STRUCTURAL DESIGN HIGHLIGHTS OF ACI 318-19
PART 2 of 2 CHAPTERS 11 – 27
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ABSTRACT:

This presentation is a chapter by chapter review of ACI 318-19 “Building Code Requirements for Structural Concrete”, released in August 2019 to replace ACI 318-14. Highlighted are the code provisions which the author of this presentation has used most often while engaged in the design of industrial, marine, and commercial reinforced concrete structures. Figures and short example problems illustrating use of the provisions are included. The emphasis is on non-prestressed, non-seismic structures designed by traditional methods.
An ACI Standard

Building Code Requirements for Structural Concrete (ACI 318-19)

Commentary on Building Code Requirements for Structural Concrete (ACI 318R-19)

Reported by ACI Committee 318

ACI 318-19

American Concrete Institute
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CHAPTER 11 – WALLS
From Chapter 2: “wall” = a vertical element designed to resist axial load, lateral load, or both, with a horizontal length-to-thickness ratio greater than 3, used to enclose or separate spaces.
11.1 SCOPE

11.1.1 This chapter shall apply to the design of nonprestressed and prestressed walls including: cast-in-place, precast in plant, and precast on site including tilt-up.

11.1.2 Design of special structural walls: Chapter 18

11.1.3 Plain concrete walls: Chapter 14

11.1.4 Cantilever retaining walls: Chapter 13
11.2 – GENERAL

11.2.1 Materials: Concrete properties Chapter 19; Reinforcement properties Chapter 20
11.3 – DESIGN LIMITS

11.3.1 Minimum wall thickness

Table 11.3.1.1

Bearing wall: \( h \geq 4 \) inches and \( 1/25 \) the lessor of unsupported length and unsupported height

Nonbearing wall: \( h \geq 4 \) inches and \( 1/30 \)

Exterior basement and foundation: 7.5 inches
11.4 – REQUIRED STRENGTH

11.4.1 General: Load combinations Chapter 5; Analysis procedures Chapter 6; Slenderness effects according to 6.6.4, 6.7, or 6.8, or 11.8 for out-of-plane slenderness analysis
11.4 – REQUIRED STRENGTH (CONT’D)

11.4.2 Factored axial force and moment

Factored axial force at a given eccentricity shall not exceed the capacity given in 22.4.2.1.
Moments must include slenderness effects (second order elastic presumed)

11.4.3 Factored shear: Design for in-plane and out-of-plane
CODE

11.4.1.3 Slenderness effects shall be calculated in accordance with 6.6.4, 6.7, or 6.8. Alternatively, out-of-plane slenderness analysis shall be permitted using 11.8 for walls meeting the requirements of that section.

\[ V_n = \left( \alpha_c \right) \left( \frac{F_{cd}}{P_{ct}} + \frac{f_y}{P_{ct}} \right) A_{cv} \]

\[ \alpha_c = \begin{cases} 3 & \text{for } \frac{h}{l} \leq 1.5 \\ 2 & \text{for } \frac{h}{l} > 2.0 \end{cases} \]

The area of transverse reinforcement (ce horizontal) reinforcement per vertical inch/height

\[ A_{cv} = A_g = h \frac{l}{w} \]

\[ h \times \text{is entire height of multi-story} \]

11.4.4 Walls shall be designed for eccentric axial loads and any lateral or other loads to which they are subjected.

11.4.3 Factored axial force and moment

11.4.3.1 Walls shall be designed for the maximum factored moment \( M_u \) that can accompany the factored axial force for each applicable load combination. The factored axial force \( P_u \) at given eccentricity shall not exceed \( \phi P_{int} \) where \( P_{int} \) shall be as given in 22.4.2.1 and strength reduction factor \( \phi \) shall be that for compression-controlled sections in 21.3.2. The maximum factored moment \( M_u \) shall be magnified for slenderness effects in accordance with 6.6.4, 6.7, or 6.8.

11.4.3.2 Factored shear

11.4.3.3 Walls shall be designed for the maximum in-plane \( V_u \) and out-of-plane \( V_u \).

11.5—Design strength

11.5.1 General

11.5.1.3 For each applicable factored load combination, design strength at all sections shall satisfy \( \phi_k \geq E \), including (a) through (c). Interaction between axial load and moment shall be considered.
11.5 – DESIGN STRENGTH

11.5.1 General: Consider axial force, moment, and shear

11.5.2 Axial load and in-plane or out-of-plane flexure: Bearing walls 22.4 or 11.5.3; Moment in nonbearing walls 22.3
11.5.3 Axial load and out-of-plane flexure – simplified design method

Nominal axial capacity for wall if the resultant of all factored loads is located within the middle third of a solid wall with rectangular cross section: \( P_n = 0.55 f'_c A_g \left[ 1 - \left( \frac{k l_c}{32h} \right)^2 \right] \),

- \( k = 0.8 \) one end fixed other pinned,
- \( k = 1.0 \) both ends pinned,
- \( k = 2.0 \) cantilever
11.5.4.1 Nominal shear capacity by 11.5.4.2 through 11.5.4.4 or strut-and-tie Chapter 23. Reinforcement limits of 11.6, 11.7.2, and 11.7.3.

11.5.4.2 Nominal wall in-plane shear capacity $\leq 8\sqrt{f'_c}(\text{gross wall area in a horizontal section})$
11.5.4.3 Nominal wall in-plane shear capacity calculated by: For normal weight concrete with wall height to length ratio LE 1.5: \( V_n = (\text{Gross wall area in a horizontal section}) \times (3\sqrt{f'_c}) + \text{steel yield stress times the area of horizontal steel reinforcing per vertical inch / wall thickness}) \)

The "3" in the above equation is reduced to "2" at \( h_w/l_w = 2 \) and above, and as low as zero if the wall has axial tension.

11.5.5 Out-of-plane shear: Nominal capacity according to 22.5
Table 11.6.1 Minimum reinforcement cast-in-place and precast walls, transverse and longitudinal steel ratios required: values range 0.001 to 0.0025 (safe to use minimum values 0.0025 for both)
11.7 – REINFORCEMENT DETAILING

11.7.1 General: Cover 20.5.1; Development lengths 25.4; Splices 25.5.

11.7.2 Spacing of longitudinal reinforcement

11.7.2.1 Spacing $s$ of longitudinal bars in cast-in-place walls shall not exceed the lesser of $3h$ and 18 inches. If shear reinforcement is required for in-plane strength, spacing of longitudinal reinforcement shall not exceed $l_{w}/3$. 
11.7.2.2 Spacing $s$ of longitudinal bars in precast walls shall not exceed the lesser of $5h$ and 18 inches for exterior walls and 30 inches for interior walls. If shear reinforcement is required for in-plane strength, $s$ shall not exceed the smallest of $3h$, 18 inches, $l_w/3$.

11.7.2.3 For walls with thickness greater than 10 inches, except single story basement walls and cantilever retaining walls, distributed reinforcement in each direction shall be placed in at least two layers, one near each face.
11.7.3.1 Spacing $s$ of transverse reinforcement in cast-in-place walls shall not exceed the lesser of $3h$ and 18 inches. If shear reinforcement is required for in-plane strength, spacing of longitudinal reinforcement shall not exceed $l_w/5$.

11.7.3.2 Spacing $s$ of transverse bars in precast walls shall not exceed the lesser of $5h$ and 18 inches for exterior walls and 30 inches for interior walls. If shear reinforcement is required for in-plane strength, $s$ shall not exceed the smallest of $3h$, 18 inches, $l_w/5$. 

11.7.3 - SPACING OF TRANSVERSE REINFORCEMENT
11.7.4 Lateral support of longitudinal reinforcement: If longitudinal reinforcement is required for compression and exceeds one percent of the gross concrete area, it shall be laterally supported by transverse ties.

11.7.5 Reinforcement around openings: Add #5 bars developed at corners.

11.8 – Alternate method for out-of-plane slender wall analysis: Simply supported axially loaded member subject to an out-of-plane uniformly distributed lateral load, with maximum moments and deflections occurring at midheight.
CHAPTER 12 – DIAPHRAGMS

(Generally cast-in-place floor slabs acting as thin deep beams to transfer lateral loads)
12.1 - Scope: Nonprestressed and prestressed cast-in place slabs, topping slabs on precast slabs, other precast systems. Diaphragms in Seismic Design Categories D, E, and F must also satisfy 18.12.

12.2 - General: Design shall consider: In-plane forces due to lateral loads; transfer forces; forces at connections to vertical framing or bracing; out-of-plane forces due to gravity or other source. Consider effect of slab openings. Concrete properties according to Chapter 19. Steel properties according to Chapter 20.
12.3 DESIGN LIMITS

12.4 REQUIRED STRENGTH

12.3 – Design limits: Thickness as required for stability, strength, and stiffness under factored load combinations.

12.4 – Required strength: Load combinations of Chapter 5; consider effect of simultaneous out-of-plane loads.
12.5.1.3 (a) For a diaphragm idealized as a beam whose depth is equal to the full diaphragm depth, with moment resisted by boundary reinforcement concentrated at the diaphragm edges, design strengths shall be in accordance with 12.5.2 through 12.5.4.

12.5.2 Moment and axial force: It shall be permitted to design a diaphragm to resist in-plane moment and axial force in accordance with 22.3 and 22.4.
12.5.3 Shear

In-plane shear; capacity reduction factor 0.75.

Nominal in-plane shear strength for cast-in-place slabs of normal weight concrete:

\[ V_n = (\text{slab thickness})(\text{slab plan dimension in the direction of the load – openings = “depth”})(2\sqrt{f'_c} + (\text{area of steel reinforcing parallel to load per inch of slab width perpendicular to load / slab thickness}) (\text{steel yield stress})) \]

\[ f'_c \leq 100\text{psi} ; V_n \text{ limited to } (0.75)(8)\sqrt{f'_c}(\text{slab thickness})(\text{slab depth}) \]
12.5.4 Collectors

12.5.4.1 Collectors shall extend from the vertical elements of the lateral-force-resisting system across all or part of the diaphragm depth as required to transfer shear from the diaphragm to the vertical element.

12.5.4.2 Collectors shall be designed as tension members, compression members, or both, in accordance with 22.4.
12.6 – Reinforcement limits: Shrinkage and temperature according to 24.4 can also be used to resist diaphragm in-plane forces; one-way slab limits in 7.6; two-way slab limits 8.6
12.7 – REINFORCEMENT DETAILING

12.7.1 General: Cover 20.5.1; Development 25.4 or Chapter 18; Splices 25.5; Bundled bars 25.6

12.7.2 Reinforcement spacing: Minimum spacing 25.2; maximum spacing the lesser of $5t$ and 18 inches
"IN-PLANE" lateral loads

Concrete slab "diaphragm"

Biaxial or triaxial shear

Dead + Live Loads "OUT-OF-PLANE"

Shear wall "Vertical framing"

"Collector" Beam "Boundary Element"

Plan

Check

$1.2D + 1.6L - $ Diaphragm forces not required

$1.2D + 1.0L + 1.0 (W ORE)$
CHAPTER 13 – FOUNDATIONS
13.1 SCOPE

13.1 – Scope: Strip footings, Isolated footings, Combined footings, Mat foundations, Grade beams, Pile caps, Piles, Drilled piers, Caissons, Cantilever retaining walls, Counterfort and buttressed cantilever retaining walls.
13.2 GENERAL

13.2.1 Materials: Concrete properties Chapter 19; Steel reinforcement Chapter 20; Embedments 20.6.
13.2.2 Connection to other members: 16.3
13.2.3 Earthquake effects: 18.2.2.3; Seismic Design Categories C, D, E, F 18.13.
13.2.4 Slabs-on-ground: If part of seismic-force-resisting system 18.13.
13.2.5 Plain concrete: Chapter 14.
13.2.6.1 Foundations shall be proportioned for bearing effects, stability against overturning and sliding at the soil-foundation interface in accordance with the general building code.

13.2.6.2 For one-way shallow foundations, two-way isolated footings, or two-way combined footings and mat foundations, it is permissible to neglect the size effect factor specified in 22.5 for one-way shear strength and 22.6 for two-way shear strength.
13.2.6.5 Foundation design by strut-and-tie method: Chapter 23.

13.2.6.6 External moment on any section of a strip footing, isolated footing, or pile cap shall be calculated by passing a vertical plane through the member and calculating the moment of the forces acting over the entire area of the member on one side of that vertical plane.
13.2.7 Critical sections for shallow foundations and pile caps

Table 13.2.7.1

Supported member: Location of critical section for $M_u$
- Column or pedestal: Face
- Column with steel base plate: Halfway between face and edge of steel base plate
- Concrete wall: Face

Masonry wall: Halfway between center and face of masonry wall
13.2.7.2 The location of critical section for factored shear in accordance with 7.4.3 and 8.4.3 for one-way shear or 8.4.4.1 for two-way shear shall be measured from the location of the critical section for $M_u$.

13.2.7.3 Circular or regular polygon-shaped concrete columns or pedestals shall be permitted to be treated as square members of equivalent area when locating critical sections for moment, shear, and development of reinforcement.
13.2.8 Development of reinforcement in shallow foundations and pile caps

13.2.8.1 Development of reinforcement: Chapter 25

13.2.8.2 Calculated tensile or compressive forces in reinforcement at each section shall be developed on each side of that section.

13.2.8.3 Critical sections for development of reinforcement as in 13.2.7.1 for moment and at all other vertical planes where changes of section or reinforcement occur.
13.3 – SHALLOW FOUNDATIONS

13.3.1 General: Size foundation for acceptable bearing pressures; Minimum “d” for bottom reinforcement is 6 inches.

13.3.2 One-way shallow foundations (strip footings, combined footings, grade beams): Must also satisfy Chapters 7 and 9; Distribute reinforcement uniformly across width.
13.3 – SHALLOW FOUNDATIONS (CONT’D)

13.3.3 Two-way isolated footings
13.3.3.1 Must also satisfy Chapters 7 and 8.
13.3.3.2 In square two-way footings, reinforcement shall be distributed uniformly across entire width of footing in both directions.
13.3.3.3 In rectangular footings:
(a) Reinforcement in the long direction distributed uniformly across width.
(b) In short direction, fraction $\frac{2}{1+\text{ratio of long footing dimension to short dimension}}$ of total steel reinforcing required shall be uniformly distributed over a strip of short footing dimension centered on the column. Remainder of reinforcing uniformly distributed over areas outside this strip.
13.3.4 Two-way combined footings and mat foundations
13.3.4.1 Must also satisfy Chapter 8.
13.3.4.2 Direct design method not permitted.
13.3.4.4 Minimum reinforcement in accordance with 8.6.1.1.

13.3.5 Walls as grade beams: Chapter 9; minimum reinforcement 11.6.

13.3.6 Wall components of cantilever retaining walls: Chapters 7 and 8, as applicable.
13.4 – DEEP FOUNDATIONS

13.4.1 General
13.4.1.1 Size foundation to satisfy geotechnical requirements
13.4.1.2 Design of deep foundation members shall be in accordance with 13.4.2 or 13.4.3.
13.4.2 Allowable axial strength
13.4.2.1 It shall be permitted to design a deep foundation member using load combinations for allowable stress design in ASCE/SEI 7, section 2.4, and the allowable strength specified in Table 13.4.2.1 if (a) and (b) are satisfied:

(a) The deep foundation member is laterally supported for its entire height

(b) The applied forces cause bending moments in the deep foundation member less than the moment due to an accidental eccentricity of 5 percent of the member diameter or width.
13.4 – DEEP FOUNDATIONS (CONT’D)

Table 13.4.2.1 – Maximum allowable compressive strength of deep foundation members

Uncased cast-in-place concrete drilled or augured pile: \( P_a = 0.3f'_c A_g + 0.4f_y A_s \)

... 

Precast prestressed concrete pile: \( P_a = (0.33f'_c - 0.27f_{pc}) A_g \)
13.4.3 Strength design

13.4.3.2 The strength design of deep foundation members shall be in accordance with 10.5 using the compressive strength reduction factors of Table 13.4.3.2 for axial load without moment, and the strength reduction factors of Table 21.2.1 for tension, shear, and combined axial force and moment. The provisions of 22.4.2.4 and 22.4.2.5 shall not apply to deep foundations.
13.4 – DEEP FOUNDATIONS (CONT’D)

Table 13.4.3.2 – Compressive strength reduction factors for deep foundation members

Uncased cast-in-place concrete drilled or augered pile: 0.55

... Precast prestressed concrete pile: 0.65

13.4.5 Precast concrete piles: Section applies to Seismic Design Categories A and B. (C-F 18.13.5.10)
13.4.5.3 For precast nonprestressed piles: Minimum 4 longitudinal bars and 0.008 reinforcement ratio

13.4.5.4 For precast prestressed piles, effective prestress after assumed 30ksi loss: 400psi for piles under 30ft length, 550psi for 30-50ft, and 700psi for lengths over 50ft.
13.4.5.6 Minimum transverse reinforcement enclosing longitudinal reinforcement:
Least horizontal pile dimension LE 16 inches: W4,D4
16 to 20 inches: W4.5, D5
Over 20 inches: W5.5, D6
Maximum spacing: First five ties or spirals each pile end, 1 inch center to center; end 24 inches, 4 inch; rest of pile, 6 inches.
13.4.6 Pile caps

13.4.6.1 Minimum “d” for bottom steel 12 inches.

13.4.6.2 Pile reactions may be assumed to be concentrated at the pile centroid.

13.4.6.3 Except for pile caps designed in accordance with 13.2.6.5, the pile cap shall be designed such that (a) is satisfied for one-way foundations and (a) and (b) are satisfied for two-way foundations.
13.4 – DEEP FOUNDATIONS (CONT’D)

(a) $0.75 V_n \geq V_u$, where $V_n$ shall be calculated in accordance with 22.5 for one-way shear.

(b) $0.75 v_n \geq v_u$, where $v_n$ shall be calculated in accordance with 22.6 for two-way shear.

(Note: $V_u$ and $v_u$ references to 13.4.2.7 are errors since there is no such section.)
13.4.6.4 Strut-and-tie method concrete compressive strength of struts in accordance with 23.4.3 and 19.2.4.

13.4.6.5 Calculation of factored shear on any section through a pile cap shall be in accordance with (a) through (c):
(a) Entire reaction from any pile with its center located $d_{\text{pile}}/2$ or more outside the section shall be considered as producing shear on that section.

(b) Reaction from any pile with its center located $d_{\text{pile}}/2$ or more inside the section shall be considered as producing no shear on that section.

(c) Linear interpolation
CHAPTER 14 – PLAIN CONCRETE
14.1.3 Plain concrete shall be permitted only in cases (a) through (d):

(a) Members that are continuously supported by soil or other...
(b) Members for which arch action provides compression under all conditions of loading.
(c) Walls
(d) Pedestals
14.1 SCOPE (CONT’D)

14.1.4 Further restrictions for Seismic Design Categories D, E, F

14.1.5 Plain concrete shall not be permitted for columns and pile caps.

14.2 GENERAL: Concrete properties Chapter 19; Steel reinforcement Chapter 20; Embedments 20.6.
14.3 – DESIGN LIMITS

14.3.1 Bearing walls: Minimum thickness the greater of 5.5 inches and 1/24 the lesser of unsupported length and unsupported height, and GE 7.5 inches for exterior basement walls or foundations.

14.3.2 Footing thickness GE 8 inches.

14.3.3 Pedestals: Ratio of unsupported height to average least lateral dimension shall not exceed 3.
14.3 – DESIGN LIMITS (CONT’D)

14.4 REQUIRED STRENGTH

14.3.4 Contraction and isolation joints: Provided to limit stress caused by restraint to movements from creep, shrinkage, and temperature effects.

14.4 Required Strength: Factored load combinations of Chapter 5 and analysis procedures of Chapter 6
14.5 – DESIGN STRENGTH

14.5.1.3 Tensile strength of concrete shall be permitted to be considered in design. \((5\sqrt{f'_c})\)

14.5.1.6 No strength shall be assigned to steel reinforcement.

14.5.6 Bearing: \(B_n = 0.85 f'_c A_1\) or up to double this if supporting surface is wider on all sides than the loaded area (increase factor is \(\sqrt{A_2/A_1}\))
CHAPTER 15 – BEAM – COLUMN AND SLAB – COLUMN JOINTS
15.1 - Scope: This chapter shall apply to the design and detailing of cast-in-place beam-column and slab-column joints.

15.2 - General

15.2.1 Beam-column joints shall satisfy the detailing provisions of 15.3 and strength requirements of 15.4.

15.2.2 Beam-column and slab-column joints shall satisfy 15.5 for transfer of column axial force through the floor system.
15.2.5 If the beam framing into the joint and generating joint shear has depth exceeding twice the column depth, analysis and design of the joint shall be based on the strut-and-tie method in accordance with Chapter 23...

15.2.8 A beam-column joint shall be considered to be confined for the direction of the joint shear considered if two transverse beams satisfying ......
15.2.9 For slab-column connections transferring moment, strength and detailing requirements shall be in accordance with applicable provisions in Chapter 8 and Sections 15.3.2 and 22.6.

15.3 – Detailing of joints

15.3.1 Beam-column joint transverse reinforcement
15.3.1.1 Beam-column joints shall satisfy 15.3.1.2 through 15.3.1.4 unless (a) through (c) are satisfied:

(a) Joint is considered confined by transverse beams in accordance with 15.2.8 for all shear directions considered

(b) .....not part of a designated seismic-force-resisting system

(c) .....not .... SDC D, E, or F
15.3 – DETAILING OF JOINTS (CONT’D)

15.3.1.4 Spacing of joint transverse reinforcement shall not exceed 8 in. within the depth of the deepest beam framing into the joint.

15.3.2 Slab-column joint transverse reinforcement

15.3.2.1 Except where laterally supported on four sides by a slab, column transverse reinforcement shall be continued through a slab column joint, including...
15.4 – Strength requirements for beam-column joints

15.4.1 Required shear strength

15.4.1.1 Joint shear force $V_u$ shall be calculated on a plane at mid-height of the joint using flexural tensile and compressive beam forces and column shear consistent with (a) or (b):
15.4 – STRENGTH REQUIREMENTS

(a) The maximum moment transferred between the beam and column as determined from factored load analysis for beam-column joints with continuous beams in the direction of joint shear considered

(b) Beam nominal moment strengths $M_n$

Table 15.4.2.3 – Nominal joint shear strength $V_n$

Example: Unconfined top of corner column of moment frame:

$$V_n = 12 \sqrt{f'_c}A_j$$
15.4.2.4 Effective cross-sectional area within a joint, $A_j$, shall be calculated as the product of joint depth and effective joint width. Joint depth shall be the overall depth of the column, $h$, in the direction of joint shear considered. Effective joint width shall be the overall width of the column where the beam is wider than the column. Where the column is wider than the beam, effective joint width shall not exceed the lesser of (a) and (b):
15.4 – STRENGTH REQUIREMENTS (CONT’D)

15.5 TRANSFER OF COLUMN AXIAL FORCE

15.4 – STRENGTH REQUIREMENTS (CONT’D)

15.5 TRANSFER OF COLUMN AXIAL FORCE

(a) Beam width plus joint depth
(b) Twice the perpendicular distance from longitudinal axis of beam to nearest side face of the column

15.5 – Transfer of column axial force through the floor system

15.5.1 If $f_c'$ of a floor system is less than $0.7f_c'$ of a column....
CHAPTER 16 – CONNECTIONS BETWEEN MEMBERS
16.1 - Scope: Precast concrete; foundations; brackets and corbels.

16.2 – Connections of precast members

16.3 – Connections to foundations

16.4 – Horizontal shear transfer in composite concrete flexural members

16.5 – Brackets and Corbels
CHAPTER 17 – ANCHORING TO CONCRETE
17.1.1 This chapter shall apply to the design of anchors in concrete used to transmit loads by means of tension, shear, or a combination of tension and shear between: (a) connected structural elements; or (b) safety-related attachments and structural elements...
17.1.2 Provisions of this chapter shall apply to the following anchor types (a) through (g):

(a) Headed studs and headed bolts....

(b) Hooked bolts...

(c) Post-installed expansion anchors...

(d) Post-installed adhesive anchors...

(f) Post-installed screw anchors...

(g) Attachments with shear lugs..

(New to ACI 318)

(New to ACI 318)
17.2 – GENERAL

17.2.1 Anchors and anchor groups shall be designed for critical effects of factored loads calculated by elastic analysis. If nominal strength is controlled by ductile steel elements, plastic analysis is permitted...

17.2.1.1 Anchor group effects shall be considered if two or more anchors loaded by a common structural element are spaced closer than the spacing required for unreduced breakout strength...
17.3 – DESIGN LIMITS
17.4 REQUIRED STRENGTH

17.3.1 The value of \( f_c \) ‘ LE 10 ksi for cast-in anchors  LE 8 ksi for post-installed.

17.4 – Required strength: Chapter 5 Load Combinations; Also section 17.10 for Seismic Design Categories C, D, E, and F,
17.5 – DESIGN STRENGTH

17.5.1.2 .... The nominal strength shall be based on the 5 percent fractile of the basic individual anchor strength....Strength of anchors shall be based on design models that satisfy 17.5.1.2 for the following:

(a) Steel strength of anchor in tension

(b) Concrete breakout strength of anchor in tension

(c) Pullout strength of a single cast-in anchor and single post-installed expansion, screw, or undercut anchor in tension
17.5 – DESIGN STRENGTH (CONT’D)

(d) Concrete side-face blowout strength of headed anchor in tension
(e) Bond strength of adhesive anchor in tension
(f) Steel strength of anchor in shear
(g) Concrete breakout strength of anchor in shear
(h) Concrete pryout strength of anchor in shear
Fig. R17.5.1.2—Failure modes for anchors.

R17.5.1.3 The method for concrete breakout design deemed to comply with the requirements of 17.5.1.2 was developed from the concrete capacity design (CCD) Method (Fuchs et al. (1993), Eligehausen and Balogh (1993), which was an adaptation of the Kappa Method (Eligehausen and Fuchs 1988, Eligehausen et al. 2006a) with a breakout failure surface angle of approximately 35 degrees (Fig.
17.5.2.1 The design strength of anchor reinforcement shall be permitted to be used instead of the concrete breakout strength if (a) or (b) is satisfied.

(a) For tension, if anchor reinforcement is developed in accordance with Chapter 25 on both sides of the concrete breakout surface.

(b) For shear, if anchor reinforcement is developed in accordance with Chapter 25 on both sides of the concrete breakout surface, or encloses and contacts the anchor and is developed beyond the breakout surface.
17.6 – TENSILE STRENGTH
17.7 SHEAR STRENGTH

17.6 – Tensile strength – based on 35 degree angle between breakout surface and exterior surface, 1:1.5. This gives a square plan view of the breakout surface of dimensions $3h_{ef} \times 3h_{ef}$ where $h_{ef}$ is the effective embedment depth of the anchor.

17.7 – Shear strength: 35 degree breakout angle also applies for shear.
17.8 – Tension and shear interaction

17.9 – Edge distances, spacings, and thicknesses to preclude splitting failure

17.10 – Earthquake-resistant anchor design requirements

SDC C, D, E, or F

17.10.5.4 Most strengths subject to additional 0.75 reduction
Section A-A

Fig. R17.52.1-Anchor reinforcement for tension.
Fig. R17.5.2.1b(a)—Hairpin anchor reinforcement for shear.
CODE

17.5.1.3.1 Anchor group effects shall be considered wherever two or more anchors have spacing less than the critical spacing in Table 17.5.1.3.1, where only those anchors susceptible to the particular failure mode under investigation shall be included in the group.

Table 17.5.1.3.1—Critical spacing

<table>
<thead>
<tr>
<th>Failure mode under investigation</th>
<th>Critical spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete tensile in tension</td>
<td>$N_{cf}$</td>
</tr>
<tr>
<td>Bond strength in tension</td>
<td>$N_{bm}$</td>
</tr>
<tr>
<td>Concrete breakout in shear</td>
<td>$N_{ct}$</td>
</tr>
</tbody>
</table>

17.5.1.4 Strength of anchors shall be permitted to be based on test evaluation using the 5 percent fractile of applicable test results for 17.5.1.2 (a) through (b).

COMMENTARY

17.5.1.3a and b). It is considered to be sufficiently accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD Method predicts the strength of an anchor or anchor group by using a basic equation for tension in cracked concrete, which is multiplied by factors that account for the number of anchors, edge distance, spacing, eccentricity, and absence of cracking. For shear, a similar approach is used. Experimental and numerical investigations have demonstrated the applicability of the CCD Method to adhesive anchors as well (Engelshausen et al. 2006).

Fig. R17.5.1.3a—Breakout cone for tension.

Fig. R17.5.1.3b—Breakout cone for shear.
The critical edge distance for headed studs, headed bolts, expansion anchors, screw anchors, and undercut anchors is $1.5h_{ef}$.

**Code**

![Diagram of a section through failure cone with symbols and equations]

**Commentary**

If $c_{ef} < 1.5h_{ef}$ and $s_{ef} < 3h_{ef}$,

$$A_{NO} = (c_{ef} + 1.5h_{ef}) \times (2 \times 1.5h_{ef})$$

If $c_{ef} < 1.5h_{ef}$ and $s_{ef} < 3h_{ef}$,

$$A_{NO} = (c_{ef} + 1.5h_{ef}) \times (2 \times 1.5h_{ef})$$

Fig. R17.6.2.1—(a) Calculation of $A_{NO}$, and (b) calculation of $A_{NO}$ for single anchors and anchor groups.
CODE

If $h_a < 1.5c_{at}$
$A_{vc} = 2(1.5c_{at})h_a$

If $h_a < 1.5c_{at}$ and $s_1 < 3c_{at}$
$A_{vc} = 2(1.5c_{at}) + s_1h_a$

COMMENTARY

If $c_{at} > 1.5c_{at}$
$A_{vc} = 1.5c_{at}(1.5c_{at} + c_{at})$

Case 1: One assumption of the distribution of forces indicates that half of the shear force would be critical on the front anchor and the projected area. For the calculation of concrete breakout, $c_{at}$ is taken as $c_{at,1}$.

Case 2: Another assumption of the distribution of forces indicates that the total shear force would be critical on the rear anchor and its projected area. Only this assumption needs to be considered when anchors are welded to a common plate independent of $s$. For the calculation of concrete breakout, $c_{at}$ is taken as $c_{at,2}$.

Note: For $s \geq c_{at,1}$, both Case 1 and Case 2 should be evaluated to determine which controls for design except as noted for anchors welded to a common plate.

Case 3: Where $s < c_{at,1}$, apply the entire shear load $V$ to the front anchor. This case does not apply for anchors welded to a common plate. For the calculation of concrete breakout, $c_{at}$ is taken as $c_{at,1}$.

Fig. R17.7.3.1b—Calculation of $A_{vc}$ for single anchors and anchor groups.
CHAPTER 18
EARTHQUAKE - RESISTANT STRUCTURES
18.1.1 This chapter shall apply to the design of nonprestressed and prestressed concrete structures assigned to Seismic Design Categories B through F.

18.1.2 Structures designed according to the provisions of this chapter are intended to resist earthquake motions through ductile inelastic response of selected members.
18.2 – General: SDC B shall satisfy 18.2.2; SDC C shall satisfy 18.2.2, 18.2.3, and 18.13; SDC D, E, and F shall satisfy 18.2.2 through 18.2.8 and 18.12 through 18.14.

18.2.1.6 Structural systems designated as part of the seismic-force-resisting system shall be restricted to those designated by the general building code... (a) through (h) shall be satisfied...
18.2 - GENERAL (CONT’D)

(a) Ordinary moment frames 18.3
(c) Intermediate moment frames 18.4
(e) Special moment frames 18.2.3 through 18.2.8 and 18.6 through 18.8.
(g) Special structural walls 18.2.3 through 18.2.8 and 18.10
18.2.2
18.2.2.1 The interaction of all structural and nonstructural members that affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

18.2.3
18.2.3.1 Anchors resisting earthquake-induced forces assigned to SDC C, D, E, or F shall be in accordance with 17.10.
18.13.4 Foundation seismic ties

18.13.4.1 For structures assigned to SDC C, D, E, or F, individual pile caps, piers, or caissons shall be interconnected by foundation seismic ties in orthogonal directions, unless it can be demonstrated that equivalent restraint is provided by other means.
18.13.4.3 Where required, foundation seismic ties shall have a design strength in tension and compression at least equal to 0.1 \( S_{DS} \) times the greater of the pile cap factored dead load plus factored live load unless...
CHAPTER 19
CONCRETE: DESIGN AND DURABILITY REQUIREMENTS
19.2 – Concrete design properties

19.2.1 Specified compressive strength: $f'_c$ based on 28-day tests

19.2.2 Modulus of elasticity: For normal weight concrete, $E_c$, ksi = $57\sqrt{f'_c}$ psi

19.2.3 Modulus of rupture: For normal weight concrete, $f_r$, psi = $7.5\sqrt{f'_c}$ psi
19.3 – Concrete durability requirements

Table 19.3.2.1 – Requirements for concrete by exposure class

For reinforcement corrosion class C2 “Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources”, minimum $f'_c = 5\text{ksi}$ and maximum water to cement weight ratio is 0.4.
### Code

#### Table 19.3.2.1 — Requirements for concrete by exposure class

<table>
<thead>
<tr>
<th>Exposure class</th>
<th>Minimum $w/c$</th>
<th>Maximum $f_{c}$, psi</th>
<th>Additional requirements</th>
<th>Limits on cementitious materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>P0</td>
<td>N/A</td>
<td>2500</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>F1</td>
<td>0.65</td>
<td>3500</td>
<td>Table 19.3.3.1 for concrete or Table 19.3.3.3 for short-time</td>
<td>N/A</td>
</tr>
<tr>
<td>F2</td>
<td>0.45</td>
<td>4500</td>
<td>Table 19.3.3.1 for concrete or Table 19.3.3.3 for short-time</td>
<td>N/A</td>
</tr>
<tr>
<td>F3</td>
<td>0.40(^{(a)})</td>
<td>5000(^{(a)})</td>
<td>Table 19.3.3.1 for concrete or Table 19.3.3.3 for short-time</td>
<td>26.4.2.20(a)</td>
</tr>
</tbody>
</table>

#### Commentary

<table>
<thead>
<tr>
<th>Type</th>
<th>Cementitious materials(^{(a)})</th>
<th>Calciumchloride admixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>S0</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>S1</td>
<td>0.50</td>
<td>4500</td>
</tr>
<tr>
<td>S2</td>
<td>0.45</td>
<td>4500</td>
</tr>
<tr>
<td>S3</td>
<td>Option 1</td>
<td>4500</td>
</tr>
<tr>
<td></td>
<td>Option 2</td>
<td>5000</td>
</tr>
</tbody>
</table>

| W0   | N/A                             | 2500                       |
| W1   | N/A                             | 2500                       |
| W2   | 0.50                            | 4500                       |

<table>
<thead>
<tr>
<th>Non-prestressed concrete</th>
<th>Prestressed concrete</th>
<th>Additional provisions</th>
</tr>
</thead>
<tbody>
<tr>
<td>C0</td>
<td>N/A</td>
<td>1.00</td>
</tr>
<tr>
<td>C1</td>
<td>N/A</td>
<td>0.50</td>
</tr>
<tr>
<td>C2</td>
<td>0.40</td>
<td>0.15</td>
</tr>
</tbody>
</table>

\(^{(a)}\)The value is based on all cementitious and supplementary cementitious materials in the concrete mixture.

\(^{(b)}\)The maximum water-cement ratio shall be 0.45 or the minimum, whichever is smaller, shall be 4500 psi.

\(^{(c)}\)Alternative combinations of cementitious materials to those listed are permitted for all sulfate exposure classes when used for sulfate resistance and meeting the criteria in 26.4.2.6\(a\).

\(^{(d)}\)For sulfate exposure, other types of portland cements with tricalcium aluminate (CA) content up to 10 percent are permitted if the above does not exceed 0.40.

\(^{(e)}\)Other available types of cement such as Type II or Type III CEMI cement are permitted Exposure Classes S1 or S2 if the CA content is less than 8 percent for Exposure Class S1 or less than 5 percent for Exposure Class S2.

\(^{(f)}\)The amount of the specific source of the portland cement or slag cement to be used shall be at least the amount that has been determined by the owner to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the portland cement or slag cement to be used shall be at least 20 percent in accordance with ASTM C1012 and meeting the criteria in 26.4.2.6\(e\).

\(^{(g)}\)Type V cement used as the basic cementitious material, the optional sulfate resistance requirement of 0.040 percent maximum expansion in ASTM C1012 shall be specified.

\(^{(h)}\)The sum of the supplementary cementitious materials used to determine the chloride content shall not exceed the sum of the portland cement.

\(^{(i)}\)Differences in the chloride content are in 26.4.2.6\(f\).

**19.3.3 Additional requirements for freezing-and-thawing exposure**

**19.3.3.1 Concrete subject to freezing-and-thawing Exposure Classes F1, F2, or F3 shall be air entrained.**

**R19.3.3.3 Additional requirements for freezing-and-thawing exposure**

**R19.3.3.3.1 A table of required air contents for concrete to resist damage from cycles of freezing and thawing.**
CHAPTER 20
STEEL REINFORCEMENT PROPERTIES, DURABILITY, & EMBEDMENTS
20.2.1.2 Yield strength by (a) or (b)
(a) 0.2% offset method

20.2.1.3 ASTM A615 Grades 40, 60, 80, 100; ASTM A706 – low alloy steel Grades 60, 80, 100

(enhanced weldability for Grades 60 and 80)
20.2.2 DESIGN PROPERTIES

20.2.2.1 For nonprestressed bars and wires, the stress below $f_y$ shall be $E_s$ times steel strain. For strains greater than that corresponding to $f_y$, stress shall be considered independent of strain and equal to $f_y$.

20.2.2.2 $E_s = 29000$ksi

Table 20.2.2.4(a) – Nonprestressed deformed reinforcement: For shear and torsion, $f_{y\text{ max}} = 60$ ksi
20.3.2.3 Stress in bonded prestressed reinforcement at nominal flexural strength, $f_{ps}$

$$f_{ps} = f_{pu} \cdot [1 - \ldots] \text{ (often about } 0.93 f_{pu} = 0.93(270) = 250 \text{ksi})$$

20.4 – Headed shear stud reinforcement
20.5 – Provisions for durability of steel reinforcement

20.5.1 Specified concrete cover

Table 20.5.1.3.1 – Specified concrete cover for cast-in-place nonprestressed concrete members

Table 20.5.1.3.2 – Specified concrete cover for cast-in-place prestressed concrete members
20.5 DURABILITY (CONT’)

Table 20.5.1.3.3 – Specified concrete cover for precast-nonprestressed or prestressed concrete members manufactured under plant conditions

Table 20.5.1.3.4 – Specified concrete cover for deep foundation members
CHAPTER 21

STRENGTH REDUCTION FACTORS
Table 21.2.1 – Strength reduction factors “phi”

Shear and torsion: 0.75

Bearing: 0.65

Plain concrete: 0.60

Anchors 0.45 to 0.75 in accordance with Chapter 17
21.2.2.1 For deformed reinforcement the yield strain shall be the yield stress divided by the modulus of elasticity. For Grade 60 deformed reinforcement, it shall be permitted to be taken as 0.002.

21.2.2.2 For all prestressed reinforcement, the tensile yield strain shall be taken as 0.002.
Table 21.2.2 – Strength reduction factors for moment, axial force, or combined moment and axial force

Compression-controlled, ie strain in extreme tension side steel reinforcing LE the yield strain in tension:

\[ \Phi = 0.65 \text{ except } \phi = 0.75 \text{ if longitudinal steel is confined by spirals} \]
Tension controlled, i.e. strain in extreme tension side steel reinforcing GE yield strain + 0.003 in tension: \( \phi = 0.9 \).

Linear Transition
CHAPTER 22 – SECTIONAL STRENGTH
22.1 SCOPE

22.1.1 This chapter shall apply to calculating nominal strength at sections of members, including (a) through (g):

(a) Flexural strength

(b) Axial strength or combined flexural and axial strength

(c) One-way shear strength
22.1 SCOPE
(CONT’D)

(d) Two-way shear strength
(e) Torsional strength
(f) Bearing
(g) Shear friction
22.1.2 Sectional strength requirements of this chapter shall be satisfied unless the member or region of the member is designed in accordance with Chapter 23.

22.1.3 Design strength at a section shall be taken as the nominal strength multiplied by the applicable strength reduction factor given in Chapter 21.
22.2 – Design assumptions for moment and axial strength

22.2.1.2 Strain in concrete and nonprestressed reinforcement shall be assumed proportional to the distance from the neutral axis.

22.2.1.3 Strain in prestressed concrete and in bonded and unbonded prestressed reinforcement shall include the strain due to effective prestress.

22.2.1.4 Changes in strain for bonded prestressed reinforcement shall be assumed proportional to the distance from the neutral axis.
22.2.2 Design assumptions for concrete

22.2.2.1 Maximum strain at the extreme concrete compression fiber shall be assumed equal to 0.003.

22.2.2.2 Tensile strength of concrete shall be neglected in flexural and axial strength calculations.
22.2.2 The relationship between concrete compressive stress and strain shall be represented by a rectangular .... or other shape that results in prediction of strength in substantial agreement with...tests. (at instant that maximum concrete compressive strain is 0.003, ie at “failure”)

22.2.2.4 The equivalent rectangular concrete stress distribution:
concrete stress $0.85f'_{c}$; depth of compression block “ $\beta_{1}c$”, $\beta_{1} = 0.85$ for $f'_{c}$ LE 4 ksi; $\beta_{1} = 0.65$ for $f'_{c}$ GE 8 ksi; linear interpolation
22.2.4 – Design assumptions for prestressed reinforcement

22.2.4.1 For members with bonded prestressed reinforcement conforming to 20.3.1, stress at nominal flexural strength, $f_{ps}$, shall be calculated in accordance with 20.3.2.3.

$$f_{ps} = f_{pu} \left( 1 - \ldots \right) \quad f_{pu} = 270 \text{ ksi for ASTM A416 Stress-relieved and low-relaxation strand}$$

22.2.4.2 For members with unbonded prestressed reinforcement conforming with 20.3.1, $f_{ps}$ shall be calculated in accordance with 20.3.2.4.
Strain at Failure

\[ E_t \geq \frac{f_y}{E_0} + 0.003 \] (Tension)

\[ f_y \leq 0.1f'c'A_s \quad \text{(Neglect)} \]

\[ \sigma \leq 0.003 \left( \frac{d}{0.003 + E_t} \right) \]

\[ \varepsilon = 0.003 \]

Stress at Failure

\[ T = \text{Tension} = A_s f_y \]

\[ M_n = T (d - r) = C (d - r) \]

\[ f = 0.9 \]

\[ \beta = 0.85 \text{ for } f'c' \leq 4 \text{ ksi} \]

\[ \beta = 0.65 \text{ for } f'c' \geq 8 \text{ ksi} \]

- Linear Interpolation
22.4.2 Maximum axial compressive strength

22.4.2.1 Nominal axial compressive strength $P_n$ shall not exceed $P_{n,max}$ in accordance with Table 22.4.2.1, where $P_o = 0.85f'_cA_g + (f_y - 0.85f'_c)A_{st}$ for nonprestressed members and is calculated by Eq.(22.4.2.3) for prestressed members. The value of $f_y$ shall be limited to 80 ksi.
Table 22.4.2.1 – Maximum axial strength

Nonprestressed column with ties: $P_n \leq 0.80P_o$

Nonprestressed column with spiral: $P_n \leq 0.85P_o$
22.5 – ONE-WAY SHEAR STRENGTH

22.5.1 General

22.5.1.1 Nominal one-way shear strength at a section: $V_n = V_c + V_s$

22.5.1.2 Cross-sectional dimensions shall be selected to satisfy: $V_u \leq 0.75(V_c + 8\sqrt{f'_c}(b_wd))$

22.5.1.3 For nonprestressed members, $V_c$ calculated by 22.5.5
22.5.1.6 $V_s$ in accordance with 22.5.8

22.5.1.7 Openings must be considered when calculating $V_n$.

22.5.1.10 Biaxial shear must be considered if the shear stress in both orthogonal directions exceeds 0.5 of design capacity values; interaction equation is that the sum of the two factored shear stresses to design capacities $\leq 1.5$. 
22.5.2.2 For calculation of $V_c$ and $V_s$, it shall be permitted to assume (a) through (c):

(a) $d$ equal to 0.8 times the diameter for circular sections

(b) $b_w$ equal to the diameter for solid circular sections

(c) $b_w$ equal to twice the wall thickness for hollow circular sections
22.5.5 \( V_c \) for nonprestressed members

Table 22.5.5.1

Normal weight concrete, at least minimum stirrups, no axial force:
\[
V_c = \max \left( 2, \frac{8}{b_w d} \left( \frac{A_s}{b_w d} \right)^{0.333} \right) \sqrt{f'_c} b_w d, \quad \text{(equality at reinforcement ratio 0.0156)}
\]

Add axial compression stress/6 but not more than 0.05\( f'_c \), and \( V_c \) shall not be taken greater than 5\( \sqrt{f'_c} b_w d \). Subtract axial tension stress/6 but \( V_c \) shall not be taken less than zero.
Reduction for lightweight concrete 19.2.4, generally 0.75.

Size effect modification factor:
\[ \sqrt{\frac{2}{1 + 0.1d}} \leq 1.0 \]

For less than minimum stirrups, normal weight concrete, no axial force:

\[ V_c = 8\left(\frac{A_s}{b_w d}\right)^{0.333}(b_w d)(\text{Size effect}). \] (Same axial force adjustment as above)
22.5.6 $V_c$ for prestressed members

22.5.8 One-way shear reinforcement

22.5.8.1 At each section where $V_u \geq 0.75V_c$, transverse reinforcement shall be provided such that $V_s \geq (V_u / 0.75 - V_c)$
22.5.8.2 For one-way members reinforced with transverse reinforcement, $V_s$ shall be calculated in accordance with 22.5.8.5.
22.5.8.2 For one-way members reinforced with transverse reinforcement, \( V_s \) shall be calculated in accordance with 22.5.8.5.

22.5.8.5 One-way shear strength provided by transverse reinforcement: 
\[
V_s = A_v f_y t d/s
\]
22.6.1 General

22.6.1.1 Provisions 22.6.1 through 22.6.8 apply to the nominal shear strength of two-way members with and without shear reinforcement.

22.6.1.2 Nominal shear strength for two-way members without shear reinforcement shall be calculated by: \( v_n = v_c \), where \( v_c \) is the nominal stress capacity of the concrete when subjected to two-way shear, psi.
22.6.1.3 Nominal shear strength for two-way members with shear reinforcement shall be calculated by \( v_n = v_c + v_s \).

22.6.1.4 Two-way shear shall be resisted by a section with a depth \( d \) and an assumed critical perimeter \( b_o \) as defined in 22.6.4. (Generally: Columns to slab joints \( d = d^- \); column to footing or pilecap joints \( d = d^+ \).)
22.6.1.7 For two-way members reinforced with single or multiple leg stirrups, \( v_s \) shall be calculated in accordance with 22.6.7.

22.6.1.8 For two-way members reinforced with headed shear stud reinforcement, \( v_s \) shall be calculated in accordance with 22.6.8.
22.6.2 Effective depth: Use average value for both directions; not less than 0.8h for prestressed members.

22.6.3 Limiting material strengths: $\sqrt{f'_c}$ LE 100 psi; $f_{yt}$ LE 60 ksi
22.6.4 Critical sections for two-way members

22.6.4.1 For two-way shear, critical sections shall be located so that the perimeter $b_o$ is a minimum but need not be closer than 0.5d to (a) and (b):

(a) Edges or corners of columns, concentrated loads, or reaction areas

(b) Changes in slab or footing thickness, such as edges of capitals, drop panels, or shear caps.
22.6 - TWO WAY SHEAR STRENGTH (CONT’D)

22.6.4.1.2 For a circular or regular polygon-shaped column, critical sections for two-way shear... permitted to be defined assuming a square column of equivalent area.

22.6.4.2 For two-way members with shear reinforcement, also check a perimeter 0.5d beyond shear reinforcement.

22.6.4.3 Deduction for openings closer than 4h from edge of column
22.6.5 Two-way shear strength provided by concrete in members without shear reinforcement

Table 22.6.5.2 – \( v_c \) for two-way members without shear reinforcement

\( \beta \) is ratio of long to short side dimension of column, axial load, or reaction

\( \alpha_s \) equals 40 for interior columns, 30 for edge columns, and 20 for corner columns
\[ v_c \leq \text{least of (a), (b), or (c)} \]

(a) \( 4 \sqrt{f'_c} \) (Size effect) (Lightweight concrete factor)

(b) \( (2 + 4/\beta) \sqrt{f'_c} \) (Size effect) (Lightweight concrete factor)

(c) \( (2 + \alpha_s d/b_o) \sqrt{f'_c} \) (Size effect) (Lightweight concrete factor)
22.6.6 Two-way shear strength provided by concrete in members with shear reinforcement

Table 22.6.6.1 – $v_c$ for two-way members with shear reinforcement

Where minimum stirrups are present:  $v_c = 2\sqrt{f'_c}(\text{Size effect})(\text{Lightweight concrete factor})$
Where Headed shear stud reinforcement is present: For critical section 0.5d from column face, $v_c$ equals the lesser of: $3\sqrt{f'_c}$(Size effect)(Lightweight concrete factor); $(2 + 4/\beta)\sqrt{f'_c}$(Size effect)(Lightweight concrete factor); $(2 + \alpha_s d/b_o) \sqrt{f'_c}$(Size effect)(Lightweight concrete factor);

For critical section beyond shear reinforcement: $v_c = 2\sqrt{f'_c}$(Size effect)(Lightweight concrete factor)
22.6.6.3 For two-way members with shear reinforcement, effective depth shall be selected such that $v_u$ calculated at critical sections does not exceed $(0.75)(6\sqrt{f'_c})$ where there is stirrup shear reinforcement or $(0.75)(8\sqrt{f'_c})$ where there is headed stud shear reinforcement.

(Recall Chapter 8: $v_u$ includes increase due to moment transfer from eccentric shear.)
22.6.7 Two-way shear strength provided by single or multiple leg stirrups

For $d \geq 6$ inches and 16 stirrup bar diameters, $\nu_s = A_v f_{yt} / b_o s$, where $A_v$ is the sum of the areas of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the column section, and $s$ is the spacing of the peripheral lines of shear reinforcement in the direction perpendicular to the column face.
22.6.8 Two-way shear strength provided by headed shear stud reinforcement

\[ v_s = \frac{A_v f_{yt}}{b_o s} \], where \( A_v \) is the sum of the areas of all shear studs on one peripheral line that is geometrically similar to the perimeter of the column section, and \( s \) is the spacing of the peripheral lines of headed shear stud reinforcement in the direction perpendicular to the column face. It is also required that \( \frac{A_v}{s} \geq 2b_o \frac{\sqrt{f'_c}}{f_{yt}} \).
Example

Given

8'0" C COL  24'0" 8'0"

Plan

#5@10" T&B
inner layers

#5@10" T&B
outer layers

18"x18" COLUMN, PINNED BASE 14' DOWN

Reinforced concrete slab \( h = 12" \), \( d = 10" \)

Normal weight, \( f'_c = 5.0 \text{ ksi} \), \( f_y = 60 \text{ ksi} \)

#5@10" Top & Bottom each way

Live load = 50 psf (No pattern loading)

Columns 18" x 18"

Column moments due to gravity loads are negligible

Seismic Design Category A

N-S Wind load (LRFD level) = 0.8 K/ft @ slab edge

Effective width of slab = 4' each side of column

(8' total, same N-S & E-W)

Required

Check slab compliance with ACI 318-19

for load cases 1.2D + 1.6L and

1.2D + 1.0L + 1.0W
**SOLUTION**

**LOAD CASE 1.2D + 1.6L**

\[ W_u = 1.2(150 \text{ psf}) + 1.6(50 \text{ psf}) = 260 \text{ psf} \]

* \( 4 \left( 4 + \frac{12}{2} \right) = 26 \text{ klf} \)

Long direction load on 8' wide beam

\[ M_{u} = rac{1}{2} (26)(8^2) = 83.2 \text{ kips-ft} \]

\[ M_{u}^{+} = \frac{1}{8} (26)(24^2) - 83.2 \]

\[ = 187.2 - 83.2 \]

\[ = 104 \text{ ft-k} \]

Check 8' wide slab w/ \( 10^\#5, d = 10'' \)

\[ \phi = \frac{3.1}{96(10)} = 0.00341 \quad A_s = 0.31 = 0.0018A_g \]

\[ A_{v} = 0 < A_{v} \text{ min} \quad V_c = \phi (A_s = 1.0)(A = 1.0)(0.00341) \times 15000 \text{ lb/10} \]

\[ d \leq 10'' \quad \text{Normal weight} \quad (N_u = 0) \]

\[ = 81.9 \text{k} \]

* \( \phi = 0.75 = 61.4 \text{k} \)

\[ V_{u} = 2.6 \text{ klf} (12') = 31.2 \text{k} < 61.4 \text{k} \]

(No Need to reduce \( V_u \) to \( d \) from face support)

**Flexure**

\[ C = T = 3.11 \text{ in}^2 (60 \text{ ksi}) = 186 \] kips

\[ \alpha = 186 \times \frac{6}{(6)\times 96} = 0.46'' \]

\[ c = \frac{1}{6} \times 0.46 = 0.07'' \]

\[ 
\epsilon = \frac{0.003 (10 - 0.57)}{0.57} = 0.05 > 0.03 
\]

\[ \phi M_u = 0.9 (186 kips)(10 - \frac{1.25}{2}) = 163.5 \text{ kips-ft} \quad > 136 \text{ ft-k} \]

\[ > 104 \text{ ft-k} \quad > 63.2 \text{ ft-k} \]

(No need to reduce \( M_u \) to face of support) OK
**Solution 1.2D + 1.6L (Cont)**

Two way shear: \( V_u = \left[ \frac{\text{20}(40)}{4} \right] - (1.25)^2 \) (260 psf)

\[ = 51.6 \text{k} \]

\( d = 10'' \quad b_o = 4(18 + 10) = 112'' \) - No moment transfer

\[ V_u = \frac{51.6 \text{k}}{10(112)} \approx 0.46 \text{kpsi} \]

\[ V_\mu = 2\phi \quad \text{No shear reinforcement} \]

\[ \phi \cdot 2\phi = 0.75(4)\sqrt{5000} = 312 \text{ps}i = 0.212 \text{kpsi} < 0.46 \text{kpsi OK} \]

1.2D + 1.0L + 1.0W - Short direction span

\[ W_u = 1.2(150) + 1.0(50) = 230 \text{psf} \]

\[ \frac{1}{2} (4.6)(14)^2 = 36.8 \text{ft-k} \]

\[ \frac{1}{8} (4.6)(12)^2 = 82.8 \text{ft-k} \]

\[ M_u = 82.8 - 36.8 = 46.0 \text{ft-k} < 136 \text{ft-k} \text{ OK} \]

Lateral loads - Check leeward column

\[ 0.8 \times 14 = 16 \text{k} \]

\[ 14' \]

\[ 8 \text{k} \]

\[ 12' \]

\[ 16(14) \]

\[ 18.7 \text{k} \]
One way shear:

\[ V_{u_t} = 4.6 \text{ kips}(6') + 18.7 = 46.3 \text{ kips} < 61.4 \text{ kips} \text{ OK} \]

**Flexure:**

\[ M_{u_t} = \frac{1}{2} (4.6)(4^2) = 36.8 \text{ kips}\cdot\text{in} \]

36.8 + 112 = \frac{149}{112} \text{ kips}\cdot\text{in} < 136 \text{ kips}\cdot\text{in} \text{ OK}

Check windward side column. \( M_{u_t} = \frac{112 - 21.6}{2.7} = 21.6 \text{ kips}\cdot\text{in} \)

No need to reduce to face of column.

Two way shear - leeward column

\[ V_u = \left( \frac{20(40)}{4} - 1.35^2 \right) 230 \text{ psf} + 18.7 = 64.3 \text{ kips} \]

\[ M_u = \frac{149}{112} \text{ in} \]

Fraction of \( M_u = 149 \text{ in} \) transferred by flexure:

\[ V_f = 1 + \frac{2}{3} \sqrt{18 + 1.6} = 1.667 \text{ kips} \]

Effective slab width: \( 18'' + 3 (h = 12'') = 54'' \)

\[ \phi M_n = 136 \text{ in} \left( \frac{54}{96} \right) = 76.5 \text{ kips} \leq 89.4 \text{ kips} \text{ N.G.} \]

\[ 76.5 > 61(114) = 68.4 \text{ kips} \text{ OK} \]
Two way shear (cont)

Moment transferred by eccentricity of shear

\[ \gamma v = 1 - \gamma f = 1 - 0.6 = 0.4 \]

\[ \times 149^k = 59.6 \text{ ft-k} \]

\[ J = \frac{d(c_1+d)^3}{6} + \frac{(c_1+d)d^3}{6} + \frac{d(c_2+d)(c_1+d)^2}{2} \]

\[ \geq \frac{10(18+10)^3}{6} + \frac{(18+10)(10)^3}{6} + \frac{10(18+10)(18+10)^2}{2} \]

\[ = 36,587 + 4667 + 109,760 \]

\[ = 151,000 \text{ in}^4 \]

\[ C = \frac{18+10}{2} = 14 \text{ in} \]

\[ \nu_{uv} = \frac{64.3}{10(113)} = 0.0574 \text{ ksi} = 57.4 \text{ psi} \]

\[ \nu_n = 57.4 \text{ psi} + \frac{59.6 \text{ ft-k} (12/4)(14/12)}{151,000 \text{ in}^4} \]

\[ = 57.4 + 66.3 \]

\[ = 123.7 \text{ psi} \quad (\text{inside face}) \]

\[ \phi \nu_n = 0.75(4)(\sqrt{5000}) = 212 \text{ psi} \quad \text{OK} \]

Conclusion: Slab design shown complies with ACI 318-19
22.7 - TORSIONAL STRENGTH

22.7.1 General

22.7.1.1 This section shall apply to members if the torsion due to factored loads \( T_u \geq 0.75 \) (Threshold torsion of 22.7.4).

22.7.1.2 Nominal torsion strength

22.7.6

22.7.1.3 Lightweight concrete requires reduction according to 19.2.4

22.7.2 Limiting material strengths
22.7.2.1 The value of \( \sqrt{f'_c} \) used to calculate threshold torsion \( T_{th} \) and cracking torsion \( T_{cr} \) shall not exceed 100psi.

22.7.2.2 Steel yield strengths for longitudinal and transverse steel shall not exceed 60 ksi, as shown in Table 20.2.2.4.

22.7.3 Factored design torsion

22.7.3.1 If \( T_u \geq 0.75T_{cr} \) and \( T_u \) is required to maintain equilibrium, the member shall be designed to resist \( T_u \).
22.7.3.2 In a statically indeterminate structure where $T_u \geq 0.75T_{cr}$ and a reduction of $T_u$ can occur due to redistribution of internal forces after torsional cracking, it shall be permitted to reduce $T_u$ to $0.75T_{cr}$, where the cracking torsion is calculated with 22.7.5.

22.7.3.3 If $T_u$ is redistributed as above, design of adjoining members must use the reduced torsion.
22.7.4.1 Threshold torsion $T_{th}$ shall be calculated in accordance with Table 22.7.4.1(a) for solid cross sections and Table 22.7.4.1(b) for hollow cross sections, where $N_u$ is positive for compression and negative for tension.

Table 22.7.4.1(a)
Nonprestressed normal weight member, conservatively neglecting axial compression if any:

\[ T_{th} = \sqrt{f'_c}(A_{cp}^2 / p_{cp}) \]

where \( A_{cp} \) and \( p_{cp} \) are the area and perimeter of the effective concrete torsion beam. (Recall 9.2.4.4: For T or L beams, up to 4 slab thicknesses each side of beam web can be included in the effective torsion beam. ??? Included portions of slabs must have their longitudinal steel enclosed by stirrups???) (??May be acceptable to neglect slabs.??)
Prestressed member:
Nonprestressed member subjected to axial force
Table 22.7.4.1(b)
22.7.5 CRACKING TORSION

Nonprestressed normalweight member, conservatively neglecting axial compression if any:

\[ T_{cr} = 4\sqrt{f'_c}(A_{cp}^2 / p_{cp}) \]

Prestressed member

Nonprestressed member subjected to axial force
**Equilibrium Torsion**

\[ T_u = P_u e \]

**Compatibility Torsion**

**Plan**

\[ T_{th} = T_{cr}/4 \]

\[ T_{cr} = 4 \sqrt{f'c'p} \left( \frac{A_{cp}^2}{P_{cp}} \right) \]

\[ P_{cp} = 2(b+h) \]

\[ A_{cp} = bh \]

**Moment Diagram**

\[ \text{Permitted to neglect torsion where:} \]

\[ \text{Factorial Torsion} \leq (\phi = 0.75) \cdot T_{th} \]
22.7.6 TORSIONAL STRENGTH

22.7.6.1 For nonprestressed and prestressed members, nominal torsional strength $T_n$ shall be the lesser of (a) and (b):

(a) $T_n = 2 A_o f_{yt} \cot \theta \left( \frac{A_t}{s} \right)$

(b) $T_n = 2 A_o f_y \tan \theta \left( \frac{A_l}{p_h} \right)$
Where:

\( A_o \) = gross area enclosed by the torsional shear flow path, permitted to be taken as 0.85 times the area enclosed by the centerline of the outermost closed transverse torsional reinforcement, in\(^2\)

\( \Theta = 45 \) degrees for nonprestressed members

\( A_t \) = area of one leg of the closed stirrup resisting torsion

\( A_l \) = area of longitudinal torsional reinforcement

\( P_h \) = perimeter of the centerline of the outermost closed stirrup.
22.7.7 Cross-sectional limits for solid and hollow sections
**Torsional Strength**

Longitudinal Reinforcement

Transverse Reinforcement

\[ d \leq 12'' \]

\[ < \frac{Ph}{B} \]

\( (9.7.5, 9.7.6) \)

Closed stirrup, perimeter

\( Ph \), enclosed area \( A_{oh} \)

Transverse reinforcement not required for shear of this load case

\( At = \) Area one leg

\[ A_L = \text{Longitudinal reinforcement not required for flexure} \]

Total area "distributed around perimeter" (developed each side of section)

\[ T_n = \text{minimum} \left[ \frac{2 A_0 At f_y}{s} \cot \theta + \right] \]

\[ \frac{2 A_0 A_L f_y}{Ph \tan \theta} \]

Generally, \( \theta = 45^\circ \), \( A_0 = 0.85 A_{oh} \)
22.8 – Bearing $B_n = 0.85f'_c A_1$, but up to twice this if supporting surface is wider on all sides than the loaded area.
22.9 – SHEAR FRICITION

22.9.1 General
22.9.1.1 This section shall apply where it is appropriate to consider shear transfer across any given plane, such as an existing or potential crack, an interface between dissimilar materials, or an interfaced between two concretes cast at different times.
22.9.4.2 If shear-friction reinforcement is perpendicular to the shear plane, nominal shear strength across the assumed shear plane shall be calculated by:

\[ V_u = \mu A_{vf} f_y \]

where \( A_{vf} \) is the area of reinforcement crossing the assumed shear plane to resist shear, and \( \mu \) is the coefficient of friction in accordance with Table 22.9.4.2.
Table 22.9.4.2 Coefficients of friction (reduce according to 19.2.4 for lightweight concrete, but $\lambda_{\text{max}} = 0.85$.

Concrete placed monolithically: $\mu = 1.4$

Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 0.25 inch $\mu = 1.0$
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened \( \mu = 0.6 \)

Concrete placed against as-rolled structural steel that is clean, free of paint, and shear transferred across the contact surface with headed studs or by welded bars or wires \( \mu = 0.7 \)
22.9.4.6 Area of reinforcement required to resist a net factored tension across an assumed shear plane shall be added to the area of reinforcement required for shear friction...
Bearing Strength

\[ A_1 = 12 \times 12 = 144 \text{ in}^2 \]

\[ A_2 = 18 \times 18 = 324 \text{ in}^2 \]

\[ \frac{A_2}{A_1} = \frac{324}{144} = 2.25 \]

\[ B_u \leq \phi B_n \leq 0.65 B_n \]

\[ B_n = 0.85 f'_c A_n \sqrt{\frac{A_2}{A_1}} \]
Shear Friction

Post-installed shear friction reinforcement

Post-installed tension reinforcement

$V_u \leq (\phi \cdot V_n = 0.75 \cdot V_n)$

$V_n = \mu A_{vf} f_y$

$\mu = 1.4 \lambda$

Concrete placed monolithically

$= 0.6 \lambda$

Concrete placed against --- not intentionally roughened
CHAPTER 23 – STRUT – AND – TIE METHOD
23.1 – SCOPE

23.1.1 This chapter shall apply to the design of structural concrete members, or regions of members, where load or geometric discontinuities cause a nonlinear distribution of longitudinal strains within the cross section.

23.1.2 Any structural concrete member, or discontinuity region in a member, shall be permitted to be designed by modeling the member or region as an idealized truss in accordance with this chapter.
23.2 – General

23.2.1 Strut-and-tie models shall consist of struts and ties connected at nodes to form an idealized truss in two or three dimensions.

23.2.5 Ties shall be permitted to cross struts and other ties.

23.2.6 Struts shall intersect or overlap only at nodes.
23.2.7 The angle between the axes of any strut and any tie entering a single node shall be at least 25 degrees.

23.4 – Strength of struts

23.5 – Minimum distributed reinforcement

23.6 – Strut reinforcement detailing
23.7 - Strength of ties
23.8 – Tie reinforcement detailing
23.9 – Strength of nodal zones
23.10 – Curved-bar nodes
23.11 – Earthquake-resistant design
CHAPTER 24 - SERVICEABILITY
24.1.1 This chapter shall apply to member design for minimum serviceability, including (a) through (d):

(a) Deflections due to service-level gravity loads (24.2)

(b) Distribution of flexural reinforcement in one-way slabs and beams to control cracking (24.3)

(c) Shrinkage and temperature reinforcement (24.4)

(d) Permissible stresses in prestressed flexural members (24.5)
24.2 – Deflections due to service-level gravity loads

24.2.1 Members subjected to flexure shall be designed with adequate stiffness to limit deflections or deformations that adversely affect strength or serviceability of a structure.

24.2.2 Deflections calculated in accordance with 24.2.3 through 24.2.5 shall not exceed the limits in Table 24.2.2.
Table 24.2.2 – Maximum permissible calculated deflections

Flat roofs not supporting or attached to deflection sensitive nonstructural elements: Limit immediate deflection due to live, snow, or rain loads to: Span / 180 (Clear or centerline ?; limit not intended to safeguard against ponding)
Floors not supporting or attached to deflection sensitive nonstructural elements: Limit immediate deflection due to live load to: \( \frac{\text{Span}}{360} \)

Roofs or floors supporting or attached to nonstructural elements likely to be damaged by large deflections: Limit that part of total deflection occurring after attachment of nonstructural element to: \( \frac{\text{Span}}{480} \) (Limit is \( \frac{\text{Span}}{240} \)) if nonstructural element not likely to be damaged by deflection)
24.2 DEFLECTIONS (CONTINUED)

24.2.3 Calculation of immediate deflections

24.2.3.1 Immediate deflections shall be calculated using methods or formulas for elastic deflections, considering effects of cracking and reinforcement on member stiffness.

24.2.3.4 Modulus of elasticity, $E_c$, shall be permitted to be calculated in accordance with 19.2.2

$E_c, \text{ ksi} = 57\sqrt{f'c, \text{ psi}}$ for normalweight concrete
24.2DEFLECTIONS (CONT’D)

24.2.3.5 For nonprestressed members,...effective moment of inertia, $I_e$, shall be calculated in accordance with Table 24.2.3.5 using a value for member cracking moment $M_{cr}$ equal to: $(f_r I_g / y_t)$

Table 24.2.3.5 – Effective moment of inertia, $I_e$

Maximum member moment due to unfactored loads, $M_a$, LE 0.667 $M_{cr}$ : $I_e = I_g$

Otherwise: $I_e = I_{cr} / [1 - (.667 M_{cr} / M_a)^2 (1 - I_{cr} / I_g)]$
(Note: $I_{cr}$ generally calculated with the transformed area method, steel transformed to concrete)

24.2.3.6 For continuous one-way slabs and beams, $I_e$ shall be permitted to be taken as the average of values obtained for the maximum positive and negative moment sections.
24.2 DEFLECTIONS (CONT’D)

24.2.3.7 For prismatic one-way slabs and beams, $I_e$ shall be permitted to be taken as the value at midspan for simple and continuous spans, and at the support for cantilevers.

24.2.3.8 For prestressed Class U slabs and beams as defined in 24.5.2, it shall be permitted to use $I_g$.
24.2.3.9 For prestressed Class T and Class C slabs and beams:

\[ I_e = R^3 I_g + (1 - R^3) I_{cr} \]

where \( R = \frac{M_{cr}}{M_a} \) and \( M_{cr} = \frac{(f_r + f_{pe}) I_g}{y_t} \)
24.2.4.1 Nonprestressed members: For loads with a cumulative duration of five years or more, the additional deflection from creep and shrinkage shall be calculated as the product of the immediate deflection caused by sustained loads and the factor: \( \frac{2}{1 + 50(\text{compression reinforcement ratio})} \)
24.2.4.1.2 Use compression reinforcement ratio at midspan for simple and continuous spans, and at the support for cantilevers.
24.3.1 Bonded reinforcement shall be distributed to control flexural cracking in tension zones of nonprestressed and Class C prestressed slabs and beams reinforced for flexure in one direction only.

24.3.2 Spacing of bonded reinforcement closest to the tension face shall not exceed the limits in Table 24.3.2, where $c_c$ is the least distance from the surface of deformed or prestressed reinforcement to the tension face.
24.3 – Distribution of flexural reinforcement in one-way slabs and beams (CONT’D)

Deformed bars or wires, maximum spacing the lesser of:
15 ( 40000psi / 0.667 f_y ) - 2.5 c_c , or 12(40000/0.667f_y)

24.3.4 If the flange of a T-beam is in tension, the portion of the bonded flexural tension reinforcement not located over the beam web shall be distributed within the lesser of the effective flange width and the 0.1(clear span) ...
24.4.3 Nonprestressed reinforcement: 0.0018bh spaced not more than 5h or 18 inches (includes all perpendicular reinforcement top and bottom of one-way slab)
24.5 – Permissible stresses in prestressed concrete flexural members
24.5.2.1 Prestressed flexural members shall be classified as Class U, T, or C in accordance with Table 24.5.2.1, based on the extreme fiber stress in tension \( f_t \) in the precompressed tension zone calculated at service loads assuming an uncracked section.

- **Class U Uncracked** \( f_t \leq 7.5\sqrt{f'_c} \)
- **Class T Transition** \( f_t \leq 12\sqrt{f'_c} \)
- **Class C Cracked** \( f_t > 12\sqrt{f'_c} \)
Classification of Prestressed Concrete

Class U "Uncracked"

Class T "Transition"

Class C "Cracked"

Maximum tensile stress, psi

$< 7.5 \sqrt{f'_c}$, psi

$7.5 \sqrt{f'_c} < \text{Maximum Tensile Stress} \leq 12 \sqrt{f'_c}$

$\text{Maximum Tensile Stress} > 12 \sqrt{f'_c}$
CHAPTER 25 – REINFORCEMENT DETAILS
25.1- Scope: Minimum spacing; Standard hooks; Development; Splices; Bundled reinforcement;...
25.2.1 For parallel nonprestressed reinforcement in a horizontal layer, clear spacing shall be at least the greatest of 1 in., $d_b$, and $1.33d_{agg}$.

25.2.2 For parallel nonprestressed reinforcement placed in two or more horizontal layers, reinforcement in the upper layers shall be placed directly above reinforcement in the bottom layer with a clear spacing between layers of at least 1 inch.
25.2 MINIMUM SPACING (CONT’D)

25.2.3 For longitudinal reinforcement in columns, pedestals, struts, and boundary elements in walls, clear spacing between bars shall be at least the greatest of 1.5 in., 1.5 \( d_b \), and 1.333\( d_{agg} \).
Minimum Spacing of Reinforcement

Beam Section

clear space > 1\" \(d_b\) or 1.33\(d_{agg}\)

Column Section

clear space > 1.5\" \(1.5d_b\) or 1.33\(d_{agg}\)
25.3 – Standard hooks, seismic hooks, crossties, and minimum inside bend diameters

90-degree hook, #9 #10 #11:
Minimum inside bend diameter $8d_b$;
$12d_b$ straight extension
25.4 Development of Reinforcement

25.4 – Development of reinforcement

25.4.1 General

25.2.1.2 Hooks and heads shall not be used to develop bars in compression.

25.4.1.3 Development lengths do not require a strength reduction factor.

25.4.1.4 The value of $\sqrt{f'_c}$ used to calculate development length shall not exceed 100 psi.
25.4.2 Development of deformed bars and deformed wires in tension

25.4.2.1 Development length $l_d$ for deformed bars and deformed wires in tension shall be the greater of 12 inches and the length calculated in accordance with 25.4.2.3 or 25.4.2.4 using the applicable modification factors of 25.4.2.5.
25.4 DEVELOPMENT OF REINFORCEMENT (CONT’D)

Table 25.4.2.3

#7 or larger with clear spacing at least 2d_b and clear cover at least d_b:

\[ l_d = \left( \frac{f_y}{20 \sqrt{f'_c}} \right) d_b \]

(Can reduce \( l_d \) up to 0.6 with more detailed formula)

 Increases in \( l_d \) for top bars, epoxy coating, \( f_y \) greater than 60 ksi, lightweight concrete
25.4 DEVELOPMENT OF REINFORCEMENT (CONT’D)

25.4.3 Development of standard hooks in tension

\[ l_{dh} = \left( \frac{f_y}{55\sqrt{f'_c}} \right) d_b^{1.5} \]

plus modification factors

25.4.4 Development of headed deformed bars in tension
25.4.4.1 Use of a head to develop a deformed bar in tension shall be permitted if conditions (a) through (f) are satisfied:

(a) Bar shall conform to 20.2.1.6 (ASTM A970)
(b) Bar size shall not exceed No. 11
(c) Net bearing area of head $A_{brg}$ shall be at least $4A_b$
(d) Concrete shall be normalweight
(e) Clear cover for bar shall be at least $2d_b$
(f) Center-to-center spacing between bars shall be at least $3d_b$
25.4.4.2 Development length $l_{dt}$ for headed deformed bars in tension shall be the longest of (a) through (c):

(a) $l_{dt} = \left( \frac{f_y}{75\sqrt{f'_c}} \right) d_b^{1.5}$

(b) $8d_b$

(c) 6 inches; plus modification factors and prescriptive requirements for reinforcement details
Tension Development Lengths

Concrete

\[ l_d = d_b \left( \frac{f_y}{f_{ci}^{0.12}} \right) \]

Epoxy 1.2 to 1.5
Size \#6, #8, #10, and smaller

1.5 GRC
1.3 GRC

Ktr \( \geq \) can use 0

Epoxy 1.2

\[ l_{dh} = \frac{f_y \psi_e \psi_r \psi_o \psi_c}{5.5 \lambda \sqrt{f_c^*}} d_b \]

7.5

\[ \psi_r = 1.0 \text{ for confined #11 and smaller} \]

1.6 \text{ other}

\[ \psi_o = 1.0 \text{ for #11 and smaller with cover} \]

1.25 \text{ other}

\[ \psi_c = \frac{f_c'}{f_c} + 0.6, f_c' \leq 6 \text{ ksi} \]

1.0 \text{ ksi} \leq f_c' \leq 6 \text{ ksi}

\[ A_{brg} > 4A_b \]

Headed deformed bar

\[ l_{dt} = \frac{f_y \psi_e \psi_r \psi_o \psi_c}{7.5 \lambda \sqrt{f_c^*}} d_b \]

\[ > E_d b \geq 6 \text{ inch} \]

Concrete - Normal weight

Date

DESIGNER

DATE

CHECKED

DATE

PROJECT

LANIER & ASSOCIATES

CONSULTING ENGINEERS, INC.

NEW ORLEANS, LA & BEAUMONT, TX
CODE

Fig. R25.4.3.3a—Confining reinforcement placed parallel to the bar being developed that contributes to anchorage strength of both 90- and 180-degree hooked bars.

Fig. R25.4.3.3b—Confining reinforcement placed perpendicular to the bar being developed, spaced along the development length \( l_{db} \), that contributes to anchorage strength of both 90- and 180-degree hooked bars.

25.4.3.4 For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover to hook less than 2-1/2 in., (a) and (b) shall be satisfied:

(a) The hook shall be enclosed along \( l_{db} \) within ties or stirrups perpendicular to \( l_{db} \) at \( s \leq 3d_0 \)

(b) The first tie or stirrup shall enclose the bent portion of the hook within \( 2d_0 \) of the outside of the bend, where \( d_0 \) is the nominal diameter of the hooked bar.

R25.4.3.4 Hooked bars are especially susceptible to a concrete splitting failure if both side cover (perpendicular to plane of hook) and top or bottom cover (in plane of hook) are small (refer to Fig. R25.4.3.4). Transverse reinforcement is required to provide additional splitting resistance. This provision applies at ends of simply-supported beams, at the first end of cantilevers, and at exterior joints for members framing into a joint where members do not extend beyond the joint. This provision does not apply for hooked bars at discontinuous ends of slabs where confinement is provided by the slab on both sides, perpendicular to the plane of the hook.
25.4.4.3 For the calculation of $L_m$, modification factors $\psi_{\text{epoxy}}$, $\psi_{\text{uncoated}}$, and $\psi_{\text{other}}$ shall be in accordance with Table 25.4.4.3.

Table 25.4.4.3—Modification factors for development of headed bars in tension

<table>
<thead>
<tr>
<th>Modification factor</th>
<th>Condition</th>
<th>Value of factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epoxy $\psi_{\text{epoxy}}$</td>
<td>Epoxy-coated or zinc and epoxy dual-coated reinforcement</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Uncoated or zinc-coated (galvanized) reinforcement</td>
<td>1.0</td>
</tr>
<tr>
<td>Parallel to reinforcement $\psi_p$</td>
<td>For No. 11 and smaller bars with $d / d_{ch} \leq 0.35$, or $d / d_{ch} &gt; 0.35$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Other</td>
<td>1.5</td>
</tr>
<tr>
<td>Location $\psi_z$</td>
<td>For headed bars: (1) Transmitting inside-column cores with side cover to bar $&gt; 2.5$ in, or (2) With side cover to bar $&gt; 6 d_{ch}$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Other</td>
<td>1.25</td>
</tr>
<tr>
<td>Concrete strength $\psi_c$</td>
<td>$f_{cm} &lt; 6000$ psi</td>
<td>(2 \times 15,000 + 0.6f_{cm} )</td>
</tr>
<tr>
<td></td>
<td>$f_{cm} \geq 6000$ psi</td>
<td>1.0</td>
</tr>
</tbody>
</table>

$\psi_{\text{epoxy}}$ is measured center-to-center spacing of headed bars.

$d_{ch}$ is nominal diameter of headed bar.

Refers to 25.4.4.5.

Fig. R25.4.4.3.1—Breakout failure precluded in joint by providing transverse reinforcement to enable a strut-and-tie mechanism.

The epoxy factor 1.2 is based conservatively on the value used for epoxy-coated standard hooks. The location factor $\psi_z$ accounts for the confinement provided by the reinforcement within columns and large side cover for other members.

The factor $\psi_p$ for headed reinforcement is similar to the confining reinforcement factor for hooked bars (Shao et al. 2016). Unlike hooked bars, however, test results indicate that only tie or hoop reinforcement parallel to headed bars contributes to anchorage strength and reduces development length (Thompson et al. 2005, 2006a,b).
Fig. R25.4.4.4—Ties in stirrups placed parallel to the headed beam bars being developed in a beam-column joint that contribute to anchorage strength.
25.4 DEVELOPMENT OF REINFORCEMENT (CONT’D)

25.4.5 Development of mechanically anchored deformed bars in tension
25.4.6 Development of welded deformed wire reinforcement in tension
25.4.7 Development of welded plain wire reinforcement in tension
25.4.8 Development of pretensioned seven-wire strands in tension
25.4.9 Development of deformed bars and deformed wires in compression: (GE 8 inch)
25.4 DEVELOPMENT OF REINFORCEMENT (CONT’D)

\[ l_{dc} = \left( \frac{f_y}{50\sqrt{f'_c}} \right) d_b, \text{ divide by } 0.75 \text{ for lightweight concrete but not less than } 0.0003 f_y d_b; \]
(\text{can multiply } l_{dc} \text{ by } 0.75 \text{ for special confinement details})

25.4.10 Reduction of development length for excess reinforcement
25.4.10.1 Reduction of development lengths.... Shall be permitted by use of the ratio of required area of reinforcement to provided area of reinforcement except where prohibited by 25.4.10.2. ....not less than the minimums specified.
25.4.10.2 Reductions not permitted: Noncontinuous supports; Development of $f_y$ required; Bars required to be continuous; Hooked, headed, and mechanically anchored deformed reinforcement; Seismic-force-resisting systems in Seismic Design Categories C, D, E, or F; Anchorage of concrete piles and concrete filled pipe piles to pile caps in structures assigned to Seismic Design Categories C, D, E, or F
25.5 Splices

25.5.1 General

25.5.5.1 Lap splices shall not be permitted for bars larger than No. 11.

25.5.1.3 For noncontact splices in flexural members, the transverse center-to-center spacing of spliced bars shall not exceed the lesser of one-fifth the required lap splice length and 6 inches.
25.5 Splices
(CONT’D)

25.5.2 Lap splice lengths of deformed bars and deformed wires in tension

25.5.2.1 Tension lap splice length $l_{st}$ for deformed bars and wires in tension...Table 25.5.2.1
25.5 Splices (CONT’D)

Area of flexural reinforcement at least twice area required over the length of the splice and not more than 50 percent of reinforcement being spliced:
Class A splice, $l_{st}$ the greater of $l_d$ and 12 inch
All other cases: Class B, $l_{st}$ the greater of $1.3l_d$ and 12 inch
25.5.2.2 If bars of different size are lap spliced in tension, $l_{st}$ shall be the greater of $l_d$ of the larger bar and $l_{st}$ of the smaller bar.
25.5.5 Lap splice lengths of deformed bars in compression

25.5.5.1 Compression lap splice length $l_{sc}$ of No. 11 or smaller deformed bars in compression shall be calculated in accordance with (a), (b), or (c):

(a) For $f_y \leq 60$ ksi, $l_{sc}$ the longer of $0.0005 f_y d_b$ and 12 inches
25.5 Splices (CONT’D)

25.5.6 End-bearing splices of deformed bars in compression: ....square cut ends held in concentric contact...

25.5.7 Mechanical and welded splices of deformed bars in tension or compression

Develop 1.25 \( f_y \).
25.6.1 Nonprestressed reinforcement

25.6.1.1 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four in any one bundle.

25.6.1.2 Bundled bars shall be enclosed within transverse reinforcement. Bundled bars in compression members shall be enclosed by transverse reinforcement at least No. 4 in size.
25.6.1.4 Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least $40d_b$ stagger.

25.6.1.5 Development length for individual bars within a bundle, in tension or compression, shall be that of the individual bar, increased 20 percent for a three-bar bundle, and 33 percent for a four-bar bundle.
25.6.1.6 A unit of bundled bars shall be treated as a single bar with an area equivalent to that of the bundle and a centroid coinciding with that of the bundle. The diameter of the equivalent bar shall be used for $d_b$ in (a) through (e): Spacing; Cover; Confinement; other
25.7.1 Stirrups
25.7.1.1 Stirrups shall extend as close to the compression and tension surfaces of the member as cover requirements and proximity to other reinforcement permits and shall be anchored at both ends. Where used as shear reinforcement, stirrups shall extend a distance d from extreme compression fiber.
25.7 – TRANSVERSE REINFORCEMENT (CONT’D)

25.7.1.2 Between anchored ends, each bend in the continuous portion of a single or multiple U-stirrup and each bend in a closed stirrup shall enclose a longitudinal bar or strand.

25.7.1.6 Stirrups used for torsion or integrity reinforcement shall be closed stirrups perpendicular to the axis of the member...
25.7.1.6.1 Stirrups used for torsion or integrity reinforcement shall be permitted to be made up of two pieces of reinforcement: a single U-stirrup... and a crosstie.....
25.7.2 TIES

25.7.2.1 Ties shall consist of a closed loop of deformed bar with spacing in accordance with (a) and (b):

(a) Clear spacing of at least $1.33 d_{agg}$

(b) Center-to-center spacing shall not exceed the least of $16d_b$ of longitudinal bar, $48d_b$ of tie bar, and smallest dimension of member
25.7.2.2 Diameter of tie bar shall be at least No. 3 for No. 10 or smaller longitudinal bar; No. 4 otherwise

25.7.2.3 Rectilinear ties shall be arranged so that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees and no unsupported bar shall be farther than 6 inches clear on each side along the tie from a laterally supported bar
25.7.2 TIES (CONT’D)

25.7.2.4 Circular ties shall be permitted where longitudinal bars are located around the perimeter of a circle.
25.7.3 SPIRALS

25.7.3.1 Spirals shall consist of evenly spaced continuous bar or wire with clear spacing at least 1 inch, at least 1.33 $d_{\text{agg}}$, but not greater than 3 inch; at least 0.375 inch diameter for cast-in-place construction.
25.7.3 SPIRALS (CONT’D)

25.7.3.3 Except for transverse reinforcement in deep foundations, the volumetric spiral reinforcement ratio shall be \( \geq 0.45 \left( \frac{A_g}{A_{ch}} -1 \right) \frac{f'_c}{f_{yt}} \); \( A_{ch} \) is the area to the outside area of the spiral.

25.7.3.4 Spirals shall be anchored by 1.5 extra turns of spiral bar or wire at each end.
Fig. R25.7.2.3e—Illustrations to clarify measurements between laterally supported column bars and rectilinear tie anchorage.
CHAPTER 26
CONSTRUCTION DOCUMENTS AND INSPECTIONS
26.1 SCOPE

26.1.1 This chapter addresses (a) through (c):

(a) Design information that the licensed design professional shall specify in the construction documents, if applicable.
26.1 SCOPE (CONT’D)

(b) Compliance requirements that the licensed design professional shall specify in the construction documents, if applicable.

(c) Inspection requirements that the licensed design professional shall specify in the construction documents, if applicable.
26.2 - Design criteria: Names and years of governing codes; Design loads; Delegated portions;

26.3 – Member information: Member sizes, locations, tolerances

26.4 – Concrete materials and mixture requirements

26.5 – Concrete production and construction
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26.11 – Formwork
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26.13 – Inspection
CHAPTER 27
STRENGTH EVALUATION OF EXISTING STRUCTURES
27.1 – Scope: Provisions of this chapter shall apply to strength evaluation of existing structures by analytical means or by load testing.
APPENDIX A
DESIGN VERIFICATION USING NONLINEQQR RESPONSE HISTORY ANALYSIS