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.

IN-LB Inch-Pound Units

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

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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 GE 4 inches and 1/25 the lessor of unsupported length and unsupported height

Nonbearing wall: h GE 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

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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.

Area of transverse (ie horizontal) reinforcement per vertical inch / h

$$V_n = (\alpha_c \lambda \sqrt{f'_c}_{\,psi} + \rho_t f_{yt}) A_{cv}$$

$\alpha_c = 3$ for $h_w/\ell_w \le 1.5$

$= 2$ for > 2.0

$A_{cv} = A_g = h\ell_w$

h_w is entire height if multistory

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

11.4.2 *Factored axial force and moment*

11.4.2.1 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 $\phi P_{n,max}$, where $P_{n,max}$ shall be as given in 22.4.2.1 and strength reduction factor ϕ shall be that for compression-controlled sections in 21.2.2. 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 *Factored shear*

11.4.3.1 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*

11.5.1.1 For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \ge U$, including (a) through (c). Interaction between axial load and moment shall be considered.

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R11.4.1.3 The forces typically acting on a wall are illustrated in Fig. R11.4.1.3.

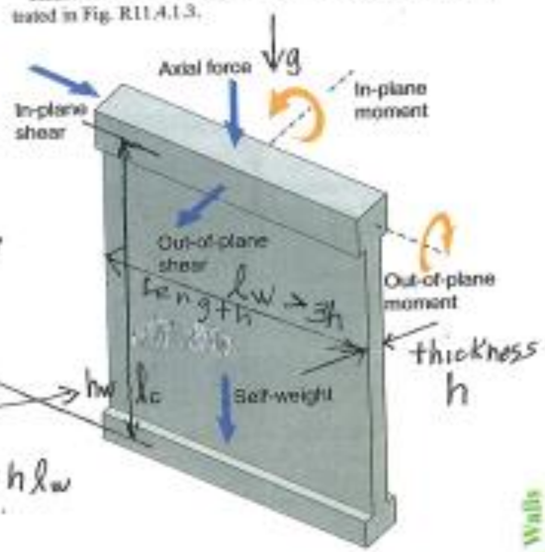


Fig. R11.4.1.3—In-plane and out-of-plane forces.

R11.5—Design strength

11

Walls

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 – DESIGN STRENGTH (CONT'D)

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: ($e = M/P$)

$P_n = 0.55\, f'_c\, A_g\, [\, 1 - (kl_c / 32h)^2\,]$,

$k = 0.8$ one end fixed other pinned,

$= 1.0$ both ends pinned,

$= 2.0$ cantilever

11.5.4 - IN-PLANE SHEAR

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 LE 8sqrt(f'_c)(gross wall area in a horizontal section)

11.5.4 - IN-PLANE SHEAR (CONT'D)

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 = (Gross wall area in a horizontal section) (3sqrt(f'_c) + 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

11.6 – REINFORCEMENT LIMITS

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 – REINFORCEMENT DETAILING (CONT'D)

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 -SPACING OF TRANSVERSE REINFORCEMENT

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 – SPACING OF REINFORCEMENT (CONT'D)

11.7.4 – LATERAL SUPPORT

11.7.5 OPENINGS

11.8 – ALTERNATE ANALYSIS

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
12.2 GENERAL

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 -DESIGN STRENGTH

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 - DESIGN STRENGTH (CONT'D)

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 = (slab thickness)(slab plan dimension in the direction of the load – openings = "depth")(2sqrt(f'_c) + (area of steel reinforcing parallel to load per inch of slab width perpendicular to load / slab thickness) (steel yield stress))

f'_c LE 100psi ; V_n limited to (0.75)(8)sqrt(f'_c)(slab thickness) (slab depth)

12.5 -DESIGN STRENGTH (CONT’D)

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.

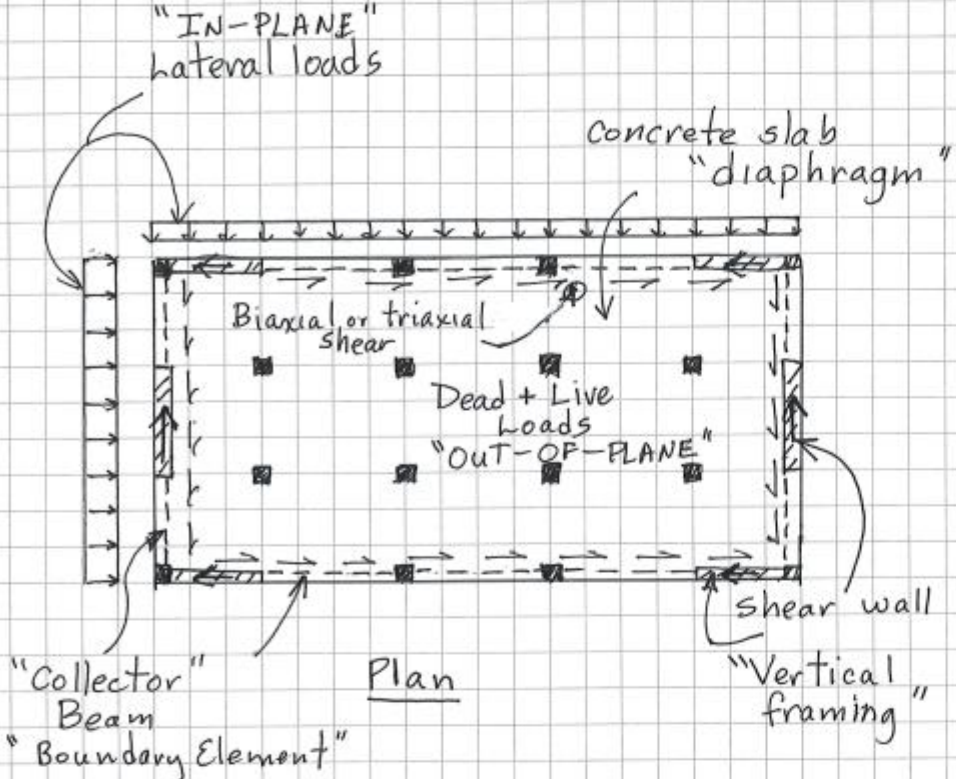
12.6 – REINFORCEMENT LIMITS

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



Check
$1.2D + 1.6L$ – Diaphragm forces not required
$1.2D + 1.0L + 1.0(W \text{ or } E)$

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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 - DESIGN CRITERIA

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 - DESIGN CRITERIA (CONT’D)

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

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 – CRITICAL SECTIONS (CONT'D)

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.

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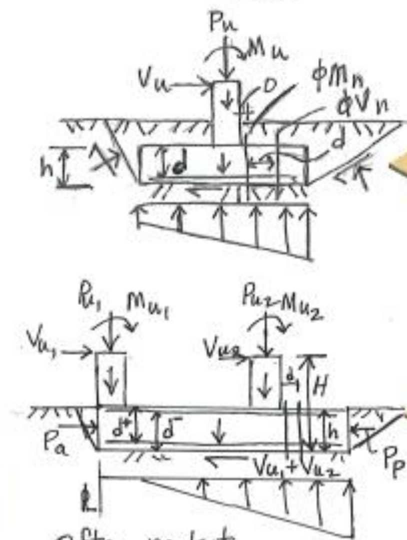


Often neglect
$P_p - P_a$

Combined Footing

Check $1.2D + 1.6L$
$1.2D + 1.0L + 1.0(W \text{ or } E)$
$0.9D + 1.0(W \text{ or } E)$

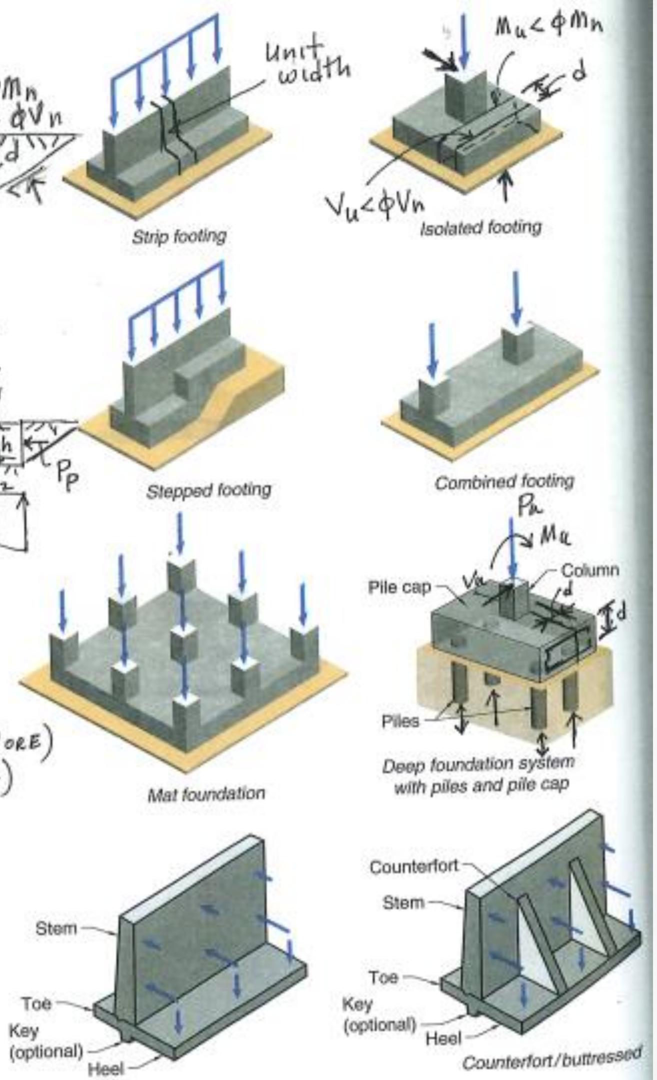


Fig. R13.1.1—Types of foundations.

13.1.2 Foundations excluded by 1.4.7 are excluded from this chapter.

13.2.8 – 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 – SHALLOW FOUNDATIONS (CONT'D)

13.3.3.3 In rectangular footings:

(a) Reinforcement in the long direction distributed uniformly across width.

(b) In short direction, fraction 2/(1+ 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 – SHALLOW FOUNDATIONS (CONT'D)

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 – SHALLOW FOUNDATIONS (CONT’D)

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 – DEEP FOUNDATIONS (CONT'D)

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, section2.4, 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 – DEEP FOUNDATIONS (CONT'D)

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 – DEEP FOUNDATIONS (CONT'D)

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 – DEEP FOUNDATIONS (CONT'D)

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 – DEEP FOUNDATIONS (CONT’D)

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

(b) $0.75 v_n$ GE 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 – DEEP FOUNDATIONS (CONT’D)

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):

13.4 – DEEP FOUNDATIONS (CONT’D)

(a) Entire reaction from any pile with its center located $d_{pile}/2$ or more outside the section shall be considered as producing shear on that section.

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

(c) Linear interpolation

CHAPTER 14 – PLAIN CONCRETE

14.1 SCOPE

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)
14.2 GENERAL

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.(5sqrt(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
15.2 GENERAL

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 – GENERAL (CONT'D)

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 – GENERAL (CONT'D)
15.3 DETAILING OF JOINTS

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 – DETAILING OF JOINTS (CONT'D)

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 s 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

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: $V_n = 12 \text{ sqrt}(f_c')A_j$

15.4 – STRENGTH REQUIREMENTS (CONT'D)

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 lessor of (a) and (b):

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

16.1-16.5

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 – SCOPE

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 – SCOPE

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… (New to ACI 318)

(g) Attachments with shear lugs.. (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

CODE

COMMENTARY

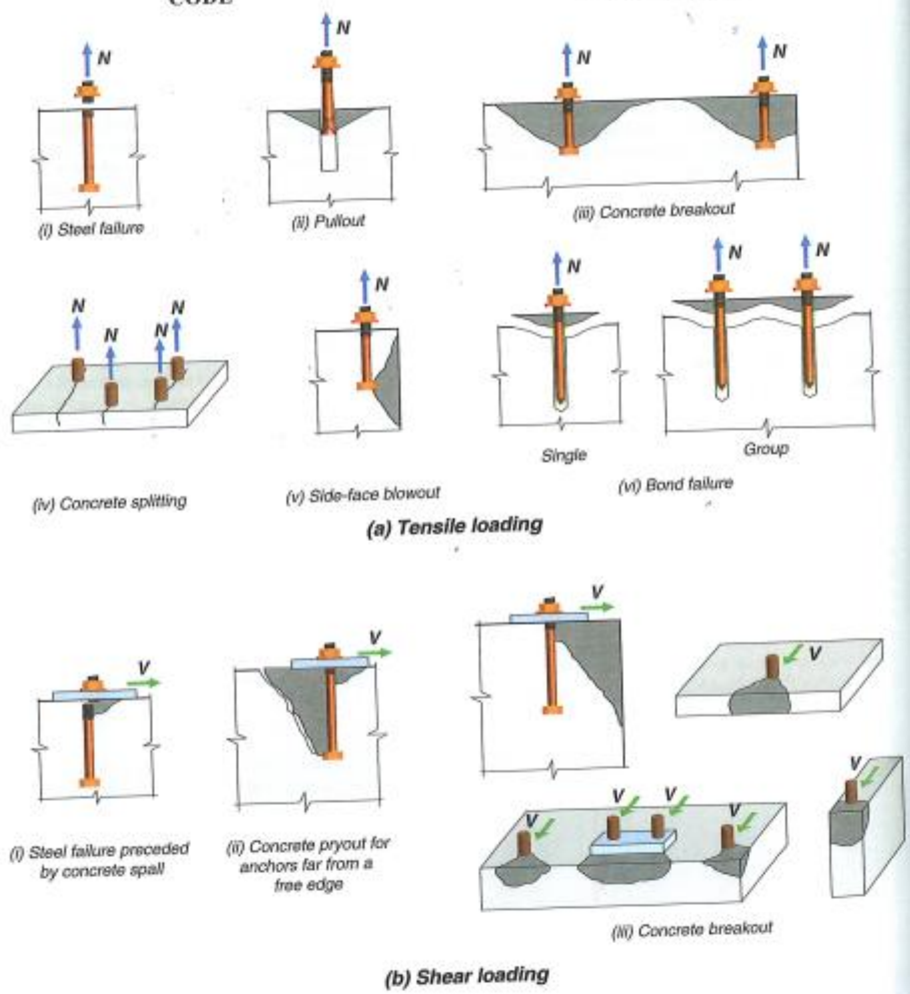


***Fig. R17.5.1.2**—Failure modes for anchors.*

17.5.1.3 Strength of anchors shall be permitted to be determined in accordance with 17.6 for 17.5.1.2(a) through (e), and 17.7 for 17.5.1.2(f) through (h). For adhesive anchors that resist sustained tension, the requirements of 17.5.2.2 shall apply.

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. (1995); Eligehausen and Balogh (1995), 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 – DESIGN STRENGTH (CONT'D)

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 $3h_{ef}$ x $3h_{ef}$ 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 – 17.10

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

CODE

COMMENTARY

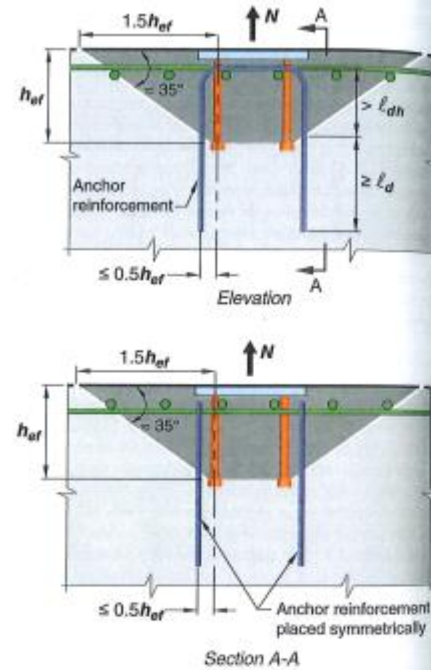


Fig. R17.5.2.1a—Anchor reinforcement for tension.

CODE

COMMENTARY

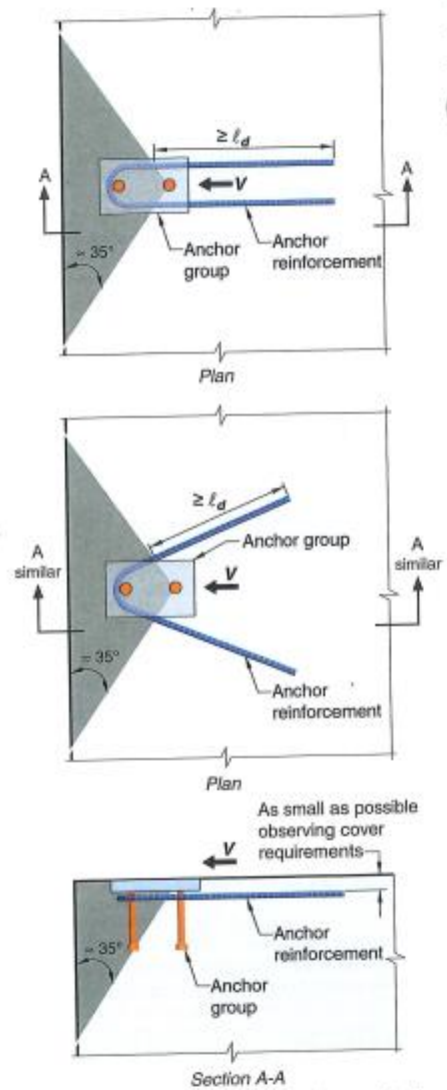


Fig. R17.5.2.1b(i)—Hairpin anchor reinforcement for shear.

CODE

17.5.1.3.1 Anchor group effects shall be considered whenever 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

Failure mode under investigation	Critical spacing
Concrete breakout in tension	$3h_{ef}$
Bond strength in tension	$2c_{Na}$
Concrete breakout in shear	$3c_{a1}$

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 (h).

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 (Eligehausen et al. 2006a).

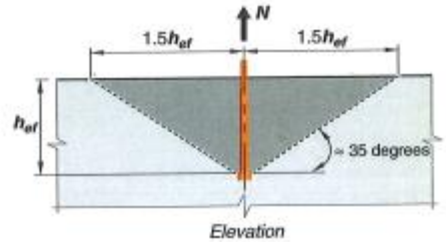


Fig. R17.5.1.3a—Breakout cone for tension.

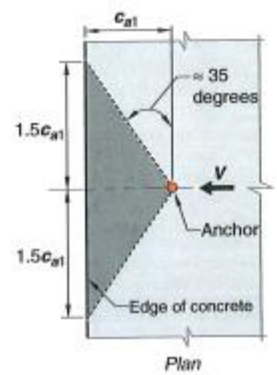


Fig. R17.5.1.3b—Breakout cone for shear.

R17.5.1.4 Sections 17.5.1.2 and 17.5.2.3 establish the performance factors for which anchor design models are required to be verified. Many possible design approaches exist, and the user is always permitted to "design by test" using 17.5.1.4 as long as sufficient data are available to verify the model. Test procedures can be used to determine the single-anchor breakout strength in tension and in shear. The test results, however, are required to be evaluated on a basis statistically equivalent to that used to select the values for the concrete breakout method considered to satisfy provisions of 17.5.1.2. The basic strength cannot be taken

CODE **COMMENTARY**

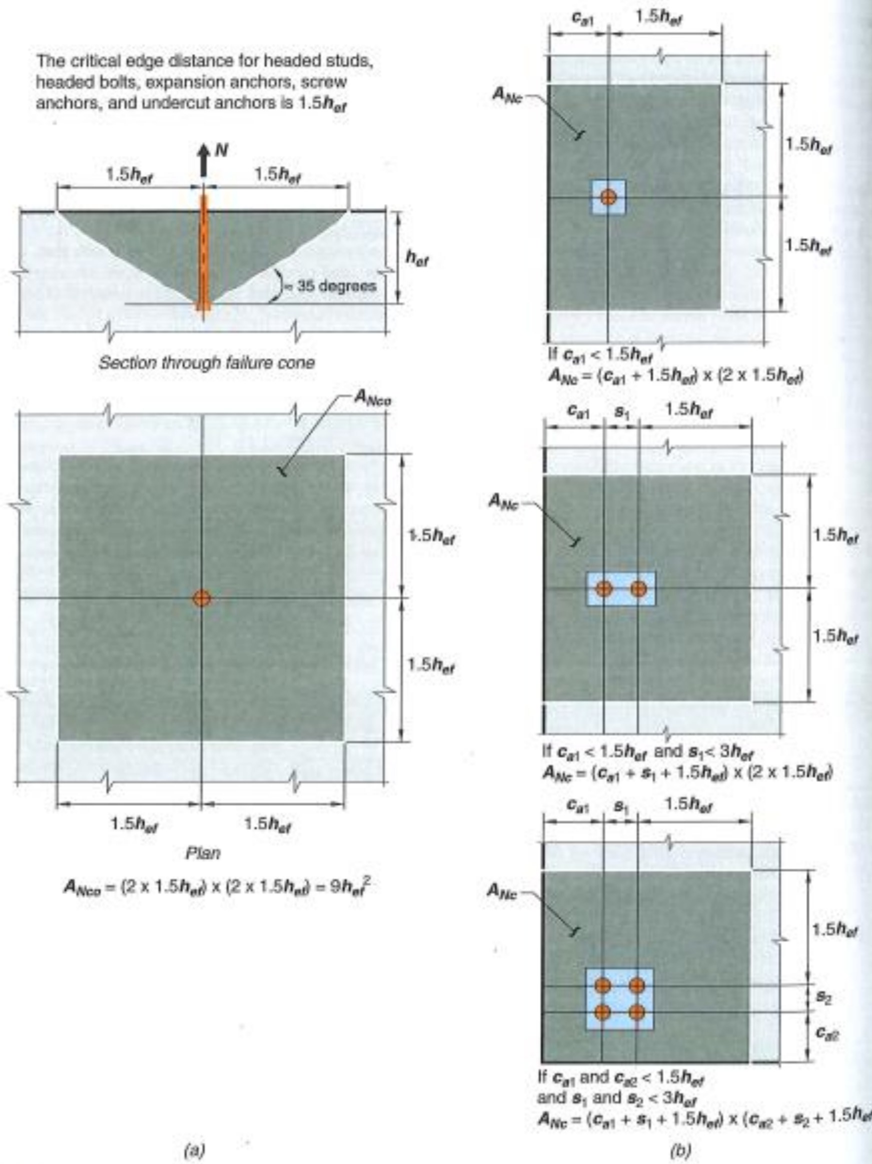
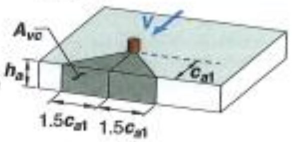


Fig. R17.6.2.1—(a) Calculation of A_{Nco} and (b) calculation of A_{Nc} for single anchors and anchor groups.

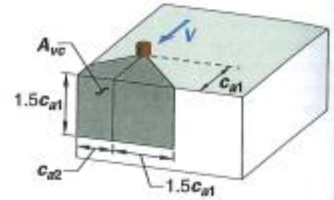
CODE

COMMENTARY

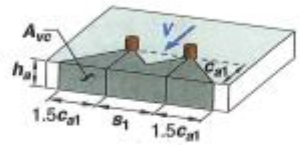
If $h_a < 1.5c_{a1}$
$A_{vc} = 2(1.5c_{a1})h_a$



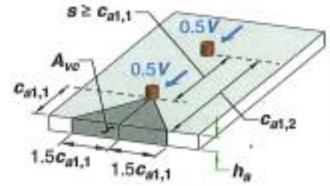
If $c_{a2} < 1.5c_{a1}$
$A_{vc} = 1.5c_{a1}(1.5c_{a1} + c_{a2})$



If $h_a < 1.5c_{a1}$ and $s_1 < 3c_{a1}$
$A_{vc} = [2(1.5c_{a1}) + s_1]h_a$

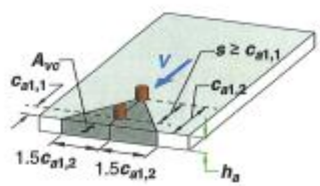


If $h_a < 1.5c_{a1}$
$A_{vc} = 2(1.5c_{a1,1})h_a$



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_{a1} is taken as $c_{a1,1}$.

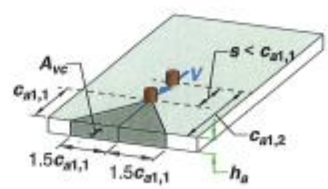
If $h_a < 1.5c_{a1}$
$A_{vc} = 2(1.5c_{a1,2})h_a$



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_{a1} is taken as $c_{a1,2}$.

Note: For $s \geq c_{a1,1}$, 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

If $h_a < 1.5c_{a1}$
$A_{vc} = 2(1.5c_{a1,1})h_a$



Case 3: Where $s < c_{a1,1}$, 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_{a1} is taken as $c_{a1,1}$.

Fig. R17.7.2.1b—Calculation of A_{vc} for single anchors and anchor groups.

CHAPTER 18

EARTHQUAKE - RESISTANT STRUCTURES

18.1 - SCOPE

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.

18.1 - GENERAL

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 - GENERAL (CONT'D)

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 - FOUNDATIONS

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 - FOUNDATIONS (CONT'D)

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 – 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 = 57sqrt(f'_c psi)

19.2.3 Modulus of rupture: For normal weight concrete, f_r, psi = 7.5sqrt(f'_c psi)

19.3 – CONCRETE DURABILITY

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 = 5ksi and maximum water to cement weight ratio is 0.4.

CODE | **COMMENTARY**

Table 19.3.2.1—Requirements for concrete by exposure class

Exposure class	Maximum w/cm[1,2]	Minimum f_c', psi	Additional requirements			Limits on cementitious materials
			Air content			
F0	N/A	2500	N/A			N/A
F1	0.55	3500	Table 19.3.3.1 for concrete or Table 19.3.3.3 for shotcrete			N/A
F2	0.45	4500	Table 19.3.3.1 for concrete or Table 19.3.3.3 for shotcrete			N/A
F3	0.40[3]	5000[3]	Table 19.3.3.1 for concrete or Table 19.3.3.3 for shotcrete			26.4.2.2(b)
			Cementitious materials[4] — Types			Calcium chloride admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	No type restriction	No type restriction	No type restriction	No restriction
S1	0.50	4000	II[5][6]	Types with (MS) designation	MS	No restriction
S2	0.45	4500	V[5]	Types with (HS) designation	HS	Not permitted
S3 Option 1	0.45	4500	V plus pozzolan or slag cement[7]	Types with (HS) designation plus pozzolan or slag cement[7]	HS plus pozzolan or slag cement[7]	Not permitted
S3 Option 2	0.40	5000	V[8]	Types with (HS) designation	HS	Not permitted
W0	N/A	2500	None			
W1	N/A	2500	26.4.2.2(d)			
W2	0.50	4000	26.4.2.2(d)			
			Maximum water-soluble chloride ion (Cl^-) content in concrete, percent by mass of cementitious materials[9,10]			
			Nonprestressed concrete	Prestressed concrete	Additional provisions	
C0	N/A	2500	1.00	0.06	None	
C1	N/A	2500	0.30	0.06		
C2	0.40	5000	0.15	0.06	Concrete cover[11]	

[1]The w/cm is based on all cementitious and supplementary cementitious materials in the concrete mixture.

[2]The maximum w/cm limits do not apply to lightweight concrete.

[3]For plain concrete, the maximum w/cm shall be 0.45 and the minimum f_c' shall be 4500 psi.

[4]Alternative combinations of cementitious materials to those listed are permitted for all sulfate exposure classes when tested for sulfate resistance and meeting the criteria in 26.4.2.2(c).

[5]For seawater exposure, other types of portland cements with tricalcium aluminate (C_3A) contents up to 10 percent are permitted if the w/cm does not exceed 0.40.

[6]Other available types of cement such as Type I or Type III are permitted in Exposure Classes S1 or S2 if the C_3A contents are less than 8 percent for Exposure Class S1 or less than 5 percent for Exposure Class S2.

[7]The amount of the specific source of the pozzolan or slag cement to be used shall be at least the amount that has been determined by service record to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slag cement to be used shall be at least the amount tested in accordance with ASTM C1012 and meeting the criteria in 26.4.2.2(c).

[8]If Type V cement is used as the sole cementitious material, the optional sulfate resistance requirement of 0.040 percent maximum expansion in ASTM C150 shall be specified.

[9]The mass of supplementary cementitious materials used in determining the chloride content shall not exceed the mass of the portland cement.

[10]Criteria for determination of chloride content are in 26.4.2.2.

[11]Concrete cover shall be in accordance with 20.5.

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. Except as

R19.3.3 *Additional requirements for freezing-and-thawing exposure*

R19.3.3.1 A table of required air contents for concrete to resist damage from cycles of freezing and thawing is

CHAPTER 20

STEEL REINFORCEMENT PROPERTIES, DURABILITY, & EMBEDMENTS

20.2 – NON-PRESTRESSED BARS AND WIRES

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 = 29000ksi

Table 20.2.2.4(a) – Nonprestressed deformed reinforcement: For shear and torsion, $f_{y\ max}$ = 60 ksi

20.3 – PRESTRESSING STRANDS, WIRES, AND BARS
20.4

20.3.2.3 Stress in bonded prestressed reinforcement at nominal flexural strength, f_{ps}

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

20.4 – Headed shear stud reinforcement

20.5 DURABILITY

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

21.2 – STRENGTH REDUCTION FACTORS (CONT'D)

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 (CONT'D)

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.

21.2 (CONT'D)

Table21.2.2 – 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 except phi = 0.75 if longitudinal steel is confined by spirals

21.2 (CONT'D)

Tension controlled, ie 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 SCOPE (CONT'D)

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 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 – Design assumptions for concrete (CONT'D)

22.2.2.3 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")

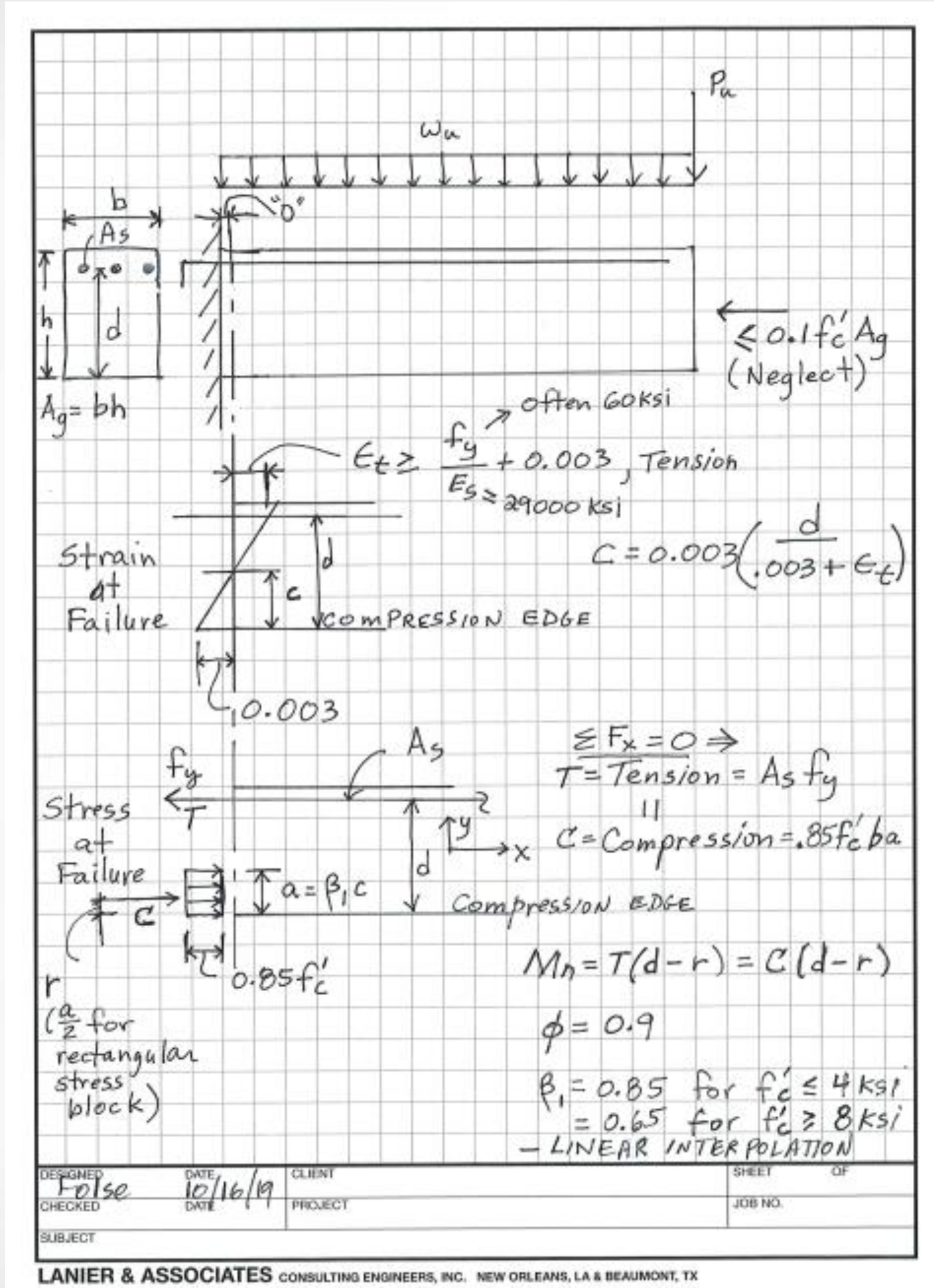
22.2.2.4 The equivalent rectangular concrete stress distribution: concrete stress $0.85f'_c$; depth of compression block " $\beta_1 c$", $\beta_1 = 0.85$ for f'_c LE 4 ksi; $= 0.65$ for f'_c GE 8 ksi; linear interpolation

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.

$f_{ps} = f_{pu}$ (1 - …) f_{pu} = 270 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.



Pu
ωu
"O"
b
As
h
d
Ag = bh
≤ 0.1 f'c Ag
(Neglect)
often 60ksi
Et ≥ fy / Es + 0.003, Tension
Es ≈ 29000 ksi
Strain at Failure
d
c
COMPRESSION EDGE
C = 0.003 (d / (.003 + Et))
0.003
As
fy
T
Stress at Failure
ΣFx = 0 ⇒
T = Tension = As fy
C = Compression = .85 f'c ba
y
x
a = β1 c
d
C
Compression EDGE
0.85 f'c
r
(a/2 for rectangular stress block)
Mn = T(d − r) = C(d − r)
φ = 0.9
β1 = 0.85 for f'c ≤ 4 ksi
= 0.65 for f'c ≥ 8 ksi
− LINEAR INTERPOLATION
DESIGNED Folse
DATE 10/16/19
CLIENT
SHEET
OF
CHECKED
DATE
PROJECT
JOB NO.
SUBJECT
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22.4 –
Axial strength or combined flexural and axial strength

22.4.2 Maximum axial compressive strength

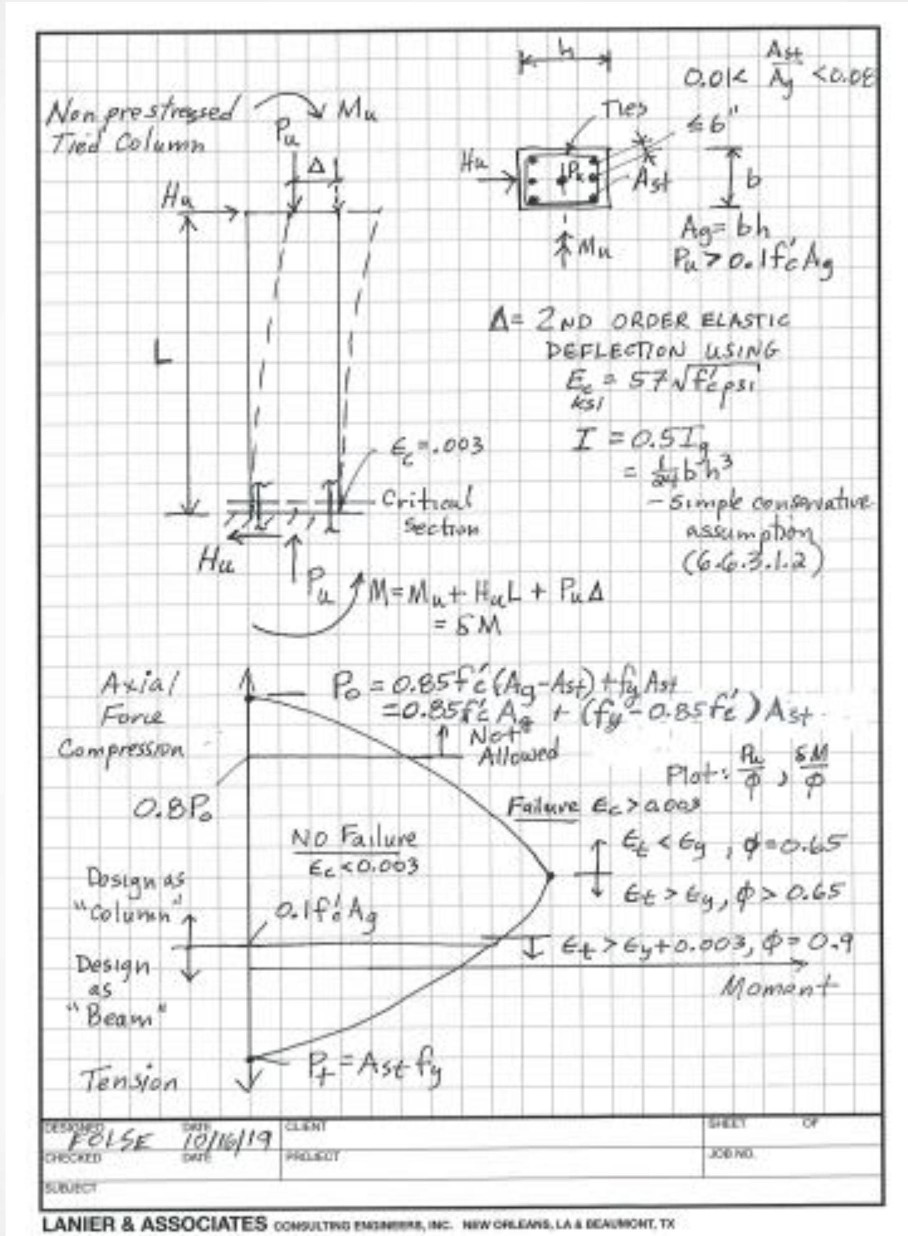
22.4.2.1 Nominal axial compressive strength P_n shall not exceed $P_{n,max}$ in accordance with Table 22.4.2.1, where $P_o = 0.85f'_c A_g + (f_y - 0.85f'_c)A_{st}$ for nonprestressed members and is calculated by Eq.(22.4.2.3) for prestressed members. The value of f_y shall be limited to 80 ksi.

22.4 – (CONT'D)

Table 22.4.2.1 – Maximum axial strength

Nonprestressed column with ties: P_n LE $0.80P_o$

Nonprestressed column with spiral: P_n LE $0.85P_o$


Non prestressed Tied Column
Mu
Pu
Δ
Hu
L
εc = .003
Critical Section
Hu
Pu
M = Mu + HuL + PuΔ = δM
h
0.01 < Ast/Ag < 0.08
Ties
≤ 6"
Hu
Pu
Ast
b
Mu
Ag = bh
Pu > 0.1f'c Ag
Δ = 2ND ORDER ELASTIC DEFLECTION USING
Ec ksi = 57√f'c psi
I = 0.5Ig = 1/24 bh³
– Simple conservative assumption (6.6.3.1.2)
Axial Force Compression
Po = 0.85f'c(Ag − Ast) + fy Ast = 0.85f'c Ag + (fy − 0.85f'c) Ast
Not Allowed
0.8Po
Plot: Pu/φ, δM/φ
Failure εc > 0.003
No Failure εc < 0.003
εt < εy, φ = 0.65
εt > εy, φ > 0.65
Design as "Column"
0.1f'c Ag
εt > εy + 0.003, φ = 0.9
Design as "Beam"
Moment
Pt = Ast fy
Tension
DESIGNED FOLSE
DATE 10/16/19
CLIENT
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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 LE 0.75(V_c + 8sqrt(f'_c)($b_w d$))

22.5.1.3 For nonprestressed members, V_c calculated by 22.5.5

22.5 – ONE-WAY SHEAR STRENGTH (CONT'D)

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 LE 1.5.

22.5 – ONE WAY SHEAR STRENGTH (CONT'D)

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 – ONE WAY SHEAR STRENGTH (CONT'D)

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(2 , or 8$(A_s/b_wd)^{0.333}$) sqrt(f'_c)b_wd , (equality at reinforcement ratio 0.0156)

Add axial compression stress/6 but not more than 0.05f'_c , and V_c shall not be taken greater than 5sqrt(f'_c) b_wd. Subtract axial tension stress/6 but V_c shall not be taken less than zero.

22.5 – ONE WAY SHEAR STRENGTH (CONT'D)

Reduction for lightweight concrete 19.2.4, generally 0.75.

Size effect modification factor: sqrt(2/(1+ 0.1d)) LE 1.0

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

$V_c = 8(A_s/b_w d)^{0.333}(b_w d)$(Size effect). (Same axial force adjustment as above)

22.5 – ONE WAY SHEAR STRENGTH (CONT'D)

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 GE 0.75V_c, transverse reinforcement shall be provided such that V_s GE (V_u / 0.75 - V_c)

22.5 – ONE WAY SHEAR STRENGTH (CONT'D)

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 ONE - WAY SHEAR STRENGTH (CONT'D)

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: $V_s = A_v f_{yt} d/s$

22.6 - TWO WAY SHEAR STRENGTH

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 - TWO WAY SHEAR STRENGTH (CONT'D)

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 - TWO WAY SHEAR STRENGTH (CONT'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 - TWO WAY SHEAR STRENGTH (CONT'D)

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 - TWO WAY SHEAR STRENGTH (CONT'D)

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 - TWO WAY SHEAR STRENGTH (CONT'D)

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

β is ratio of long to short side dimension of column, axial load, or reaction

α_s equals 40 for interior columns, 30 for edge columns, and 20 for corner columns

22.6 - TWO WAY SHEAR STRENGTH (CONT'D)

v_c LE least of (a), (b), or (c)

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

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

(c) $(2 + \alpha_s d/b_o)\text{sqrt}(f'_c)$ (Size effect)(Lightweight concrete factor)

22.6 - TWO WAY SHEAR STRENGTH (CONT'D)

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: $v_c = 2sqrt(f'_c)$(Size effect)(Lightweight concrete factor)

22.6 - TWO WAY SHEAR STRENGTH (CONT'D)

Where Headed shear stud reinforcement is present: For critical section 0.5d from column face, v_c equals the lesser of: 3sqrt(f'_c)(Size effect)(Lightweight concrete factor); (2 + 4/β)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 =2sqrt(f'_c)(Size effect)(Lightweight concrete factor)

22.6 - TWO WAY SHEAR STRENGTH (CONT'D)

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)(6sqrt(f'_c) where there is stirrup shear reinforcement or (0.75)(8sqrt(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 - TWO WAY SHEAR STRENGTH (CONT'D)

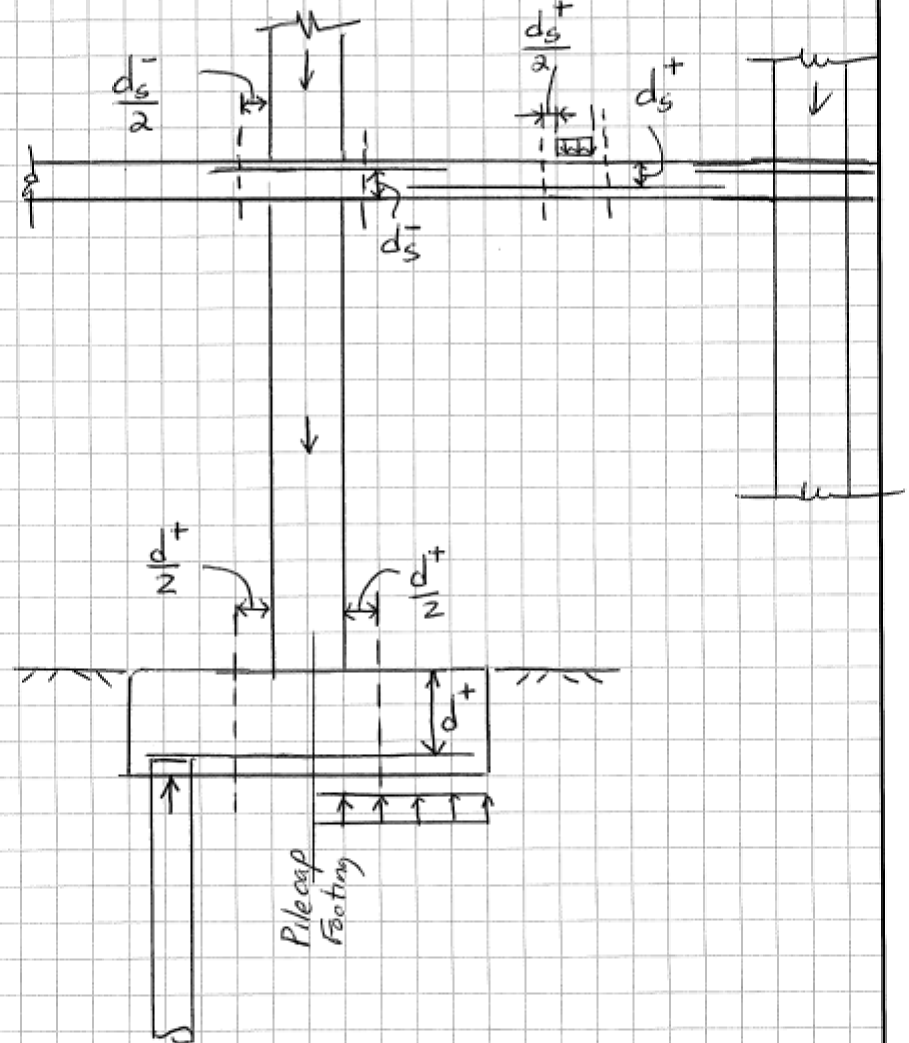
22.6.7 Two-way shear strength provided by single or multiple leg stirrups

For d GE 6inches and 16 stirrup bar diameters, $v_s = A_v f_{yt} / b_o s$, 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 - TWO WAY SHEAR STRENGTH (CONT'D)

22.6.8 Two-way shear strength provided by headed shear stud reinforcement

$v_s = A_v f_{yt} / b_o s$, 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 A_v / s GE $2b_o \text{sqrt}(f'_c) / f_{yt}$.



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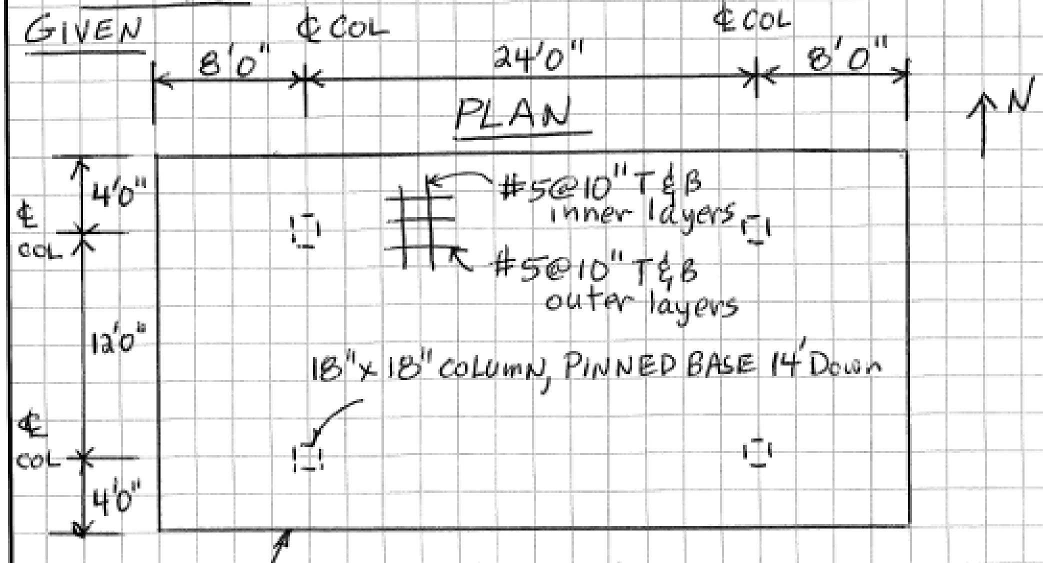
EXAMPLE

GIVEN



Reinforced concrete slab $h = 12"$, $d \approx 10"$

Normalweight, $f'_c = 5.0$ ksi, $f_y = 60$ 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

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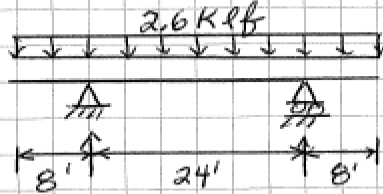
SOLUTION

LOAD CASE $1.2D + 1.6L$

$w_u = 1.2(150 psf) + 1.6(50 psf) = 260 psf$

$* (4 + \frac{12}{2}) = 2.6 klf$

Long direction load on 8' wide beam



$M_u^- = \frac{1}{2}(2.6)(8^2) = 83.2^{\prime K}$

$M_u^+ = \frac{1}{8}(2.6)(24^2) - 83.2$
$= 187.2 - 83.2$
$= 104$ ft-K

Check 8' wide slab w/ 10 #5, d = 10"

$A_s = 10(.31) = 3.1 IN^2$

One way shear: $\rho = \frac{3.1}{96(10)} = 0.00341$ $\quad (A_{s\,min} = .0018 A_g$

$A_v = 0 < A_{v\,min} \Rightarrow V_c = 8(\lambda_s = 1.0)(\lambda = 1.0)(.00341)^{1/3}\sqrt{5000}(96)(10)$

d ≤ 10" Normal weight $(N_u = 0)$

$= 81.9^K$

$* (\phi = 0.75) = \underline{61.4^K}$

$V_{u\,℄} = 2.6 klf (12') = 31.2^K < 61.4^K$

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

Flexure: $C = T = 3.1 IN^2 (60 ksi) = 186^K$

$a = 186^K / .85(5)(96) = 0.46"$

$c = a/\beta_1 = .46/.8 = 0.57"$

$\epsilon_t = .003 (10 - .57)/.57 = 0.05 > \frac{60}{29000} + .003$
$= .00507$

$\phi M_n = .9(186^k)(10 - \frac{.46}{2}) = 1635 IN\text{-}K = 136$ ft-K
> 104 ft-K
> 83.2 ft-K

(No need to reduce M_u to face of support) OK

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SOLUTION $1.2D+1.6L$ (CONT)

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

$= 51.6^k$

-No moment transfer

$d = 10''$ $\quad b_o = 4(18+10) = 112''$

$v_u = \frac{51.6^k}{10(112)} = .046\,ksi$

$v_n = v_c$ — No shear reinforcement

$\phi v_c = 0.75(4)(\sqrt{5000}) = 212\,psi = 0.212\,ksi$
$> .046\,ksi$ ok

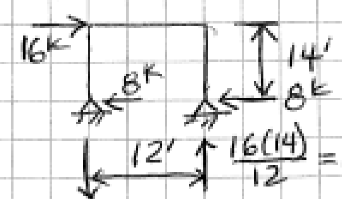
$1.2D+1.0L+1.0W$ – short direction span

$w_u = 1.2(150)+1.0(50) = 230\,psf$
$*\left(8+\frac{34}{2}\right) = 4.6\,klf$

$\frac{1}{2}(4.6)(4^2) = 36.8\,ft\text{-}k$
$\frac{1}{8}(4.6)(12^2) = 82.8\,ft\text{-}k$
$M_u^+ = 82.8-36.8 = 46.0\,ft\text{-}k$
$< 136\,ft\text{-}k$ ok

Lateral Loads – check leeward column

$.8^k/ft\left(\frac{40'}{2}\right)$
$= 16^k$



$\frac{16(14)}{12} = 18.7^k$

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One way shear:

$V_{u_{\text{CL}}} = 4.6\,\text{klf}(6') + 18.7 = 46.3^k \quad < 61.4^k \quad \text{OK}$

Flexure:

$M^-_{u_{\text{CL}}}$:

$\frac{1}{2}(4.6)(4^2) = 36.8^{'k}$

$8^k(14') = 112^{'k}$

$36.8 + 112 = \underline{149^{'k}} > 136^{'k}$

To column Face

$149^{'k} - 46.3^k(.75') = 114^{'k} < 136^{'k}$ OK

Check windward side column M_u^+ for $0.9D + 1.0W$

$.9D = .9(150)(20) = 2.7^k$

$\frac{1}{2}(2.7)(4^2) = 21.6$

21.6, $112^{'k}$

$M^+_{u_{\text{CL}}} = 112 - 21.6 = 90.4^{'k} < 136^{'k}$ OK

No need to reduce to face of column

Two way shear – leeward column

$$V_u = \left[\frac{20(40)}{4} - 1.25^2\right] 230\,\text{psf} + 18.7^k = 64.3^k$$

$M_u = 149^{'k} \quad (114^{'k}?)$

Fraction of $M_u = 149^{'k}$ transferred by flexure:

$$\gamma_f = \frac{1}{1 + \frac{2}{3}\sqrt{\frac{18+10}{18+10}}} = \frac{1}{1.667} = 0.6$$

$.6(149) = 89.4$

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

$\phi M_n = 136^{'k}(54/96) = 76.5^{'k} < 89.4^{'k}$ N.G

$76.5^{'k} > .6(114) = 68.4^{'k}$ OK

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Two way shear (cont)

Moment transferred by eccentricity of shear

$$\gamma_v = 1 - \gamma_f = 1 - 0.6 = 0.4$$
$$\times 149^{K} = 59.6 \text{ ft-k}$$

$$J = \frac{d(c_1+d)^3}{6} + \frac{(c_1+d)d^3}{6} + \frac{d(c_2+d)(c_1+d)^2}{2}$$
$$= \frac{10(18+10)^3}{6} + \frac{(18+10)(10^3)}{6} + \frac{10(18+10)(18+10)^2}{2}$$
$$= 36{,}587 + 4667 + 109760$$
$$= 151{,}000 \text{ IN}^4$$

$$c = \frac{18+10}{2} = 14 \text{ in}$$

$$v_{uv} = \frac{64.3^K}{10(112)} = 0.0574 \text{ ksi} = 57.4 \text{ psi}$$

$$v_u = 57.4 \text{ psi} + \frac{59.6 \text{ ft-k}(12 \text{ in/ft})(14 \text{ in})(1000 \text{ lb/k})}{151{,}000 \text{ IN}^4}$$
$$= 57.4 + 66.3$$
$$= 123.7 \text{ psi} \quad \text{(inside face)}$$

$$\phi v_n = 0.75(4)(\sqrt{5000}) = 212 \text{ psi} > 123.7 \text{ psi}$$
OK

CONCLUSION: Slab design shown complies with ACI 318-19

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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 GE 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-TORSIONAL STRENGTH (CONT'D)

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 GE $0.75T_{cr}$ and T_u is required to maintain equilibrium, the member shall be designed to resist T_u .

22.7 - TORSIONAL STRENGTH (CONT'D)

22.7.3.2 In a statically indeterminate structure where T_u GE $0.75T_{cr}$ 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 THRESHOLD 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)

22.7.4 THRESHOLD TORSION

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.??)

22.7.4 THRESHOLD TORSION (CONT'D)

Prestressed member:

Nonprestressed member subjected to axial force

Table 22.7.4.1(b)

22.7.5 CRACKING TORSION

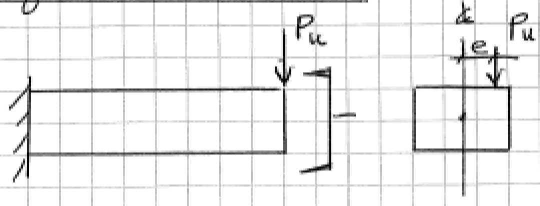
Nonprestressed normalweight member, conservatively neglecting axial compression if any :

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

Prestressed member

Nonprestressed member subjected to axial force

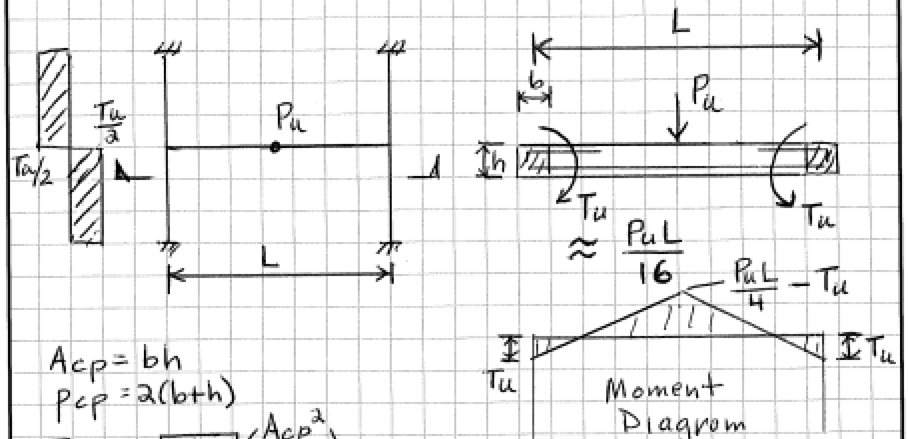
Equilibrium Torsion



$T_u = P_u e$

Compatibility Torsion

Plan



$A_{cp} = bh$

$p_{cp} = 2(b+h)$

$T_{cr} = 4\sqrt{f'_{c\,psi}}\left(\frac{A_{cp}^2}{p_{cp}}\right)$

$T_{th} = T_{cr}/4$

Permitted to neglect torsion where:
Factored torsion $\leq (\phi = 0.75)\, T_{th}$

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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) $T_n = 2 A_o f_{yt} \cot\theta (A_t / s)$

(b) $T_n = 2 A_o f_y \tan\theta (A_l / p_h)$

22.7.6 TORSIONAL STRENGTH (CONT'D)

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

Θ = 45 degrees for nonprestressed members

A_t = area of one leg of the closed stirrup resisting torsion

A_l = area of longitudinal torsional reinforcement

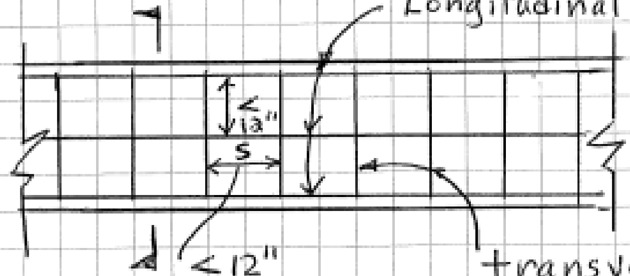
P_h = perimeter of the centerline of the outermost closed stirrup.

22.7.7

22.7.7 Cross-sectional limits for solid and hollow sections

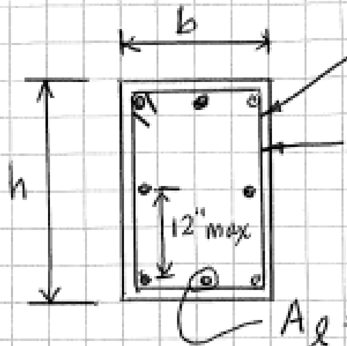
Torsional Strength

Longitudinal Reinforcement



transverse reinforcement

< 12"
< $P_h/8$
(9.7.5, 9.7.6



closed stirrup, perimeter P_h, enclosed area A_{oh}

Transverse reinforcement not required for shear of this load case
A_t = Area one leg

A_ℓ = Longitudinal reinforcement not required for flexure

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

$$T_n = \text{minimum} \begin{bmatrix} \dfrac{2 A_o A_t f_{yt}}{s} \cot\theta \\ \dfrac{2 A_o A_\ell f_y}{P_h} \tan\theta \end{bmatrix}$$

Generally, $\theta = 45^\circ$, $A_o = 0.85 A_{oh}$

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22.8 BEARING

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 FRICTION

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 – SHEAR FRICTION (CONT'D)

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 μ is the coefficient of friction in accordance with Table 22.9.4.2.

22.9 – SHEAR FRICTION (CONT'D)

Table 22.9.4.2 Coefficients of friction (reduce according to 19.2.4 for lightweight concrete, but λ_{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 $\mu = 1.0$

22.9 – SHEAR FRICTION (CONT'D)

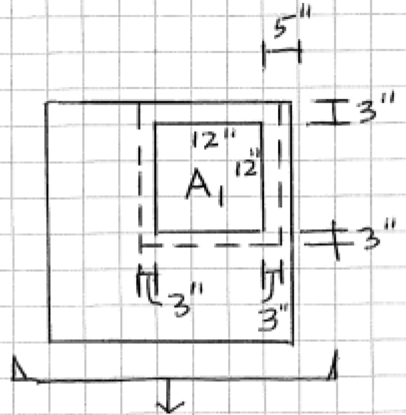
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened $\mu = 0.6$

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 – SHEAR FRICTION (CONT'D)

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



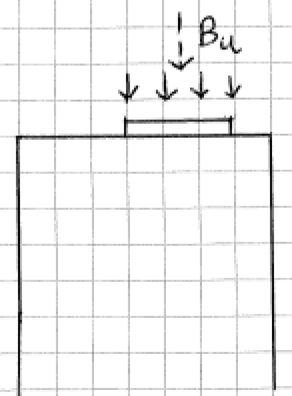
$A_1 = 12(12) = 144 \text{ IN}^2$

$A_2 = 18(18) = 324 \text{ IN}^2$

$\frac{A_2}{A_1} = \frac{324}{144} = 2.25$

> 2.0

use 2.0



$B_u \le \phi B_n \le 0.65 B_n$

$B_n = .85 f'_c A_1 \sqrt{A_2/A_1}$

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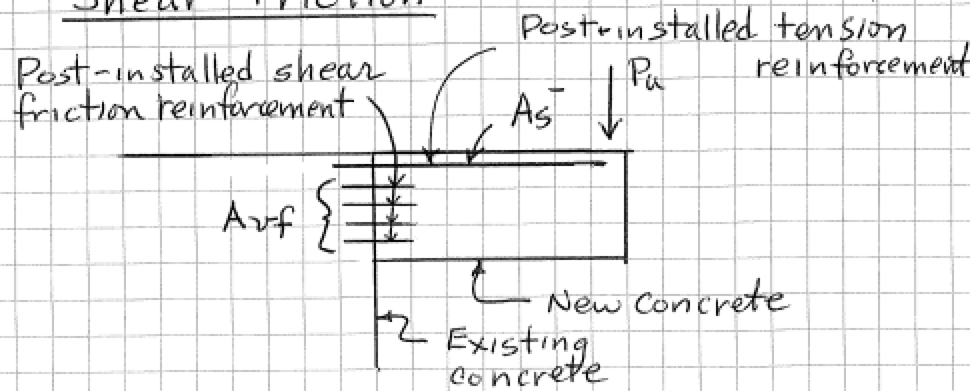
Shear Friction



$$V_u \le (\phi V_n = 0.75\, V_n)$$

$$V_n = \mu A_{vf} f_y$$

$\mu = 1.4\lambda$ Concrete placed monolithically

$ = 0.6\lambda$ Concrete placed against --- not intentionally roughened

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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 – General

23.2.1 Strut-and-tie models shall consist of struts and ties connected at nodes to form an idealized truss ion 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 – (CONT'D)
23.4
23.5
23.6

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
23.8
23.9
23.10
23.11

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 – SCOPE

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

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.

24.2 – DEFLECTIONS (CONT'D)

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)

24.2 – DEFLECTIONS (CONT’D)

Floors not supporting or attached to deflection sensitive nonstructural elements: Limit immediate deflection due to live load to: 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: Span / 480 (Limit is 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 , ksi = 57sqrt(f'_c psi) for normalweight concrete)

24.2 DEFLECTIONS (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: $I_e = I_{cr} / [1 - (.667\ M_{cr} / M_a)^2 (1 - I_{cr} / I_g)]$

24.2 DEFLECTIONS (CONT'D)

(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 DEFLECTIONS (CONT'D)

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 $R = M_{cr} / M_a$) and $M_{cr} = (f_r + f_{pe}) I_g / y_t$

24.2 DEFLECTIONS (CONT'D)

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: 2 / (1 + 50(compression reinforcement ratio)

24.2 DEFLECTIONS (CONT'D)

24.2.4.1.2 Use compression reinforcement ratio at midspan for simple and continuous spans, and at the support for cantilevers.

24.3 – Distribution of flexural reinforcement in one-way slabs and beams

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 – Shrinkage and temperature reinforcement

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

24.5 – Permissible stresses in prestressed concrete flexural members

24.5 – (CONT'D)

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.

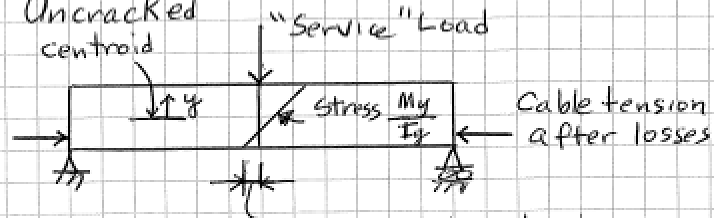
Class U Uncracked f_t LE 7.5sqrt(f'_c)

Class T Transition f_t LE 12sqrt(f'_c)

Class C Cracked f_t GT 12sqrt(f'_c)

Classification of Prestressed Concrete Flexural Members

Class U "Uncracked"



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

Class T "Transition"

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

Class C "Cracked

Maximum Tensile stress $> 12\sqrt{f'_c}$

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CHAPTER 25 – REINFORCEMENT DETAILS

25.1 Scope

25.1- Scope: Minimum spacing; Standard hooks; Development; Splices; Bundled reinforcement;…

25.2 – Minimum Spacing of 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 $1.33d_{agg}$

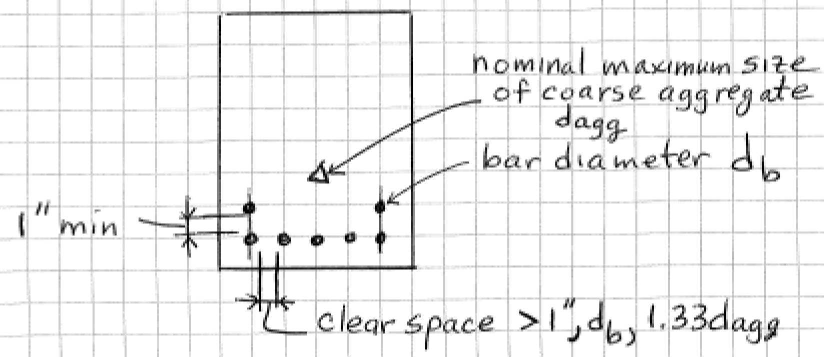
25.2.2 For parallel nonprestressed reinforcement placed in two or more horizontal layers, reinforcement in the upper layers shall be placed directly above reinforcement in the bottom layer with a clear spacing between layers of at least 1 inch.

25.2 MINIMUM SPACING (CONT'D)

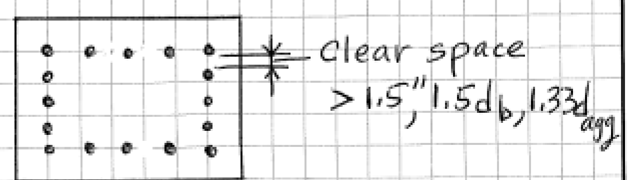
25.2.3 For longitudinal reinforcement in columns, pedestals, struts, and boundary elements in walls, clear spacing between bars shall be at least the greatest of 1.5 in., 1.5 d_b, and 1.333d_{agg}.

Minimum Spacing of Reinforcement

Beam Section



Column Section



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25.3

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 is 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 - DEVELOPMENT OF REINFORCEMENT (CONT'D)

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 : $l_d = (f_y / 20sqrt(f'_c)) \; d_b$

(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

$l_{dh} = (f_y / 55\text{sqrt}(f'_c)\ )\ d_b{}^{1.5}$
plus modification factors

25.4.4 Development of headed deformed bars in tension

25.4 DEVELOPMENT OF REINFORCEMENT (CONT'D)

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 $4A_b$

25.4 DEVELOPMENT OF REINFORCEMENT (CONT'D)

(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 $3d_b$

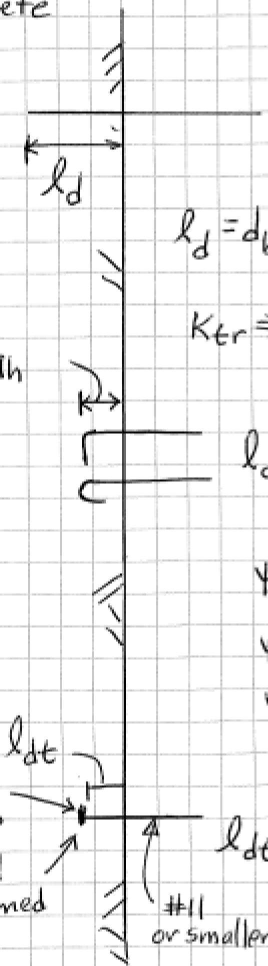
25.4 DEVELOPMENT OF REINFORCEMENT (CONT'D)

25.4.4.2 Development length l_{dt} for headed deformed bars in tension shall be the longest of (a) through (c):

(a) $l_{dt} = (f_y / 75\text{sqrt}(f'_c)) \, d_b^{1.5}$ (b) $8d_b$ (c) 6 inches ; plus modification factors and prescriptive requirements for reinforcement details

Tension Development Lengths

Concrete f'_c



$$\ell_d = d_b\left(\frac{3}{40}\,\frac{f_y}{\lambda\sqrt{f'_c}_{psi}}\,\frac{\psi_t\,\psi_e\,\psi_s\,\psi_g}{\boxed{\left(\frac{c_b+K_{tr}}{d_b}\right)}}\right)$$

"top bar" factor 1.3 >12" concrete below

Epoxy 1.2 to 1.5

Size .8 for #6 and smaller

$\psi_g \rightarrow$ 1.15 Gr80, 1.3 Gr100

$\left(\frac{c_b+K_{tr}}{d_b}\right) \Rightarrow < 2.5$

$K_{tr} \Rightarrow$ can use 0

$$\ell_{dh} = \frac{f_y\,\psi_e\,\psi_r\,\psi_o\,\psi_c}{55\lambda\sqrt{f'_c}}\,d_b^{1.5}$$

ψ_e: Epoxy 1.2

$> 8d_b$

> 6 inch

$\psi_r = 1.0$ for confined #11 and smaller
$= 1.6$ other

$\psi_o = 1.0$ for #11 and smaller w/ cover
$= 1.25$ other

$\psi_c = f'_c/15 + 0.6$, $f'_c < 6$ ksi (f'_c in ksi)
$= 1.0$, $f'_c \geq 6$ ksi

$A_{brg} > 4A_b$

Headed deformed bar

$$\ell_{dt} = \frac{f_y\,\psi_e\,\psi_p\,\psi_o\,\psi_c}{75\sqrt{f'_c}}\,d_b^{1.5}$$

#11 or smaller — Normalweight Concrete —

$> 8d_b$

> 6 inch

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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_{dh} within ties or stirrups perpendicular to ℓ_{dh} at $s \leq 3d_b$

(b) The first tie or stirrup shall enclose the bent portion of the hook within $2d_b$ of the outside of the bend

where d_b is the nominal diameter of the hooked bar.

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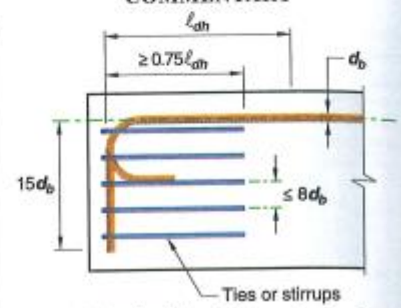

Fig. R25.4.3.3a—Confining reinforcement placed parallel to the bar being developed that contributes to anchorage strength of both 90- and 180-degree hooked bars.

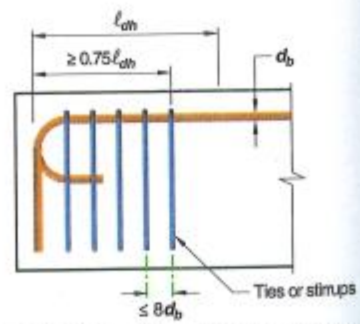

Fig. R25.4.3.3b—Confining reinforcement placed perpendicular to the bar being developed, spaced along the development length ℓ_{dh}, that contributes to anchorage strength of both 90- and 180-degree hooked bars.

R25.4.3.4 Hoooked 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 free 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.

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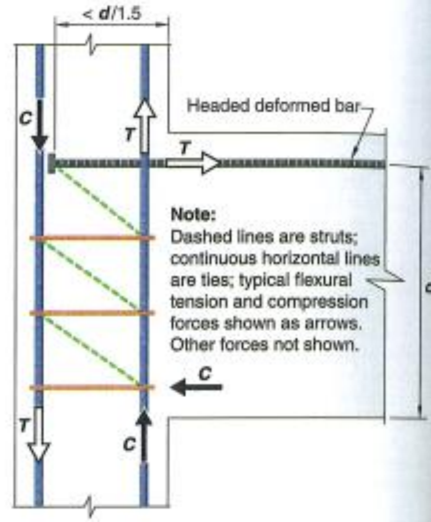

***Fig. R25.4.4.2d**—Breakout failure precluded in joint by providing transverse reinforcement to enable a strut-and-tie mechanism.*

25.4.4.3 For the calculation of ℓ_{dt}, modification factors ψ_e, ψ_p, ψ_o, and ψ_c shall be in accordance with Table 25.4.4.3.

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

Modification factor	Condition	Value of factor
Epoxy ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Parallel tie reinforcement ψ_p	For No. 11 and smaller bars with $A_{tt} \geq 0.3A_{hs}$ or s[1] $\geq 6d_b$[2,3]	1.0
	Other	1.6
Location ψ_o	For headed bars: (1) Terminating inside column core with side cover to bar $\geq$ 2.5 in.; or (2) With side cover to bar $\geq 6d_b$	1.0
	Other	1.25
Concrete strength ψ_c	For $f_c' < 6000$ psi	$f_c'/15{,}000 + 0.6$
	For $f_c' \geq 6000$ psi	1.0

[1]s is minimum center-to-center spacing of headed bars.

[2]d_b is nominal diameter of headed bar.

[3]Refer to 25.4.4.5.

R25.4.4.3 The epoxy factor 1.2 is based conservatively on the value used for epoxy-coated standard hooks. The location factor ψ_o accounts for the confinement provided by the reinforcement within columns and large side cover for other members.

The factor ψ_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).

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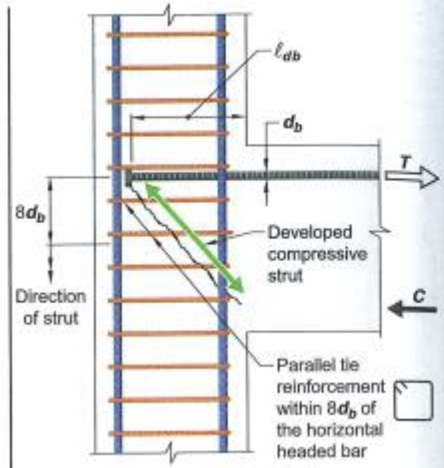
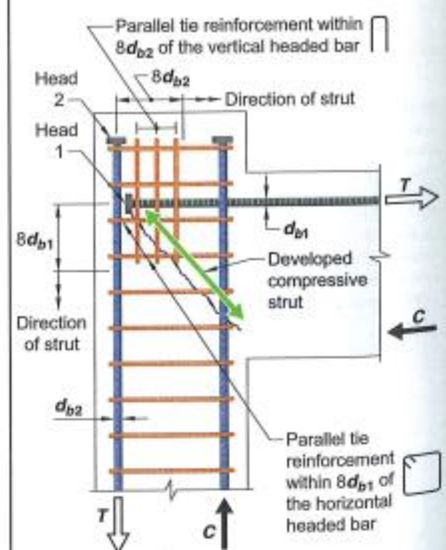


(a) Horizontal headed bars



(b) Vertical and horizontal headed bars

Fig. R25.4.4.4—Ties or 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} = (f_y / 50sqrt(f'_c)) d_b , divide by 0.75 for lightweight concrete

but not less than 0.0003 f_y $d_{b;}$ (can multiply l_{dc} by 0.75 for special confinement details)

25.4.10 Reduction of development length for excess reinforcement

25.4 DEVELOPMENT OF REINFORCEMENT (CONT'D)

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 DEVELOPMENT OF REINFORCEMENT (CONT'D)

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 Splices (CONT’D)

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 Splices (CONT'D)

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 LE 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 - Bundled Reinforcement

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 -Bundled Reinforcement (CONT'D)

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 Bundled Reinforcement (CONT'D)

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 – TRANSVERSE REINFORCEMENT

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 – TRANSVERSE REINFORCEMENT (CONT'D)

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 16 d_b of longitudinal bar, 48 d_b of tie bar, and smallest dimension of member

25.7.2 TIES (CONT'D)

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 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_{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 GE $0.45(A_g / A_{ch} - 1) f'_c / f_{yt}$; 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.

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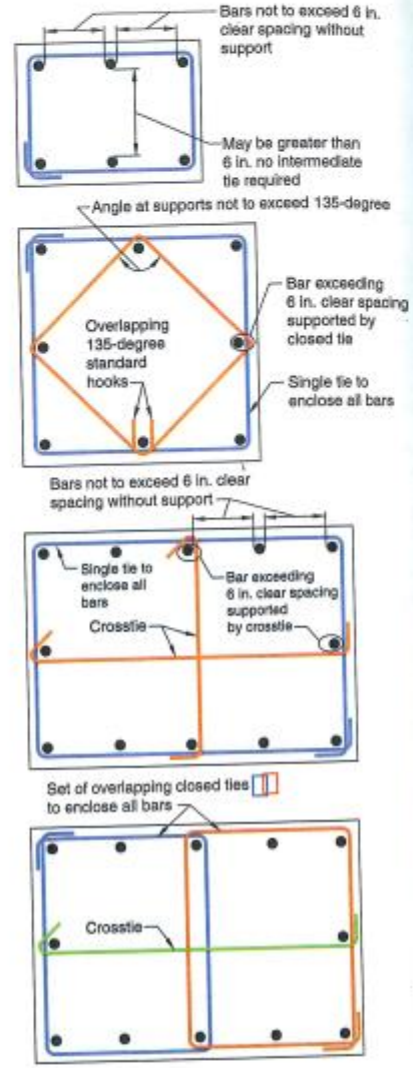

Fig. R25.7.2.3a—Illustrations to clarify measurements between laterally supported column bars and rectilinear tie anchorage.

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
26.3
26.4
26.5

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

26.6
26.7
26.8
26.9
26.10

26.6 – Reinforcement materials and construction requirements

26.7 – Anchoring to Concrete

26.8 – Embedments

26.9 – Additional requirements for precast concrete

26.10 – Additional requirements for prestressed concrete

26.11
26.12
26.13

26.11 – Formwork

26.12 – Evaluation and acceptance of hardened concrete

26.13 – Inspection

CHAPTER 27

STRENGTH EVALUATION OF EXISTING STRUCTURES

27.1 SCOPE (CONT'D)

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 NONLINEQR RESPONSE HISTORY ANALYSIS