load and Biaxial Bending," *ACI JOURNAL ,Proceedings* V. 57, No. *5,* Nov. 1960, pp. 481-490.

10.7 Parme, A. L.; Nieves, J. M.; and Gouwens, A., "Capacity of Reinforced Rectangular Columns Subjected to Biaxial Bending," *ACI JOURNAL, Proceedings* V. 63, No. 9, Sept. 1966, pp. 911-923.

10.8 Heimdahl, P. D., and Bianchini, A. C., "Ultimate Strength of Bi-axially Eccentrically Loaded Concrete Columns Reinforced with High Strength Steel," *Reinforced Concrete Columns,* SP-50, American Concrete Institute, Farmington Hills, MI, 1975, pp. 100-101.

10.9 Furlong, R. W., "Concrete Columns Under Bi-axially concentric Thrust," *ACI JOURNAL*, *Proceedings* V. 76, No. 10, Oct. 1979, pp. 1093-11 18.

10.10 Hansell, W., and Winter, G., "Lateral Stability of Reinforced Concrete Beams," *ACI JOURNAL*, *Proceedings* V. 56, No. 3, Sept. 1959,pp. 193-214.

10.11 Sant, J. K., and Bletzacker, R. W., "Experimental Study of Lateral Stability of Reinforced Concrete Beams," *ACI JOURNAL*, *Proceedings* V. 58, No. 6, Dec. 1961, pp. 713-736.

10.12 Gergeiy, P., and Lutz, L. A., "Maximum Crack Width in Reinforced Concrete Flexural Members," *Causes, Mechanism, and Control of Cracking in Concrete,* SP-20, American Concrete Institute, Farmington Hills, MI, 1968, pp. 87-1 17.

10.13 Kaar, P. H., "High Strength Bars as Concrete Reinforcement, Part 8: Similitude in Flexural Cracking of T-Beam Flanges," *Journal,* PCA Research and Development Laboratories, V. 8, No.2, May 1966, pp. 2-12.

10.14 Base, G. D.; Reed, J. B.; Beeby, A. W.; and Taylor, H. P. J., "An Investigation of the Crack Control Characteristics of Various Types of Bar in Reinforced Concrete Beams," *Research Report* No. 18, Cement and Concrete Association, London, Dec. 1966,44 pp.

10.15 Beeby, A. W., "The Prediction of Crack Widths in Hardened Concrete," *The Structural Engineer,* V. 57A, No. 1,Jan. 1979,pp. 9-17.

10.16 Frosch, R. J., "Another Look at Cracking and Crack Control in Reinforced Concrete," *ACI Structural Journal,* V. 96, No. 3, May-June 1999, pp. 437-442.

10.17 ACI Committee 3 18, "Closure to Public Comments on ACI 318-99," *Concrete International,* May 1999, pp. 318-1 to 3 18-50.

10.18 Darwin, D., et al., "Debate: Crack Width, Cover, and Corrosion," *Concrete International,* V. 7, No. *5,* May 1985, American Concrete Institute, Farmington Hills, MI, pp. 20-35.

10.19 Oesterle, R. G., "The Role of Concrete Cover in Crack Control Criteria and Corrosion Protection," RD Serial No. 2054, Portland Cement Association, Skokie, IL, 1997.

10.20 Chow, L.; Conway, H.; and Winter, G., "Stresses in Deep Beams," *Transactions,* ASCE, V. 118, 1953, pp. 686-708.

10.21 "Design of Deep Girders," IS079D, Portland Cement Association, Skokie, IL, 1946, 10 pp.

10.22 Park, R., and Paulay, T., *Reinforced Concrete Structures,* Wiley-Inter-Science, New York, 1975,769 pp.

10.23 Furlong, R. W., "Column Slenderness and Charts for Design*'', ACI JOURNAL, Proceedings* V. 68, No. l, Jan. 197l, pp. 9-18.

10.24 "Reinforced Concrete Column Investigation-Tentative Final Report of Committee 105," *ACI JOURNAL*, *Proceedings* V. 29, No. *5,* Feb. 1933, pp. 275-282.

10.25 MacGregor, J. G., "Design of Slender Concrete Columns- Revisited," *ACI Structural Journal,* V. 90, No. 3, May-June 1993, pp. 302-309.

10.26 MacGregor, J. G.; Breen, J. E.; and Pfrang, E. O., "Design of Slender Concrete Columns," *ACI JOURNAL*, *Proceedings* V. 67, NO.1, Jan. 1970, pp. 6-28.

10.27 Ford, J. S.; Chang, D. C.; and Breen, J. E., "Design Indications from Tests of Un-braced Multipanel Concrete Frames," *Concrete International: Design and Construction,* V. 3, NO. 3, Mar. 1981, pp. 37-47.

10.28 MacGregor, J. G., and Hage, S. E., "Stability Analysis and Design Concrete," *Proceedings,* ASCE, V. 103, NO. ST 10, Oct. 1977.

10.29 Grossman, J. S., "Slender Concrete Structures-The New *Edge,"ACI Structural Journal,* V. 87, No. 1,Jan.-Feb. 1990,pp. 39-52.

10.30 Grossman, J. S., "Reinforced Concrete Design," *Building Structural Design Handbook,* R. N. White and C. G. Salmon, eds., John Wiley and Sons, New York, 1987.

10.31 "Guide to Design Criteria for Metal Compression Members," 2nd Edition, Column Research Council, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Penn., 1966.

10.32 ACI Committee340, *Design Handbook in Accordance with the Strength Design Method of ACI 318-77, V. 2-Columns,*SP-l7A(78), American Concrete Institute, Farmington Hills, M1,1978,228 pp.

10.33 "Code of Practice for the Structural Use of Concrete, Part 1. Design Materials and Workmanship," CP110: Part 1, British Standards Institution, London, Nov. 1972, 154 pp.

10.34 Cranston, W. B., "Analysis and Design of Reinforced Concrete Columns," *Research Report* No. 20, Paper 41.020, Cement and Concrete Association, London, 1972,54 pp.

10.35 Mirza, S. A.; Lee, P. M.; and Morgan, D. L, "ACI Stability Resistance Factor for RC Columns," *ASCE Structural Engineering,* American Society of Civil Engineers, V. 113, No. 9, Sept. 1987, pp. 1963-1976.

10.36 Mirza, S. A., "Flexural Stiffness of Rectangular Reinforced Concrete Columns," *ACI Structural Journal,* V. 87, No. 4, July- Aug. 1990, pp. 425-435.

10.37 Lai, S. M. A., and Macgregor, J. G., "Geometric Nonlinearities in Un-braced Multistory Frames," *ASCE Structural Engineering,* American Society of Civil Engineers, V. 109, No. 11, Nov.1983, pp. 2528-2545.

10.38 Bianchini, A. C.; Woods, Robert E.; and Kesler, C. E., "Effect of Floor Concrete Strength on Column Strength," *ACI JOURNAL, Proceedings* V. 56, No. 11, May 1960, pp. 1149-1169.

10.39 Ospina, C. E., and Alexander, S. D. B., "Transmission of Interior Concrete Column Loads through Floors," *ASCE Journal of Structural Engineering,* V. 124, No. 6., 1998.

10.40 Everard, N. J., and Cohen, E., "Ultimate Strength Design of Reinforced Concrete Columns," SP-7, American Concrete Institute, Farmington Hills, MI, 1964, 182 pp.

10.41 Hawkins, N. M., "Bearing Strength of Concrete Loaded through Rigid Plates," *Magazine of Concrete Research* (London), V. 20, NO.62, Mar. 1968, pp. 31-40.

REFERENCES, CHAPTER 11

11.1 ACI-ASCE Committee 426, "Shear Strength of Reinforced Concrete Members (ACI 426R-74) (Reapproved 1980)," *Proceedings,* ASCE, V. 99, No. ST6, June 1973, pp. 1148-1157.

11.2 Macgregor, J. G., and Hanson, J. M., "Proposed Changes in Shear Provisions for Reinforced and Pre-stressed Concrete Beams," *ACI JOURNAL*, *Proceedings* V. 66, No. 4, Apr. 1969, pp. 276-288.

11.3 ACI-ASCE Committee 326 (now 426), "Shear and Diagonal Tension," *ACI JOURNAL, Proceedings* V. 59, No.1,Jan. 1962, pp. 1-30; No. 2, Feb. 1962,pp. 277-334; and No. 3, Mar. 1962,pp. 352-396.

11.4 Barney, G. B.; Corley, W. G.; Hanson, J. M.; and Parmelee, R. A., "Behavior and Design of Pre-stressed Concrete Beams with Large Web Openings," *Journal of the Pre-stressed Concrete Institute,* V. 22, No. 6, Nov.-Dec. 1977, pp. 32-61.

11.5 Schlaich, J.; Schafer, K.; and Jennewein, M., "Toward a Consistent Design of Structural Concrete," *Journal of the Pre-stressed Concrete Institute,* V. 32, No. 3, May-June 1987, pp. 74-150.

11.6 Joint Committee, "Recommended Practice and Standard Specification for Concrete and Reinforced Concrete," *Proceedings,* ASCE, V. 66, No. 6, Part 2, June 1940,81 pp.

11.7 Mphonde, A. G., and Frantz, G. C., "Shear Tests of High- and Low-Strength Concrete Beams without Stirrups," *ACI JOURNAL*, *Proceedings* V. 81, No. 4, July-Aug. 1984, pp. 350-357.

11.8 Elzanaty, A. H.; Nilson, A. H.; and Slate, F. O., "Shear Capacity of Reinforced Concrete Beams Using High Strength Concrete", *ACI JOURNAL*, *Proceedings* V. 83, No. 2, Mar.-Apr. 1986, pp. 290-296.

11.9 Roller, J. J., and Russell, H. G., "Shear Strength of High-Strength Concrete Beams with Web Reinforcement," *ACI Structural Journal,* V. 87, No. 2, Mar.-Apr. 1990, pp. 191-198.

11.10 Johnson, M.K., and Ramirez, J.A., "Minimum Amount of Shear Reinforcement in High Strength Concrete Members," *ACI Structural Journal,* V. 86, No. 4, July-Aug. 1989, pp. 376-382.

11.11 Ozcebe, G.; Ersoy, U.; and Tankut, T., "Evaluation of Minimum Shear Reinforcement for Higher Strength Concrete,'' *ACI Structural Journal,* V. 96, No., 3, May-June 1999, pp. 361-368.

11.12 Ivey, D. L., and Buth, E., "Shear Capacity of Lightweight Concrete Beams," *ACI JOURNAL*, *Proceedings* V. 64, No. 10, Oct. 1967, pp. 634-643.

11.13 Hanson, J. A., "Tensile Strength and Diagonal Tension Resistance of Structural Lightweight Concrete,'' *ACI JOURNAL*, *Proceedings* V. 58, No. l, July 1961, pp. 1-40.

11.14 Kani, G. N. J., "Basic Facts Concerning Shear Failure," *ACI JOURNAL, Proceedings* V. 63, No. 6, June 1966, pp. 675-692.

11.15 Kani, G. N. J., "How Safe Are Our Large Reinforced Concrete Beams," *ACI JOURNAL*, *Proceedings* V. 64, No. 3, Mar. 1967, pp. 128-141.

11.16 Faradji, M. J., and Diaz de Cossio, R., "Diagonal Tension in Concrete Members of Circular Section" (in Spanish) Institut de Ingeniena, Mexico (translation by Portland Cement Association, Foreign Literature Study No. 466).

11.17 Khalifa, J. U., and Collins, M. P., "Circular Reinforced Concrete Members Subjected to Shear," Publications No. 81-08, Department of Civil Engineering, University of Toronto, Dec. 1981.

11.18 PCI Design Handbook-Precast and Prestressed Concrete, 4th Edition, Pre-cast/Prestressed Concrete Institute, Chicago, 1992,580 pp.

11.19 ACI Committee 318, "Commentary on Building Code Requirements for Reinforced Concrete (ACI 3 18-63)," SP-10, American Concrete Institute, Farmington Hills, MI, 1965, pp. 78-84.

11.20 Guimares, G. N.; Kreger, M. E.; and Jirsa, J. O., "Evaluation of Joint-Shear Provisions for Interior Beam-Column-Slab Connections Using High Strength Materials," *ACI Structural Journal,* V. 89, No. 1, Jan.-Feb. 1992, pp. 89-98.

11.21 Griezic, A.; Cook, W. D.; and Mitchell, D., "Tests to Determine Performance of Deformed Welded-Wire Fabric Stirrups," *ACI Structural Journal,* V. 91, No. *2,* Mar.-Apr. 1994, pp. 2 11-220.

11.22 Furlong, R. W.; Fenves, G. L.; and Kasl, E. P., "Welded Structural Wire Reinforcement for Columns," *ACI Structural Journal,* V. 88, No. 5, Sept.-Oct. 1991, pp. 585-591.

11.23 Angelakos, D.; Bentz, E. C.; and Collins, M. D., "Effect of Concrete Strength and Minimum Stirrups on Shear Strength of Large Members," *ACI Structural Journal,* V. 98, No. 3, May-June2001, pp. 290-300.

11.24 Olesen, S. E.; Sozen, M. A.; and Siess, C. P., "Investigation of Pre-stressed Reinforced Concrete for Highway Bridges, Part IV: Strength in Shear of Beams with Web Reinforcement," *Bulletin* No. 493, University of Illinois, Engineering Experiment Station, Urbana, 1967.

11.25 Anderson, N. S., and Ramirez, J. A., "Detailing of Stirrup Reinforcement," *ACI Structural Journal,* V. 86, No. *5,* Sept.-Oct. 1989, pp. 507-515.

11.26 Leonhardt, F., and Walther, R., "The Stuttgart Shear Tests," *C&CA Translation,* No. 111, Cement and Concrete Association, 1964, London, 134 pp.

11.27 Macgregor, J. G., and Ghoneim, M. G., "Design for Torsion," *ACI Structural Journal,* V. 92, No. 2, Mar.-Apr. 1995, pp. 211-218.

11.28 Hsu, T. T. C., "ACI Shear and Torsion Provisions for Prestressed Hollow Girders," *ACI Structural Journal,* V. 94, No. 6, Nov.-Dec. 1997, pp. 787-799.

11.29 Hsu, T. T. C., "Torsion of Structural Concrete-Behavior of Reinforced Concrete Rectangular Members," *Torsion of Structural Concrete,* SP- 18, American Concrete Institute, Farmington Hills, MI, 1968, pp. 291-306.

11.30 Collins, M.P., and Lampert, P., "Redistribution of Moments at Cracking-The Key to Simpler Torsion Design?" *Analysis of Structural Systems for Torsion,* SP-35, American Concrete Institute, Fannington Hills, MI, 1973, pp. 343-383.

11.31 Hsu, T. T. C., and Burton, K. T., "Design of Reinforced Concrete Spandrel Beams," *Proceedings,* ASCE, V. 100,No. ST1, Jan. 1974, pp. 209-229.

11.32 Hsu, T. C., "Shear Flow Zone in Torsion of Reinforced Concrete," *ASCE Structural Engineering,* American Society of Civil Engineers, V. 116, No. 11, Nov. 1990, pp. 3206-3226.

11.33 Mitchell, D., and Collins, M. P., "Detailing for Torsion," ACI JOURNAL, *Proceedings* V. 73, No. 9, Sept. 1976, pp. 506-5 11.

11.34 Behera, U., and Rajagopalan, K. S., "Two-Piece U-Stirrups in Reinforced Concrete Beams," *ACI JOURNAL*, *Proceedings* V. 66, NO.7, July 1969, pp. 522-524.

11.35 Birkeland, P. W., and Birkeland, H. W., "Connections in Precast Concrete Construction," *ACI JOURNAL*, *Proceedings* V. 63, NO. 3, Mar. 1966, pp. 345-368.

11.36 Mattock, A. H., and Hawkins, N. M., "Shear Transfer in Reinforced Concrete-Recent Research," *Journal of the Prestressed Concrete Institute,* V. 17, No. 2, Mar.-Apr. 1972, pp. 55-75.

11.37 Mattock, A. H.; Li, W. K.; and Want, T. C., "Shear Transfer in Lightweight Reinforced Concrete,'' *Journal of the Pre-stressed Concrete Institute,* V. 21, No. I, Jan.-Feb. 1976, pp. 20-39.

11.38 Mattock, A. H., "Shear Transfer in Concrete Having Reinforcement at an Angle to the Shear Plane," *Shear in Reinforced Concrete,* SP-42, American Concrete Institute, Farmington Hills, MI, 1974, pp. 17-42.

11.39 Mattock, A. H., discussion of "Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads," by PCI Committee on Precast Concrete Bearing Wall Buildings, *Journal of the Pre-stressed Concrete Institute,* V. 22, No. 3, May-June 1977, pp. 105-106.

11.40 "Chapter l-Composite Members," *Load and Resistance Factor Design specification for Structural Steel for Buildings,* American Institute of Steel Construction, Chicago, Sept. 1986, pp. 51-58.

11.41 Mattock, A. H.; Johal, L.; and Chow, H. C., "Shear Transfer in Reinforced Concrete with Moment or Tension Acting Across the Shear Plane," *Journal of the Pre-stressed Concrete Institute,* V. *20,* NO.4, July-Aug. 1975, pp. 76-93.

11.42 Rogowsky, D. M., and Macgregor, J. G., "Design of Reinforced Concrete Deep Beams," *Concrete International: Design and Construction,* V. 8, No. 8, Aug. 1986, pp. 46-58.

11.43 Marti, P., "Basic Tools of Reinforced Concrete Beam Design," *ACI JOURNAL*, *Proceedings* V. 82, No. 1, Jan.-Feb. 1985,pp. 46-56.

11.44 Crist, R. A., "Shear Behavior of Deep Reinforced Concrete Beams," *Proceedings,* Symposium on the Effects of Repeated Loading of Materials and Structural Elements (Mexico City, 1966), V. 4, RILEM, Paris, 3 1 pp.

11.45 Kriz, L. B., and Raths, C. H., "Connections in Precast Concrete Structures-Strength of Corbels," *Journal of the Prestressed Concrete Institute,* V. 10, No. 1, Feb. 1965, pp. 16-47.

11.46 Mattock, A. H.; Chen, K. C.; and Soongswang, K., "The Behavior of Reinforced Concrete Corbels," *Journal of the Pre-stressed Concrete Institute,* V. 21, No. 2, Mar.-Apr. 1976, pp. 52-77.

11.47 Cardenas, A. E.; Hanson, J. M.; Corley, W. G.; and Hognestad, E., "Design Provisions for Shear Walls," *ACI JOURNAL, Proceedings* V. 70, No. 3, Mar. 1973, pp. 221-230.

11.48 Barda, F.; Hanson, J. M.; and Corley, W. G., "Shear Strength of Low-Rise Walls with Boundary Elements," *Reinforced Concrete Structures in Seismic Zones,* SP-53, American Concrete Institute, Farmington Hills, MI, 1977, pp. 149-202.

11.49 Hanson, N. W., and Conner, H. W., "Seismic Resistance of Reinforced Concrete Beam-Column Joints," *Proceedings,* ASCE, V. 93, ST5, Oct. 1967, pp. 533-560.

11.50 ACI-ASCE Committee 352, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-9 i)," American Concrete Institute, Farmington Hills, MI, 1991, 18 pp. Also *ACI Manual of Concrete Practice.*

11.51 ACI-ASCE Committee 426, "The Shear Strength of Reinforced Concrete Members," *Proceedings,* ASCE, V. 100, No. ST8, Aug. 1974, pp. 1543-1591.

11.52 Vanderbilt, M. D."Shear Strength of Continuous Plates," *Journal of the Structural Division,* ASCE, V. 98, No. ST5, May1972, pp. 961-973.

11.53 ACI-ASCE Committee 423, "Recommendations for Concrete Members Pre-stressed with Unbonded Tendons (ACI 423.3R-89)," American Concrete Institute, Farmington Hills, MI, 18 pp. Also *ACI Manual of Concrete Practice.*

11.54 Bums, N. H., and Hemakom, R., "Test of Scale Model of Post-Tensioned Flat Plate," *Proceedings,* ASCE, V. 103, ST6, June 1977, pp. 1237-1255.

11.55 Hawkins, N. M., "Shear Strength of Slabs with Shear Reinforcement," *Shear in Reinforced Concrete,* SP-42, V. 2, American Concrete Institute, Farmington Hills, MI, 1974, pp. 785-815.

11.56 Broms, C.E., "Shear Reinforcement for Deflection Ductility of Flat Plates," *ACI Structural Journal,* V. 87, No. 6, Nov.-Dec. 1990, pp. 696-705.

11.57 Yamada, T.; Nanni, A.; and Endo, K., "Punching Shear Resistance of Flat Slabs: Influence of Reinforcement Type and Ratio," *ACI Structural Journal,* V. 88, No. 4, July-Aug. 1991, pp. 555-563.

11.58 Hawkins, N. M.; Mitchell, D.; and Hannah, S. N., "The Effects of Shear Reinforcement on Reversed Cyclic Loading Behavior of Fiat Plate Structures," *Canadian Journal of Civil Engineering* (Ottawa), V. 2, 1975, pp. 572-582.

11.59 ACI-ASCE Committee 421, "Shear Reinforcement for Slabs (ACI 421. IR-99)," American Concrete Institute, Farmington Hills, MI, 1999, 15 pp.

11.60 Corley, W. G., and Hawkins. N. M., "Shearhead Reinforcement for Slabs," ACI JOURNAL, Proceedings V. 65, No. 10, Oct. 1968, pp. 81 1-824.

11.61 Hanson, N. W., and Hanson, J. M., "Shear and Moment Transfer between Concrete Slabs and Columns," *Journal,* PCA Research and Development Laboratories, V. 10, No. 1, Jan. 1968, pp. 2-16.

11.62 Hawkins, N. M., "Lateral Load Resistance of Unbonded Post-Tensioned Fiat Plate Construction," *Journal of the Pre-stressed Concrete Institute,* V. 26, No. I, Jan.-Feb. 1981, pp. 94-1 15.

11.63 Hawkins, N. M., and Corley, W. G., "Moment Transfer to Columns in Slabs with Shearhead Reinforcement," *Shear in Reinforced Concrete,* SP-42, American Concrete Institute, Farmington Hills, MI, 1974, pp. 847-879.

REFERENCES, CHAPTER 12

12.1 ACI Committee 408, "Suggested Development, Splice, and Standard Hook Provisions for Deformed Bars in Tension," (ACI 408.1R-90), American Concrete Institute, Farmington Hills, MI, 1990, 3 pp. Also *ACI Manual of Concrete Practice.*

12.2 Jirsa, J. O.; Lutz, L. A.; and Gergely, P., "Rationale for Suggested Development, Splice, and Standard Hook Provisions for Deformed Bars in Tension," *Concrete International: Design & Construction,* V. 1, No. 7, July 1979, pp. 47-61.

12.3 Treece, R. A., "Bond Strength of Epoxy-Coated Reinforcing Bars," Master's thesis, Department of Civil Engineering, University of Texas at Austin, May 1987.

12.4 Johnston, D. W., and Zia, P., "Bond characteristics of Epoxy- Coated Reinforcing Bars," Department of Civil Engineering, North Carolina State University, *Report* No. F"C/82-002, Aug. 1982.

12.5 Mathey, R. G., and Clifton, J. R., "Bond of Coated Reinforcing Bars in Concrete," *Journal of the Structural Division,* ASCE, V.102, No. ST1, Jan. 1976, pp. 215-228.

12.6 Orangun, C. O.; Jirsa, J. O.; and Breen, J. E., "A Reevaluation of Test Data on Development Length and Splices," *ACI JOURNAL*, *Proceedings* V. 74, No. 3, Mar. 1977, pp. 114-122.

12.7 Azizinamini, A.; Pavel, R.; Hatfield, E.; and Ghosh, S. K., "Behavior of Spliced Reinforcing Bars Embedded in High-Strength Concrete," *ACI Structural Journal,* V. 96, No. 5, Sept.-Oct. 1999, pp. 826-835.

12.8 Azizinamini, A.; Darwin, D.; Eligehausen, R.; Pavel, R.; and Ghosh, S. K., "Proposed Modifications to ACI 318-95 Development and Splice Provisions for High-Strength Concrete," *ACI Structural Journal,* V. 96, No. 6, Nov.-Dec. 1999, pp. 922-926.

12.9 Jirsa, J. O., and Marques, J. L. G., "A Study of Hooked Bar Anchorages in Beam-Column Joints," *ACI JOURNAL, Proceedings* V. 72, No. 5, May 1975, pp. 198-200.

12.10 Hamad, B. S.; Jirsa, J. O.; and D'Abreu, N. I., "Anchorage Strength of Epoxy-Coated Hooked Bars," *ACI Structural Journal,* V. 90, No. 2, Mar.-Apr. 1993, pp. 210-217.

12.11 Bartoletti, S. J., and Jirsa, J. O., "Effects of Epoxy-Coating on Anchorage and Development of Welded Wire Fabric," *ACI Structural Journal,* V. 92, No. 6, Nov.-Dec. 1995, pp. 757-764.

12.12 Rose, D. R., and Russell, B. W., 1997, "Investigation of Standardized Tests to Measure the Bond Performance of Pre-stressing Strand," *PCI Journal,* V. 42, No. 4, Jul.-Aug., 1997, pp. 56-60.

12.13 Logan, D. R., "Acceptance Criteria for Bond Quality of Strand for Pre-tensioned Pre-stressed Concrete Applications," *PCI Journal,* V. 42, No. 2, Mar.-Apr., 1997, pp. 52-90.

12.14 Martin, L., and Korkosz, W., "Strength of Pre-stressed Members at Sections Where Strands Are Not Fully Developed," *PCI Journal,* V. 40, No. 5, Sept.-Oct. 1995, pp. 58-66.

12.15 *PCI Design Handbook -Precast and Prestressed Concrete,* 5th Edition, Pre-cast/Pre-stressed Concrete Institute, Chicago, IL, 1998, pp. 4-27 to 4-29.

12.16 Kaar, P., and Magura, D., "Effect of Strand Blanketing on Performance of Pre-tensioned Girders," *Journal of the Prestressed Concrete Institute,* V. 10, No. 6, Dec. 1965, pp. 20-34.

12.17 Hanson, N. W., and Kaar, P. H., "Flexural Bond Tests Pre-tensioned Beams," *ACI JOURNAL, Proceedings* V. 55, No. 7, Jan. 1959, pp. 783-802.

12.18 Kaar, P. H.; La Fraugh, R. W.; and Mass, M. A., "Influence of Concrete Strength on Strand Transfer Length," *Journal of the Prestressed Concrete Institute,* V. 8, No. 5, Oct. 1963, pp. 47-67.

12.19 Rabbat, B. G.; Kaar, P. H.; Russell, H. G.; and Bruce, R. N., Jr., "Fatigue Tests of Pre-tensioned Girders with Blanketed and Draped Strands," *Journal of the Prestressed Concrete Institute,* V. 24. NO. 4, July-Aug. 1979, pp. 88-1 14.

12.20 ACI Committee 408, "Bond Stress- The State of the *Art," ACI JOURNAL, Proceedings* V. 63, No.11, Nov.1966, pp. 1161-1188.

12.21 Rogowsky, D. M., and Macgregor, J. G., "Design of Reinforced Concrete Deep Beams," Concrete *International: Design* & *Construction,* V. 8, No. 8, Aug. 1986, pp. 46-58.

12.22 Joint PCVWRI Ad Hoc Committee on Welded Wire Fabric for Shear Reinforcement, "Welded Wire Fabric for Shear Reinforcement," *Journal of the Pre-stressed Concrete Institute,* V. 25, NO.4, July-Aug. 1980, pp. 32-36.

12.23 Pfister, J. F., and Mattock, A. H., "High Strength Bars as Concrete Reinforcement, Part 5: Lapped Splices in Concentrically Loaded Columns," *Journal,* PCA Research and Development Laboratories, V. 5, No. 2, May 1963, pp. 27-40.

12.24 Lloyd, J. P., and Kesler, C. E., "Behavior of One-way Slabs Reinforced with Deformed Wire and Deformed Wire Fabric," *T&AM Report* No. 323, University of Illinois, 1969, 129 pp.

12.25 Lloyd, J. P., "Splice Requirements for One-way Slabs Reinforced with Smooth Welded Wire Fabric," *Publication* No. R(S)4, Civil Engineering, Oklahoma State University, June 1971, 37 pp.

REFERENCES, CHAPTER 13

13.1 Hatcher, D. S.; Sozen, M. A.; and Siess, C. P., "Test of a Reinforced Concrete Flat Plate," *Proceedings,* ASCE, V. 91, ST5, Oct. 1965, pp. 205-231.

13.2 Guralnick, S. A., and LaFraugh, R. W., "Laboratory Study of a Forty-Five-Foot Square Flat Plate Structure," *ACI JOURNAL*, *Proceedings* V. 60, No. 9, Sept. 1963, pp. 1107-1185.

13.3 Hatcher, D. S.; Sozen, M. A.; and Siess, C. P., "Test of a Reinforced Concrete Flat Slab," *Proceedings,* ASCE, V. 95, No. ST6, June 1969, pp. 1051-1072.

13.4 Jirsa, J. O.; Sozen, M. A.; and Siess, C. P., "Test of a Flat Slab Reinforced with Welded Wire Fabric," *Proceedings,* ASCE, V. 92, No. ST3, June 1966, pp. 199-224.

13.5 Gamble, W. L.; Sozen, M. A.; and Siess, C. P., "Tests of a Two-way Reinforced Concrete Floor Slab," *Proceedings,* ASCE, V. 95, No. ST6, June 1969, pp. 1073-1096.

13.6 Vanderbilt, M. D.; Sozen, M. A.; and Siess, C. P., "Test of a Modified Reinforced Concrete Two-way Slab," *Proceedings,* ASCE, V. 95, No. ST6, June 1969, pp. 1097-1116.

13.7 Xanthakis, M., and Sozen, M. A., "An Experimental Study of Limit Design in Reinforced Concrete Flat Slabs," Civil Engineering Studies, *Structural Research Series* No. 277, University of Illinois, Dec. 1963, 159 pp.

13.8 *ACI Design Handbook, V. 3-Two-way Slabs,* SP-l7(91)(S), American Concrete Institute, Farmington Hills, MI, 1991, 104 pp.

13.9 Mitchell, D., and Cook, W. D., "Preventing Progressive Collapse of Slab Structures," *Journal of Structural Engineering,* V.110, No.7, July 1984, pp. 1513-1532.

13.10 Carpenter, J. E.; Kaar, P. H.; and Corley, W. G., "Design of Ductile Flat-Plate Structures to Resist Earthquakes," *Proceedings,* Fifth World Conference on Earthquake Engineering Rome, June 1973, International Association for Earthquake Engineering, V. 2, pp. 2016-2019.

13.11 Morrison, D. G., and Sozen, M. A., "Response to Reinforced Concrete Plate-Column Connections to Dynamic and Static Horizontal Loads," Civil Engineering Studies, *Structural Research Series* No. 490, University of Illinois, Apr. 1981, 249 pp.

13.12 Vanderbilt, M. D., and Corley, W. G., "Frame Analysis of Concrete Buildings," *Concrete International: Design and Construction,* V. *5,* No. 12, Dec. 1983, pp. 33-43.

13.13 Grossman, J. S., "Code Procedures, History, and Shortcomings: Column-Slab Connections," *Concrete International,* V. 11, No. 9, Sept. 1989, pp. 73-77.

13.14 Moehle, J. P., "Strength of Slab-Column Edge Connections," *ACI Structural Journal,* V. 85, No. 1, Jan.-Feb. 1988, pp. 89-98.

13.15 ACI-ASCE Committee 352,"Recommendations for Design of Slab-Column Connections in Monolithic Reinforced Concrete Structures (ACI352.1R-S9),"ACI *Structural Journal,* V. 85, No. 6, Nov.-Dec. 1988, pp. 675-696.

13.16 Jirsa, J. O.; Sozen, M. A.; and Siess, C. P., "Pattern Loadings on Reinforced Concrete Floor Slabs," *Proceedings,* ASCE, V. 95, No. ST6, June 1969, pp. 1117-1137.

13.17 Nichols, J. R., "Statical Limitations upon the Steel Requirement in Reinforced Concrete Flat Slab Floors," *Transactions,* ASCE, V. 77, 1914, pp. 1670-1736.

13.18 Corley, W. G.; Sozen, M. A.; and Siess, C. P., "Equivalent- Frame Analysis for Reinforced Concrete Slabs," Civil Engineering Studies, *Structural Research Series* No. 218, University of Illinois, June 1961, 166 pp.

13.19 Jirsa, J. O.; Sozen, M. A.; and Siess, C. P., "Effects of Pattern Loadings on Reinforced Concrete Floor Slabs," Civil Engineering Studies, *Structural Research Series* No. 269, University of Illinois, July 1963.

13.20 Corley, W. G., and Jirsa, J. O., "Equivalent Frame Analysis for Slab Design," *ACI JOURNAL, Proceedings* V. 67, No. 11, Nov. 1970, pp. 875-884.

13.21 Gamble, W. L., "Moments in Beam Supported Slabs," *ACI JOURNAL, Proceedings* V. 69, No. 3, Mar. 1972, pp. 149-157.

REFERENCES, CHAPTER 14

14.1 *Uniform Building Code,* V. 2, "Structural Engineering Design Provisions," International Conference of Building Officials, Whittier, CA, 1997,492 pp.

14.2 Athey, J. W., ed., "Test Report on Slender Walls," Southern California Chapter of the American Concrete Institute and Structural Engineers Association of Southern

California, Los Angeles, CA,1982,129 pp.

14.3 ACI Committee 551, "Tilt-Up Concrete Structures (ACI *55* 1R-92)," American Concrete Institute, Farmington Hills, MI, 1992,46 pp. Also *ACI Manual of Concrete Practice.*

14.4 Carter III, W., Hawkins, N. M., and Wood, S. L. "Seismic Response of Tilt-Up construction," Civil Engineering Series, SRS No.581, University of Illinois, Urbana, IL, Dec. 1993,224 pp.

REFERENCES, CHAPTER 15

15.1 ACI Committee 336, "Suggested Analysis and Design Procedures for Combined Footings and Mats (ACI 336.2R-88)," American Concrete Institute, Farmington Hills, MI, 1988, 21 pp. Also *ACI Manual of Concrete Practice.*

15.2 Kramrisch, F., and Rogers, P., "Simplified Design of Combined Footings," *Proceedings,* ASCE, V. 87, No. SM5, Oct. 1961, p. 19.

15.3 Adebar, P.; Kuchma, D.; and Collins, M. P., "Strut-and-Tie Models for the Design of Pile Caps: An Experimental Study," *ACI Structural Journal,* V. 87, No. 1, Jan.-Feb. 1990, pp. 81-92.

15.4 *CRSI Handbook,* 7th Edition, Concrete Reinforcing Steel Institute, Schaumburg, IL, 1992, 840 pp.

REFERENCES, CHAPTER 16

16.1 *Industrialization in Concrete Building Construction,* SP-48, American Concrete Institute, Farmington Hills, MI, 1975, 240 pp.

16.2 Waddell, J. J., "Precast Concrete: Handling and Erection," *Monograph* No. 8, American Concrete Institute, Farmington Hills, MI, 1974,146 pp.

16.3 "Design and Typical Details of Connections for Precast and Prestressed Concrete," MNL- 123-88, 2nd Edition, Pre-cast/Pre-stressed Concrete Institute, Chicago, 1988, 270 pp.

16.4 *PCI Design Handbook-Precast and Prestressed Concrete,* MNL- 120-92, 4th Edition, Pre-cast/Pre-stressed Concrete Institute, Chicago, 1992,580 pp.

16.5 "Design of Prefabricated Concrete Buildings for Earthquake Loads," *Proceedings of Workshop,* Apr. 27-29, 1981, ATC-8, Applied Technology Council, Redwood City, CA, 717 pp.

16.6 PCI Committee on Building Code and PCI Technical Activities Committee, "Proposed Design Requirements for Precast Concrete," *PCI Journal, V.* 31, No. 6, Nov.-Dec. 1986, pp. 32-47.

16.7 ACI-ASCE Committee 550, "Design Recommendations for Precast Concrete Structures (ACI 550R-93)," *ACI Structural Journal,* V. 90, No. 1, Jan.-Feb. 1993, pp. 115-121. *Also ACI Manual of Concrete Practice.*

16.8 ACI Committee 551, "Tilt-Up Concrete Structures (ACI551R-92)," American Concrete Institute, Farmington Hills, MI, 1992,46 pp. Also *ACI Manual of Concrete Practice.*

16.9 Manual for Quality Control for Plants and Production of Precast and Pre-stressed Concrete Products, MNL-116-85, 3rd Edition, Pre-cast/Pre-stressed Concrete Institute, Chicago, 1985, 123 pp.

16.10 "Manual for Quality Control for Plants and Production of Architectural Precast Concrete," MNL- 117-77, Pre-cast/Pre-stressed Concrete Institute, Chicago, 1977,226 pp.

16.11 PCI Committee on Tolerances, "Tolerances for Precast and Pre-stressed Concrete," *PCI Journal,* V. 30, No. 1, Jan.-Feb. 1985, pp. 26-112.

16.12 ACI Committee 117, “Standard Specifications for Tolerances for Concrete Construction and Materials (ACI 117-90) and Commentary (1 17R-90),” American Concrete Institute, Farmington Hills, MI, 1990. Also *ACI Manual of Concrete Practice.*

16.13 LaGue, D. J., “Load Distribution Tests on Precast Pre-stressed Hollow-Core Slab Construction,” *PCI Journal,* V. 16, No. 6, Nov.-Dec. 1971, pp. 10-18.

16.14 Johnson, T., and Ghadiali, Z., “Load Distribution Test on Precast Hollow Core Slabs with Openings,” *PCI Journal,* V. 17, No. *5,* Sept.-Oct. 1972, pp. 9-19.

16.15 Pfeifer, D. W., and Nelson, T. A., “Tests to Determine the Lateral Distribution of Vertical Loads in a Long-Span Hollow-Core Floor Assembly,” *PCI Journal,* V. 28, No. 6, Nov.-Dec. 1983, pp. 42-57.

16.16 Stanton, J., “Proposed Design Rules for Load Distribution in Pre-cast Concrete Decks,” *ACI Structural Journal,* V. 84, NO. 5, Sept.-Oct. 1987, pp. 371-382.

16.17 *PCI Manual for the Design of Hollow Core Slabs,* MNL-126-85, Pre-cast\Pre-stressed Concrete Institute, Chicago, 1985, 120 pp.

16.18 Stanton, J. F., “Response of Hollow-Core Floors to Concentrated Loads,” *PCI Journal,* V. 37, No. 4, July-Aug. 1992, pp. 98-1 13.

16.19 Aswad, A., and Jacques, F. J., “Behavior of Hollow-Core Slabs Subject to Edge Loads,” *PCI Journal,* V. 37, No. 2, Mar.-Apr. 1992, pp. 72-84.

16.20 “Design of Concrete Structures for Buildings,” CAN3- A23.3-MS4, and “Precast Concrete Materials and Construction,” CAN3-A23.4-M84, Canadian Standards Association, Rexdale, Ontario, Canada.

16.21 “Design and Construction of Large-Panel Concrete Structures,” six reports, 762 pp., 1976-1980, EB 100D; three studies, 300 pp., 1980, EB 102D, Portland Cement Association, Skokie, IL

16.22 PCI Committee on Precast Concrete Bearing Wall Buildings, “Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads,” *PCI Journal,* V. 21, No. 2, Mar.-Apr. 1976, pp. 18-51.

16.23 Salmons, J. R., and McCrate, T. E., “Bond Characteristics of Untensioned Pre-stressing Strand,” *PCI Journal,* V. 22, No. I, Jan.- Feb. 1977, pp. 52-65.

16.24 PCI Committee on Quality Control and Performance Criteria, “Fabrication and Shipment Cracks in Pre-stressed Hollow-Core Slabs and Double Tees,” *PCI Journal,* V. 28, No. 1, Jan.-Feb. 1983,pp. 18-39.

16.25 PCI Committee on Quality Control and Performance Criteria, “Fabrication and Shipment Cracks in Precast or Pre-stressed Beams and Columns,” *PCI Journal,* V. 30, No. 3, May-June 1985,pp. 24-49.

REFERENCES, CHAPTER 17

17.1 “Specification for Structural Steel Buildings-Allowable Stress Design and Plastic Design, with Commentary” June 1989, and “Load and Resistance Factor Design Specification for Structural Steel Buildings,’’ Sept. 1986, American Institute of Steel Construction, Chicago.

17.2 Kaar, P. H.; Kriz, L. B.; and Hognestad, E., “Pre-cast\Pre-stressed Concrete Bridges: (1) Pilot Tests of Continuous Girders,” *Journal,* PCA Research and Development Laboratories, V. 2, No. 2, May 1960, pp. 21-37.

17.3 Saemann, J. C., and Washa, G. W., “Horizontal Shear Connections between Precast Beams and Cast-in-Place Slabs,” *ACI JOURNAL*, *Proceedings* V. 61, No.1 Nov. 1964, pp. 1383-1409. Also see discussion, ACI JOURNAL, June 1965.

17.4 Hanson, N. W., “Pre-cast\Pre-stressed Concrete Bridges: Horizontal Shear Connections,” *Journal,* PCA Research and Development Laboratories, V. 2, NO.2, May 1960, pp. 38-58.

17.5 Grossfield, B., and Birnstiel, C., "Tests of T-Beams with Precast Webs and Cast-in-Place Flanges," *ACI JOURNAL*, *Proceedings* V. 59, No. 6, June 1962, pp. 843-851.

17.6 Mast, R. F., "Auxiliary Reinforcement in Concrete Connections," *Proceedings,* ASCE, V. 94, No. ST6, June 1968,pp. 1485-1504.

REFERENCES, CHAPTER 18

18.1 "Analysis of Cracked Pre-stressed Concrete Sections: A Practical Approach," *PCI Journal,* V. 43, No. 4, Jul.-Aug., 1998.

18.2 *PCI Design Handbook-Precast and Pre-stressed Concrete,* 4th Edition, Pre-cast\Pre-stressed Concrete Institute, Chicago, 1992, pp. 4-42 through 4-44.

18.3 ACI-ASCE Committee 423, "Tentative Recommendations for Pre-stressed Concrete," *ACI JOURNAL*, *Proceedings* V. 54, No.7, Jan. 1958, pp. 545-578.

18.4 ACI Committee 435, "Deflections of Pre-stressed Concrete Members (ACI 435.1R-63)(Reapproved 1989)," ACI JOURNAL, *Proceedings* V. 60, No. 12, Dec. 1963, pp. 1697-1728. Also *ACI Manual of Concrete Practice.*

18.5 PCI Committee on Pre-stress Losses, "Recommendations for Estimating Pre-stress Losses," *Journal of* the *Pre-stressed Concrete Institute,* V. 20, No. 4, July-Aug. 1975, pp. 43-75.

18.6 Zia, P.; Preston, H. K.; Scott, N. L.; and Workman, E. B., "Estimating Pre-stress Losses," *Concrete International: Design & Construction,* V. 1, No. 6, June 1979, pp. 32-38.

18.7 Mojtahedi, S., and Gamble, W. L., "Ultimate Steel Stresses in Unbonded Pre-stressed Concrete," *Proceedings,* ASCE, V. 104, ST7, July 1978, pp. 1159-1165.

18.8 Mattock, A. H.; Yamazaki, J.; and Kattula, B. T., "Comparative Study of Pre-stressed Concrete Beams, with and without Bond," *ACI JOURNAL*, *Proceedings* V. 68, No. 2, Feb. 1971, pp. 116-125.

18.9 ACI-ASCE Committee 423, "Recommendations for Concrete Members Pre-stressed with Unbonded Tendons (ACI 423.3R3-89)," *ACI Structural Journal,* V. 86, No. 3, May-June 1989, pp. 301-318. Also *ACI Manual of Concrete Practice.*

18.10 Odello, R. J., and Mehta, B. M., "Behavior of a Continuous Pre-stressed Concrete Slab with Drop Panels," *Report,* Division of Structural Engineering and Structural Mechanics, University of California, Berkeley, 1967.

18.11 Smith, S. W., and Burns, N. H., "Post-Tensioned Flat Plate to Column Connection Behavior," *Journal of the Pre-stressed Concrete Institute,* V. 19, No. 3, May-June 1974, pp. 74-91.

18.12 Burns, N. H., and Hemakom, R., "Test of Scale Model Post- Tensioned Flat Plate," *Proceedings,* ASCE, V. 103, ST6, June 1977, pp. 1237-1255.

18.13 Hawkins, N. M., "Lateral Load Resistance of Unbonded Post- Tensioned Flat Plate Construction," *Journal of the Pre-stressed Concrete Institute,* V. 26, No. 1, Jan.-Feb. 1981, pp. 94- 116.

18.14 "Guide Specifications for Post-Tensioning Materials," *Post- Tensioning Manual,* 5th Edition, Post-Tensioning Institute, Phoenix, Anz., 1990, pp. 208-216.

18.15 Foutch, D. A.; Gamble, W. L.; and Sunidja, H., "Tests of Post-Tensioned Concrete Slab-Edge Column Connections," *ACI Structural Journal,* V. 87, No. 2, Mar.-Apr. 1990, pp. 167-179.

18.16 Mast, R.F., "Unified Design Provision for Reinforced and Pre-stressed Concrete Flexural and Compression Members," *ACI Structural Journal,* V. 89, No. 2, Mar.-Apr., 1992, pp. 185-199.

18.17 "Design of Post-Tensioned Slabs," Post-Tensioning Institute, Phoenix, Ariz., 1984,54 pp.

18.18 Gerber, L. L., and Bums, N. H., "Ultimate Strength Tests of Post-Tensioned Flat Plates," *Journal of the Pre-stressed Concrete Institute,* V. 16, No. 6, Nov.-Dec. 1971, pp. 40-58.

18.19 Scordelis, A. C.; Lin, T. Y.; and Itaya, R., "Behavior of a Continuous Slab Pre-stressed in Two Directions," ACI JOURNAL, *Proceedings* V. 56, No. 6, Dec. 1959, pp. 441-459.

18.20 American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Bridges," 16th Edition, 1996.

18.21 Breen, J. E.; Burdet, O.; Roberts, C.; Sanders, D.;Wollmann, G.; and Falconer, B., "Anchorage Zone Requirements for Post-Tensioned Concrete Girders," NCHRP *Report* 356, Transportation Research Board, National Academy Press, Washington, D.C., 1994.

18.22 ACI-ASCE Committee 423, "Recommendations for Concrete Members Pre-stressed with Unbonded Tendons," *ACI Structural Journal,* V. 86, No. 3, May-June 1989, p. 312.

18.23 "Specification for Unbonded Single Strand Tendons," revised 1993, Post-Tensioning Institute, Phoenix, AZ, 1993,20 pp.

18.24 "Guide Specifications for Design and Construction of Segmental Concrete Bridges, AASHTO, Washington, DC, 1989,50 pp.

18.25 Gerwick, B. C. Jr., "Protection of Tendon Ducts," *Construction of Pre-stressed Concrete Structures,* John Wiley and Sons, Inc., New York, 1971,411 pp.

18.26 "Recommended Practice for Grouting of Post-Tensioned Pre-stressed Concrete," *Post-Tensioning Manual,* 5th Edition, Post- Tensioning Institute, Phoenix, AZ, 1990, pp. 230-236.

18.27 Manual for Quality Control for Plants and Production of Pre-cast and Pre-stressed Concrete Products, 3rd Edition, MNL-116-85, Pre-cast/Pre-stressed Concrete Institute, Chicago, 1985, 123 pp.

18.28 ACI Committee 301, "Standard Specifications for Structural Concrete for Buildings (ACI 301-96)," American Concrete Institute, Farmington Hills, MI, 1996, 34 pp. Also *ACI Manual of Concrete Practice.*

18.29 Salmons, J. R., and McCrate, T. E., "Bond Characteristics of Untensioned Pre-stressing Strand," *Journal of the Pre-stressed Concrete Institute,* V. 22, No. 1, Jan.-Feb. 1977, pp. 52-65.

18.30 ACI Committee 215, "Considerations for Design of Concrete Structures Subjected to Fatigue Loading (ACI 215R- 4)(Revised 1992)," American Concrete Institute, Farmington Hills, MI, 1992,24 pp. Also *ACI Manual of Concrete Practice.*

18.31 Barth, F., "Unbonded Post-Tensioning in Building Construction," *Concrete Construction Engineering Handbook,* CRC Press, 1997, pp. 12.32-12.47.

REFERENCES, CHAPTER 19

19.1 ACI Committee 334, "Concrete Shell Structures-Practice and Commentary (ACI334.1R-92),"American Concrete Institute, Farmington Hills, MI, 14 pp. Also *ACI Manual of Concrete Practice.*

19.2 IASS Working Group No. 5, "Recommendations for Reinforced Concrete Shells and Folded Plates," International Association for Shell and Spatial Structures, Madrid, Spain, 1979,66pp.

19.3 Tedesko, A., "How Have Concrete Shell Structures Performed?' *Bulletin,* International Association for Shell and Spatial Structures, Madrid, Spain, No. 73, Aug. 1980, pp. 3-13.

19.4 ACI Committee 334, "Reinforced Concrete Cooling Tower Shells-Practice and Commentary (ACI 334.2R-91)," American Concrete Institute, Farmington Hills, MI, 1991, 9 pp. Also *ACI Manual of Concrete Practice.*

19.5 ACI Committee 373R3,"Design and Construction of Circular Pre-stressed Concrete Structures with Circumferential Tendons (ACI 373R-97)," American Concrete Institute, Farmington Hills, MI, 1997,26 pp. Also *ACI Manual of Concrete Practice.*

19.6 Billington, D. P., *Thin Shell Concrete Structures,* 2nd Edition, McGraw-Hill Book Co., New York, 1982,373 pp.

19.7 "Phase I Report on Folded Plate Construction," ASCE Task Committee, ASCE, *Journal of Structural Division, V.* 89, No. ST6 1963, pp. 365-406.

19.8 *Concrete Thin Shells,* SP-28, American Concrete Institute, Farmington Hills, MI, 1971,424 pp.

19.9 Esquillan N., "The Shell Vault of the Exposition Palace, Paris," ASCE, *Journal of Structural Division,* V. 86, No. STl, Jan. 1960, pp. 41-70.

19.10 *Hyperbolic Paraboloid Shells,* SP-110, American Concrete Institute, Farmington Hills, MI, 1988, 184 pp.

19.11 Billington, D. P., "Thin Shell Structures," *Structural Engineering Handbook,* Gaylord and Gaylord, eds., McGraw-Hill, New York, 1990, pp. 24.1-24.57.

19.12 Scordelis, A. C., "Non-Linear Material, Geometric, and Time Dependent Analysis of Reinforced and Pre-stressed Concrete Shells," *Bulletin,* International Association for Shells and Spatial Structures, Madrid, Spain, No. 102, Apr. 1990, pp. 57-90.

19.13 Schnobrich, W. C., "Reflections on the Behavior of Reinforced Concrete Shells," *Engineering Structures,* Butterworth, Heinemann, Ltd., Oxford, V. 13, No. 2, Apr. 1991, pp. 199-210.

19.14 Sabnis, G. M.; Harris, H. G.; and Mirza, M. S., *Structural Modeling and Experimental Techniques,* Prentice-Hall, Inc., Englewood Cliffs, N.J., 1983.

19.15 *Concrete Shell Buckling,* SP-67, American Concrete Institute, Farmington Hills, MI, 1981,234 pp.

19.16 Gupta, A. K., "Membrane Reinforcement in Concrete Shells: A Review," *Nuclear Engineering and Design,* Nofi-Holland Publishing, Amsterdam, V. 82, Oct. 1984, pp. 63-75.

19.17 Vecchio, F. J., and Collins, M. P., "Modified Compression-Field Theory for Reinforced Concrete Beams Subjected to Shear," *ACI JOURNAL Proceedings* V. 83, No. 2, Mar.-Apr.1986, pp. 219-223.

19.18 Fialkow, M. N., "Compatible Stress and Cracking in Reinforced Concrete Membranes with Multidirectional Reinforcement," *ACI Structural Journal,* V. 88, No. 4, July-Aug. 1991, pp. 445-457.

19.19 Medwadowski, S., "Multidirectional Membrane Reinforcement," *ACI Structural Journal,* V. 86, No. 5, Sept.-Oct.1989, PP. 563-569.

19.20 ACI Committee 224, "Control of Cracking in Concrete Structures (ACI 224R-90)," American Concrete Institute, Farmington Hills, MI, 1990,43 pp. Also *ACI Manual of Concrete Practice.*

19.21 Gupta, A. K., "Combined Membrane and Flexural Reinforcement in Plates and Shells," *Structural Engineering,* ASCE, V.112, NO. 3, Mar, 1986, pp. 550-557.

19.22 Tedesko, A., "Construction Aspects of Thin Shell Structures," *ACI JOURNAL*, *Proceedings* V. 49, No. 6, Feb. 1953, pp. 505-520.

19.23 Huber, R. W., "Air Supported Forming-Will it Work" *Concrete International,* V. 8, No. 1, Jan. 1986, pp. 13-17.

REFERENCES, CHAPTER 21

21.1 "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures," Part 1: Provisions (FEMA 302,353 pp.) and Part 2: Commentary

(FEMA 303,335 pp.), Building Seismic Safety Council, Washington, D. C., 1997.

21.2 *Uniform Building Code,* V. 2, "Structural Engineering Design Provisions," 1997 Edition, International Conference of Building Officials, Whittier, CA, 1997, 492 pp.

21.3 "BOCA National Building Code," 13th Edition, Building Officials and Code Administration International, Inc., Country Club Hills, IL, 1996, 357 pp.

21.4 "Standard Building Code," Southern Building Code Congress International, Inc., Birmingham, Ala., 1996, 656 pp.

21.5 "International Building Code," International Code Council, Falls Church, VA, International Council, 2000.

21.6 Blume, J. A.; Newmark, N. M.; and Coming, L. H., *Design of Multistory Reinforced Concrete Buildings for Earthquake Motions,* Portland Cement Association, Skokie, IL, 1961, 318 pp.

21.7 Clough, R. W., "Dynamic Effects of Earthquakes," *Proceedings,* ASCE, V. 86, ST4, Apr. 1960, pp. 49-65.

21.8 Gulkan, P., and Sozen, M. A., "Inelastic Response of Reinforced Concrete Structures to Earthquake Motions," ACI JOURNAL, *Proceedings V.* 71, No. 12, Dec. 1974., pp. 604-610.

21.9 "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures: Part 1: Provisions (FEMA 368, 374 pp.); and Part 2: Commentary (FEMA 369, 444 pp.), Building Seismic Safety Council, Washington, DC, 2000.

21.10 ACI-ASCE Committee 352, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-9 I)," American Concrete Institute, Farmington Hills, MI, 1991, 18 pp. Also *ACI Manual of Concrete Practice.*

21.11 Hirosawa, M., "Strength and Ductility of Reinforced Concrete Members," *Report* No. 76, Building Research Institute, Ministry of Construction, Tokyo, Mar. 1977 (in Japanese). Also, data in Civil Engineering Studies, *Structural Research Series* No. 452, University of Illinois, 1978.

21.12 "Recommended Lateral Force Requirements and Commentary," Seismology Committee of the Structural Engineers Association of California, Sacramento, CA, 6th Edition, 504 pp.

21.13 Popov, E. P.; Bertero, V. V.; and Krawinkler, H., "Cyclic Behavior of Three R/C Flexural Members with High Shear," EERC *Report* No. 72-5, Earthquake Engineering Research Center, University of California, Berkeley, Oct. 1972.

21.14 Wight, J. K., and Sozen, M. A., "Shear Strength Decay of RC Columns under Shear Reversals," *Proceedings,* ASCE, V. 101, ST5, May 1975, pp. 1053-1065.

21.15 French, C. W., and Moehle, J. P., "Effect of Floor Slab on Behavior of Slab-Beam-Column Connections," ACI SP- 123, *Design of Beam-Column Joints for Seismic Resistance,* American Concrete Institute, Farmington Hills, MI, 1991, pp. 225-258.

21.16 Sivakumar, B.; Gergely, P.; White, R. N., "Suggestions for the Design of R/C Lapped Splices for Seismic Loading," *Concrete International,* V. *5,* No. 2, Feb. 1983, pp. 46-50.

21.17 Sakai, K., and Sheikh, S. A., "What Do We Know about Confinement in Reinforced Concrete Columns" (A Critical Review of Previous Work and Code Provisions)," *ACI Structural Journal,* V. 86, No. 2, Mar.-Apr. 1989, pp. 192-207.

21.18 Park, R. "Ductile Design Approach for Reinforced Concrete Frames," *Earthquake Spectra,* V. 2, No. 3, May 1986, pp. 565-619.

21.19 Watson, S.; Zahn, F. A.; and Park, R., "Confining Reinforcement for Concrete Columns," *Journal of Structural Engineering,* V. 120, No. 6, June 1994, pp. 1798-1824.

21.20 Meinheit, D. F., and Jirsa, J. O., "Shear Strength of Reinforced Concrete Beam-Column Joints," *Report* No. 77-1, Department of Civil Engineering, Structures Research Laboratory, University of Texas at Austin, Jan. 1977.

21.21 Briss, G. R.; Paulay, T; and Park, R., "Elastic Behavior of Earthquake Resistant R. C. Interior Beam-Column Joints," *Report* 78- 13, University of Canterbury, Department of Civil Engineering, Christchurch, New Zealand, Feb. 1978.

21.22 Ehsani, M. R., "Behavior of Exterior Reinforced Concrete Beam to Column Connections Subjected to Earthquake Type Loading,'' *Report* No. UMEE 82R5, Department of Civil Engineering, University of Michigan, July 1982,275 pp.

21.23 Durrani, A. J., and Wight, J. K., "Experimental and Analytical Study of Internal Beam to Column Connections Subjected to Reversed Cyclic Loading," *Report* No. UMEE 82R3, Department of Civil Engineering, University of Michigan, July 1982,275 pp.

21.24 Leon, R. T., "Interior Joints with Variable Anchorage Lengths," *Journal of Structural Engineering,* ASCE, V. 115,No. 9, Sept. 1989, pp. 2261- 2275.

21.25 Zhu, S., and Jirsa, J. O., "Study of Bond Deterioration in Reinforced Concrete Beam-Column Joints," PMFSEL *Report* No. 83-1, Department of Civil Engineering, University of Texas at Austin, July 1983.

21.26 Meinheit, D. F., and Jirsa, J. O., "Shear Strength of R/C Beam-Column Connections," *Journal of the Structural Division,* ASCE, V. 107, NO.ST11, NOV.1982, pp. 2227-2244.

21.27 Ehsani, M. R., and Wight, J. K., "Effect of Transverse Beams and Slab on Behavior of Reinforced Concrete Beam to Column Connections," *ACI JOURNAL ,Proceedings* V. 82, No. 2, Mar.- Apr. 1985, pp. 188-195.

21.28 Ehsani, M. R., "Behavior of Exterior Reinforced Concrete Beam to Column Connections Subjected to Earthquake Type Loading," *ACI JOURNAL, Proceedings* V. 82, No. 4, July-Aug. 1985,pp. 492-499.

21.29 Durrani, A. J., and Wight, J. K., "Behavior of Interior Beam to Column Connections under Earthquake Type Loading," *ACI JOURNAL, Proceedings* V. 82, No. 3, May-June 1985, pp. 343-349.

21.30 ACI-ASCE Committee 326, "Shear and Diagonal Tension," *ACI JOURNAL, Proceedings* V. 59, No. 1, Jan. 1962, pp. 1-30; No.2, Feb. 1962, pp. 277-334; and No. 3, Mar. 1962, pp. 352-396.

21.31 Yoshioka, K., and Sekine, M., "Experimental Study of Prefabricated Beam-Column Sub-assemblages," *Design of Beam-Column Joints for Seismic Resistance,* SP-123, American Concrete Institute, Farmington Hills, MI, 1991, pp. 465-492.

21.32 Kurose, Y,; Nagami, K.; and Saito, Y., "Beam-Column Joints in Precast Concrete Construction in Japan," *Design of Beam- Column Joints for Seismic Resistance,* SP-123, American Concrete Institute, 1991, pp. 493-514.

21.33 Restrepo, J.; Park, R.; and Buchanan, A., "Tests on Connections of Earthquake Resisting Precast Reinforced Concrete Perimeter Frames," *Pre-cast/Pre-stressed Concrete Institute Journal,* V. 40, NO.*5,* pp. 44-61.

21.34 Restrepo, J.; Park, R.; and Buchanan, A., "Design of Connections of Earthquake Resisting Precast Reinforced Concrete Perimeter Frames," *Pre-cast/Pre-stressed Concrete Institute Journal,* V. 40, NO. *5,* 1995, pp. 68-80.

21.35 Palmieri, L.; Saqan, E.; French, C.; and Kreger, M., "Ductile Connections for Precast Concrete Frame Systems," *Mete A. Sozen Symposium, ACI SP-162,* American Concrete Institute, Farmington Hills, MI, 1996, pp. 315-335.

21.36 Stone, W.; Cheok, G.; and Stanton, J., "Performance of Hybrid Moment-Resisting Precast Beam-Column Concrete Connections Subjected to Cyclic Loading," *ACI JOURNAL, Proceedings* V. 40, No. *5,* Sept.-Oct. 1944, pp. 68-80.

21.37 Nakaki, S. D.; Stanton, J.F.; and Sritharan, S., "An Overview of the PRESSS Five-Story Precast Test Building," *Pre-cast/Pr-estressed Concrete Institute Journal,* V. 44, No. 2, pp. 26-39.

21.38 Barda, F.; Hanson, J. M.; and Corley, W. G., "Shear Strength of Low-Rise Walls with Boundary Elements," *Reinforced Concrete Structures in Seismic Zones,* SP-53, American Concrete Institute, Farmington Hills, MI, 1977, pp. 149-202.

21.39 Taylor, C. P.; Cote, P. A.; and Wallace, J. W., "Design of Slender RC Walls with Openings," *ACI Structural Journal,* V. 95, NO. 4, July-Aug. 1998, pp. 420-433.

21.40 Wallace, J. W., "Evaluation of UBC-94 Provisions for Seismic Design of RC Structural Walls," *Earthquake Spectra,* V. 12, No. 2, May 1996, pp. 327-348.

21.41 Moehle, J. P., "Displacement-Based Design of RC Structures Subjected to Earthquakes," *Earthquake Spectra,* V. 8, No. 3, Aug. 1992, pp. 403-428.

21.42 Wallace, J. W., "A New Methodology for Seismic Design of Reinforced Concrete Shear Walls," *Journal of Structural Engineering,* ASCE, V. 120, No. 3, Mar. 1994, pp. 863-884.

21.43 Paulay, T., and Binney, J. R., "Diagonally Reinforced Coupling Beams of Shear Walls," *Shear in Reinforced Concrete,* SP-42, American Concrete Institute, Farmington Hills, MI, 1974,pp. 579-598.

21.44 Barney, G. G. et al., *Behavior of Coupling Beams under Load Reversals* (RD068.01B), Portland Cement Association, Skokie, IL, 1980.

21.45 Wyllie, L. A., Jr., "Structural Wails and Diaphragms -How They Function," *Building Structural Design Handbook,* R. N. White, and C. G. Salmon, eds., John Wiley & Sons, 1987, pp. 188-215.

21.46 Wood, S. L., Stanton, J. F., and Hawkins, N. M., "Development of New Seismic Design Provisions for Diaphragms Based on the Observed Behavior of Precast Concrete Parking Garages during the 1994 Northridge Earthquake," *Journal,* Pre-cast/pre-stressed Concrete Institute, V. 45, No. 1, Jan.-Feb. 2000, pp. 50-65.

21.47 Nilsson, I. H. E., and Losberg, A., "Reinforced Concrete Corners and Joints Subjected to Bending Moment," *Journal of the Structural Division,* ASCE, V. 102, No. ST6, June 1976, pp. 1229-1254.

21.48 ACI-ASCE Committee 352, "Recommendations for Design of Slab-Column Connections in Monolithic Reinforced Concrete Structures (ACI 352.1R-89)," American Concrete Institute, Farmington Hills, MI, 1989.

21.49 Pan, A., and Moehle, J. P., "Lateral Displacement Ductility of Reinforced Concrete Flat Plates," *ACI Structural Journal,* V. 86, No. 3, May-June, 1989, pp. 250-258.

REFERENCES, APPENDIX A

A.1 Schlaich, J.; Schäfer, K.; and Jennewein, M., "Toward a Consistent Design of Structural Concrete," *PCI Journal,* V. 32, No. 3, May-June, 1987, pp 74-150.

A.2 Collins, M. P., and Mitchell, D., *Pre-stressed Concrete Structures,* Prentice Hall Inc., Englewood Cliffs, NJ, 1991, 766 pp.

A.3 Macgregor, J. G., *Reinforced Concrete: Mechanics and Design, 3rd Edition.,* Prentice Hall, Englewood Cliffs, NJ, 1997, 939 pp. A.4.

A.4 *FIP Recommendations, Practical Design* of *Structural Concrete,* FIP-Commission *3,* "Practical Design," Sept. 1996, Pub.: SE- TO, London, Sept. 1999.

A.5 Menn, C, *Prestressed Concrete Bridges,* Birkhäuser, Basie, 535 pp.

A.6 MuttoN, A; Schwartz,J.; and Thiirlimann,B., *Design* of *Concrete Structures with Stress Fields,* Birkhauser,Boston, Mass., 1997,143pp.

A.7 Joint ACI-ASCE Committee 445, "Recent Approaches to Shear Design of Structural Concrete," *ASCE Journal of Structural Engineering,* Dec., 1998, pp 1375-1417.

A.8 Bergmeister, K.; Breen, J. E.; and Jirsa, J. O., "Dimensioning of the Nodes and Development of Reinforcement," *IABSE Colloquium Stuttgart 1991,* International Association for Bridge and Structural Engineering, Zurich, 1991, pp. 551-556.

REFERENCES, APPENDIX B

B.l Cohn, M. A., "Rotational Compatibility in the Limit Design of Reinforced Concrete Continuous Beams," *Flexural Mechanics of Reinforced Concrete, ACI SP-12,* American Concrete Institute/ American Society of Civil Engineers, Farmington Hills, MI, 1965, pp. 35-46.

B.2 Mattock, A. H., "Redistribution of Design Bending Moments in Reinforced Concrete Continuous Beams," *Proceedings,* Institution of Civil Engineers, London, V. 13, 1959, pp. 35-46.

B.3 "Design of Post-Tensioned Slabs," Post-Tensioning Institute, Phoenix, Ariz., 1984, 54 pp.

B.4 Gerber, L. L., and Bums, N. H., "Ultimate Strength Tests of Post-Tensioned Flat Plates," *Journal of the Pre-stressed Concrete Institute,* V. 16, No. 6, Nov.-Dec. 1971, pp. 40-58.

B.5 Smith, S. W., and Bums, N. H., "Post-Tensioned Flat Plate to Column Connection Behavior," *Journal of the Pre-stressed Concrete Institute,* V. 19, No. 3, May-June, 1974, pp. 74-91.

B.6 Bums, N. H., and Hemakom, R., "Test of Scale Model Post- Tensioned Hat Plate," *Proceedings,* ASCE, V. 103, ST6, June 1977, pp. 1237-1255.

B.7 Bums, N. H., and Hemakom, R., "Test of Flat Plate with Bonded Tendons," *Proceedings,* ASCE, V. 111, No. 9, Sept. 1985, pp. 1899-1915.

B.8 Kosut, G. M.; Bums, N. H.; and Winter, C. V., "Test of Four- Panel Post-Tensioned Flat Plate," *Proceedings,* ASCE, V. 111, No. 9, Sept. 1985, pp. 1916-1929.

REFERENCES, APPENDIX D

D.1 ANSI/ASME B1.l, "Unified Inch Screw Threads (UN and UNR Thread Form), ASME, Fairfield, N.J., 1989.

D.2 ANSVASME B18.2.1, "Square and Hex Bolts and Screws, Inch Series," ASME, Fairfield, N.J., 1996.

D.3 ANSUASME B 18.2.6, "Fasteners for Use in Structural Applications," ASME, Fairfield, N.J., 1996.

D.4 Cook, R. A., and Klingner, R. E., "Behavior of Ductile Multiple- Anchor Steel-to-Concrete Connections with Surface-Mounted Base- plates," *Anchors in Concrete: Design and Behavior,* SP-130, 1992, American Concrete Institute, Farmington Hills, MI, pp. 61-122.

D.5 Cook, R. A., and Klingner, R. E., "Ductile Multiple-Anchor Steel-to-Concrete Connections," *Journal of Structural Engineering,* ASCE, V. 118, No. 6, June 1992, pp. 1645-1665.

D.6 Lotze, D., and Klingner, R.E., "Behavior of Multiple-Anchor Attachments to Concrete from the Perspective of Plastic Theory," *Report PMFSEL 96-4,* Ferguson Structural Engineering Laboratory, The University of Texas at Austin, Mar., 1997.

D.7 Primavera, E. J.; Pinelli, J.-P.; and Kalajian, E. H., "Tensile Behavior of Cast-in-Place and Undercut Anchors in High-Strength Concrete," *ACI Structural Journal,* V. 94, No. 5, Sept.-Oct. 1997, pp. 583-594.

D.8 *Design of Fastenings in Concrete,* Comite Euro-Intemational du Beton (CEB), Thomas Telford Services Ltd., London, Jan. 1997.

D.9 Fuchs, W.; Eligehausen, R.; and Breen, J., "Concrete Capacity Design (CCD) Approach for Fastening to Concrete," *ACZ Structural Journal,* V. 92, No. 1, Jan.-Feb., 1995, pp. 73-93. Also discussion, *ACI Structural Journal,* V. 92, No. 6, Nov.-Dec., 1995, pp. 787-802.

D.10 Eligehausen, R., and Balogh, T., "Behavior of Fasteners Loaded in Tension in Cracked Reinforced Concrete," *ACZ Structural Journal,* V. 92, No. 3, May-June 1995, pp. 365-379.

D.11 "Fastenings to Concrete and Masonry Structures, State of the Art Report," Comite Euro-International du Beton, (CEB), *Bulletin* No. 216, Thomas Telford Services Ltd., London, 1994.

D.12 Klingner, R.; Mendonca, J.; and Malik, J., "Effect of Reinforcing Details on the Shear Resistance of Anchor Bolts under Reversed Cyclic Loading," ACI JOURNAL,*Proceedings* V. 79, No. 1, Jan.-Feb. 1982, pp. 3-12.

D.13 Eligehausen, R.; Fuchs, W.; and Mayer, B., "Load Bearing Behavior of Anchor Fastenings in Tension," Betonwerk + Fertigteiltechnik, 12/1987, pp. 826-832, and 111988,pp. 29-35.

D.14 Eligehausen, R., and Fuchs, W., "Load Bearing Behavior of Anchor Fastenings under Shear, Combined Tension and Shear or Flexural Loadings," Betonwerk + Fertigteiltechnik,2/1988, pp. 48-56.

D.15 ACI Committee 349, "Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-85)," See also *ACI Manual of Concrete Practice,* Part 4, 1987.

D.16 Farrow, C.B., and Klingner, R.E., "Tensile Capacity of Anchors with Partial or Overlapping Failure Surfaces: Evaluation of Existing Formulas on an LRFD Basis," *ACI Structural Journal,* V. 92, No. 6, Nov.-Dec. 1995, pp. 698-710.

D.17 *PCI Design Handbook, 5th Edition,* Precast/Prestressed Concrete Institute, Chicago, 1999.

D.18 "AISC Load and Resistance Factor Design Specifications for Structural Steel Buildings," Dec. 1999, 327 pp.

D.19 Zhang, Y., "Dynamic Behavior of Multiple Anchor Connections in Cracked Concrete," PhD dissertation, The University of Texas at Austin, Aug. 1997.

D.20 Lutz, L., "Discussion to Concrete Capacity Design (CCD) Approach for Fastening to Concrete," *ACI Structural Journal,* Nov.-Dec. 1995, pp. 791-792. Also authors' closure, pp. 798-799.

D.21 Kuhn, D., and Shaikh, F., "Slip-Pullout Strength of Hooked Anchors," *Research Report,* University of Wisconsin-Milwaukee, submitted to the National Codes and Standards Council, 1996.

D.22 Furche, J., and Eligehausen, R., "Lateral Blow-out Failure of Headed Studs Near a Free Edge," Anchors in Concrete-*Design and Behavior,* SP- 130, American Concrete Institute, Farmington Hills, Mich., 1991, pp. 235-252.

D.23 Wong, T. L., "Stud Groups Loaded in Shear" MS thesis, Oklahoma State University, 1988.

D.24 Shaikh, A. F., and Yi, W., "In-Place Strength of Welded Studs," *PCI Journal,* V.30, No. 2, Mar.-Apr. 1985.