

ACI 340.4R-91

DESIGN HANDBOOK

VOLUME 3

Two-Way Slabs

In accordance with the strength design method
of ACI 318-89



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DESIGN HANDBOOK

In Accordance with the Strength Design Method of ACI 318-89

VOLUME 3 — Two-Way Slabs

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FOREWORD

The three volumes of the ACI Design Handbook are intended for use by persons having a general familiarity with the strength design method and with "Building Code Requirements for Reinforced Concrete (ACI 318-89)."

This volume, Volume 3, presents Design Aids for use in the engineering design and analysis of reinforced concrete slab systems carrying loads by two-way action. These Design Aids consist of tables and graphs that eliminate the need to perform certain routine calculations repeatedly.

The Design Aids (buff-colored pages) have been developed for use in simplifying the application of both the Direct Design Method and the Equivalent Frame Method as outlined in ACI 318-89. Design Aids are provided for the selection of the slab thickness, for the computation of slab, beam, and column stiffnesses; and for the selection of stiffness, carry-over, and fixed-end moment factors for slab-beam elements used in the Equivalent Frame Method.

The Commentary (gray pages) describes the procedures followed in solving the Design Examples (white pages) and explains the use and derivation of the Design Aids.

History of Slabs Portion of Handbook

The first edition of this material was published as Division II of the 1981 edition of Volume 1 of the Design Handbook. That version was written soon after requirements for slab thickness and shear-moment transfer—as well as the Direct Design Method and the Equivalent Frame Method—had been introduced in the 1971 Code. Therefore, selection of slab thickness and checking of shear requirements were emphasized, although in practice they are a relatively small part of the design process. Design Examples and Design Aids still reflect this emphasis on determining slab thickness and providing for shear-moment transfer.

The second edition was issued in 1985 as a separate spiral-bound "Supplement to Design Handbook Volume 1—Design of Two-Way Slabs".

Updating to ACI 318-89

This third edition updates the Design Aids and their Commentary and the Design Procedures and Design Examples to comply with two provisions introduced in ACI 318-89:

- Table 9.5(c), which was added to simplify determination of minimum thickness required for slabs without internal beams (This table is now SLABS 1.1 and is used in the related steps of Design Procedures and Examples.)
- a third equation—Eq. (11-37) in ACI 318-89—which with preceding and following equations is used to determine nominal shear strength provided by concrete (This change is reflected in SLABS 9 and the related steps in Design Procedures and Examples.)

The volume at hand, like its predecessors, illustrates the coefficient method of calculating bending moments introduced in ACI 318-83 for the Direct Design Method. This coefficient method is used in Slabs Example 1.

Although the coefficient method eliminates determination of the relative stiffness factor α_{ec} in the Direct Design Method, Design Aids for this determination have been retained in this volume for use by designers desiring a more refined calculation, as is acceptable according to the Commentary on Section 13.6 of ACI 318-89. Use of these Design Aids is illustrated in an alternate example.

Note that although Grade 60 reinforcement is commonly used in floor systems, Grade 40 reinforcement has been used in the examples because it yields thinner minimum slab thickness as required by deflection equations 9.11, 9.12, and 9.13 for slabs on beams. Thinner slabs make shear calculations more critical for illustration purposes.

Similarly, although drop panels and column capitals are not commonly used together, and it is rare to have a floor slab with spandrel beams on two sides and not on the other two sides, these features are used in the Slabs Examples to illustrate calculation procedures.

Acknowledgments

Many members of ACI Committee 340 have contributed to this volume, giving time and effort to developing and reviewing its contents. Especially noteworthy are the contributions of:

- Prof. Edward G. Nawy of Rutgers University, who formulated much of the slabs material and reviewed all of it, and
- Prof. Sidney H. Simmonds of the University of Alberta, who developed the SLABS 9 series of Design Aids.

Handbook Users' Comments

Committee 340 welcomes users' comments aimed at improving future editions. Such comments should be addressed to Committee 340 in care of the American Concrete Institute, P.O. Box 19150, Detroit, Michigan 48219, U.S.A.

NOTATION

- a = length of column section considered rigid (one-half slab thickness) or length of rigid column section at near end (square column with boxed capital), in.
 = factor for computing K_t , in.⁻¹
 A_c = area of critical shear section = $b_o d$, in.²
 A_s = area of nonprestressed tension reinforcement, in.²
 A_{s1} = area of steel per ft of slab width, in.² (Crack Control)
 b = width of compression portion of beam, in.
 = length of rigid column section at far end (square column with boxed capital), in.
 b' = capital depth measured from lower surface of slab, in.
 b_c = width of column transverse to direction of applied moment (= c_2 when there is no capital), in.
 b_{dp} = size of square drop panel, ft
 b_o = perimeter of critical shear section, in.
 b_w = width of beam stem, in.
 c_{c1} = clear concrete cover to surface of outer layer of reinforcement, in. (Crack Control)
 c_1 = dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined
 c_2 = dimension of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which moments are being determined
 C = torsion constant, in.⁴ See Eq. (13-7).
 = moment carry-over factor (Equivalent Frame Method)
 d = distance from extreme compression fiber to centroid of tension reinforcement, in.
 d_{b1} = diameter of reinforcement bar closest to concrete extreme tensile surface, in. (Crack Control)
 d_c = distance from extreme tensile surface to center of closest tensile reinforcing bar, in. (Crack Control)
 d_{dp} = distance from extreme compression fiber to centroid of tension reinforcement of drop panel, in.
 E_c = modulus of elasticity of concrete = $(w_c^{1.5})33\sqrt{f'_c}$, psi
 E_{cb} = modulus of elasticity of beam concrete, ksi
 E_{cc} = modulus of elasticity of column concrete, ksi
 E_{cs} = modulus of elasticity of slab concrete, ksi
 f'_c = specified compressive strength of concrete, psi
 f_s = calculated stress in reinforcement at service loads, ksi (Crack Control)
 f_y = specified tensile yield strength of reinforcement, psi.
 F = flexural coefficient = $bd^2/12,000 = M_u/K_u$
 h = overall thickness of member, in.
 h_c = size of column in direction of bending (= c_1 when there is no capital), in.
 h_{dp} = total thickness of drop panel (slab thickness plus drop), in.
 h_o = diameter of round column, in.
 h_s = thickness of slab, in.
 $h_{s(d)}$ = minimum slab thickness, governed by deflection requirements, in.
 $h_{s(s)}$ = minimum slab thickness governed by shear requirements, in.
 I_b = moment of inertia about centroidal axis of gross section of beam (including part of adjacent slab section as defined in ACI 318-89, Section 13.2.4), in.⁴
 I_c = moment of inertia of gross section of column, in.⁴
 I_s = moment of inertia about centroidal axis of gross section of slab = $h_s^3/12$ times width of slab, in.⁴
 J_c = property of assumed critical section analogous to polar moment of inertia
 k = steel strength factor used in evaluation of $h_{s(d)}$
 k_c = column stiffness coefficient (Equivalent Frame Method)
 k_s = flexural stiffness coefficient (Equivalent Frame Method)
 k_1 = perimeter shear stress factor, in.⁻²
 k_2 = moment-shear transfer stress factor, in.⁻² ft⁻¹
 k_3 = moment-shear transfer stress factor, in.⁻² ft⁻¹
 k_2' = moment-shear transfer stress factor for square column or capital, in.⁻² ft⁻¹
 k_3' = moment-shear transfer stress factor for square column or capital, in.⁻² ft⁻¹
 K = fracture coefficient used in crack width determination to obtain maximum allowable spacing of reinforcement in two-way slabs and plates (Crack Control)
 K_b = flexural stiffness of beam; moment per unit of rotation = $k_b E_{cb} I_b / l$ (in Equivalent Frame Method); $4E_{cb} I_b / l$ (in Direct Design Method)
 K_c = flexural stiffness of column; moment per unit of rotation = $k_c E_{cc} I_c / l_c$ (in Equivalent Frame Method); $4E_{cc} I_c / l_c$ (in Direct Design Method)

K_{ec} = flexural stiffness of equivalent column; moment per unit of rotation; defined by:

$$1/K_{ec} = 1/\Sigma K_c + 1/\Sigma K_t$$

K_s = flexural stiffness of slab; moment per unit of rotation = $k_s E_{cs} I_s / l_1$ (in Equivalent Frame Method); = $4 E_{cs} I_s / l_1$ (in Direct Design Method)

K_t = torsional stiffness of transverse torsional member; moment per unit of rotation =

$$9 E_{cs} C / \left\{ l_2 \left(1 - \frac{c_2}{l_2} \right)^3 \right\}$$

K_u = strength coefficient of resistance = $M_u / F = 12,000 M_u / (b d^2) = \phi f_c' \omega (1 - 0.59 \omega)$

ΣK = combined flexural stiffness of slab and column

l = span length of beam, center-to-center of supports

= width of slab strip used to calculate α

l_a = average of l_1 and l_s

l_c = height of column

l_l = longer of l_1 or width of design frame l_2

l_n = length of clear span in direction that moments are being determined, measured face-to-face of supports; measured face-to-face of beams in slabs with beams

l_s = shorter of l_1 or width of design frame l_2

l_u = unsupported height of column

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

l_2 = length of slab span transverse to l_1 , measured center-to-center of supports

= width of interior design frame (transverse to l_1), measured from center line to center line of adjacent slab panels (ACI 318-89, Section 13.6.2.3)

= width of exterior design frame measured

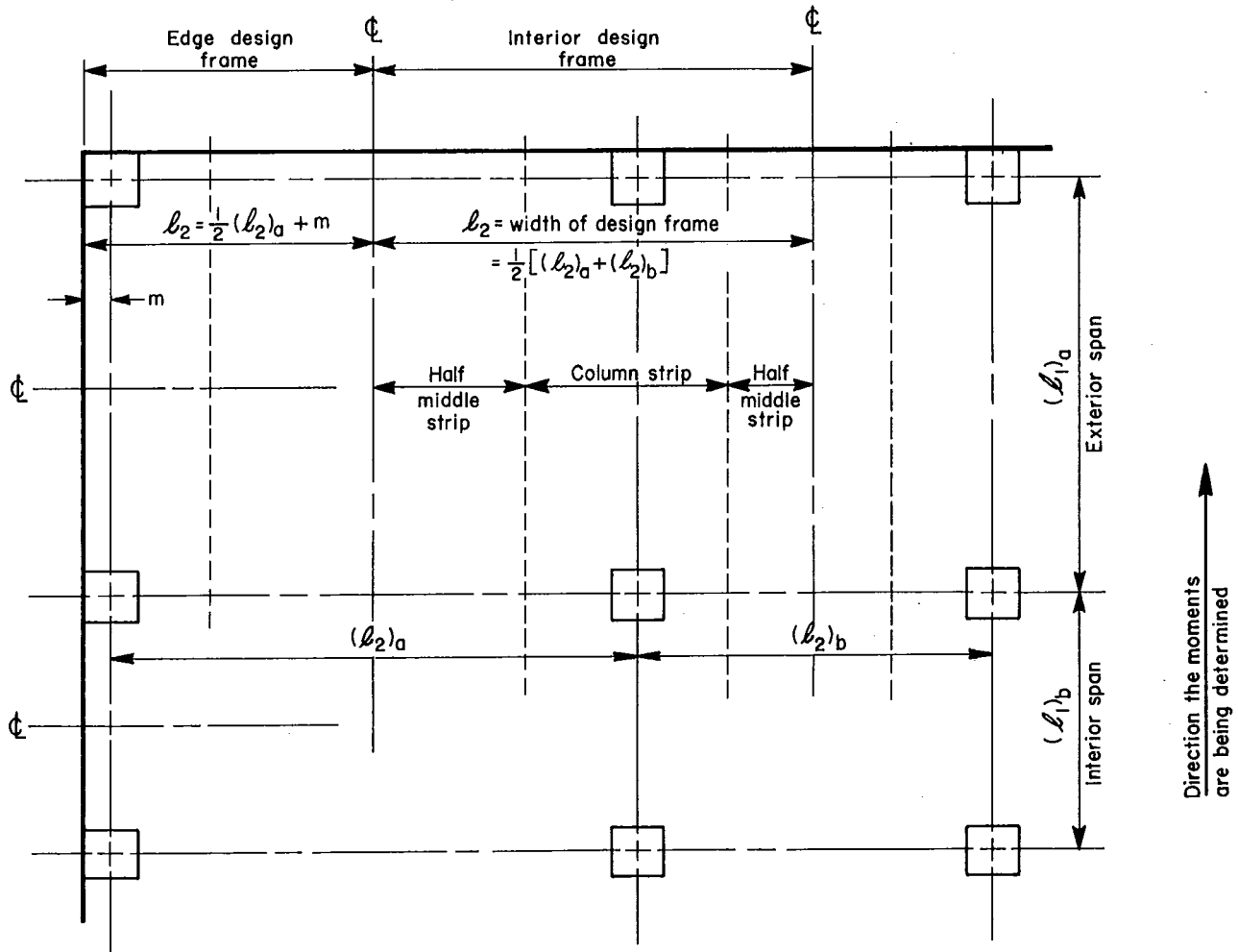


Fig. 1—Notation for slabs

from center line of edge panel to exterior face of support (ACI 318-89, Section 13.6.2.4)

m = distance from exterior face of edge panel to center of exterior column

M = fixed-end moment coefficient (Equivalent Frame Method)

$-M_i$ = factored negative moment at interior column (except first interior column), kip-ft

$+M_i$ = factored positive moment in interior span, kip-ft

$-M_e$ = factored negative moment at exterior support, kip-ft

$-M_{ie}$ = factored negative moment at first interior support, kip-ft

M_l = reinforcement spacing grid index for crack control

$+M_m$ = factored positive moment at midspan of exterior span, kip-ft

M_n = nominal moment strength at section, kip-ft

M_o = total factored static moment in a span, kip-ft

M_u = factored moment at a section, kip-ft

M_1 = unbalanced moment at support, in direction of span for which moments are being determined

M_2 = unbalanced moment perpendicular to M_1

s_1 = reinforcement spacing measured in direction of span for which moments and crack control are being analyzed, in. (Crack Control)

s_2 = reinforcement spacing measured perpendicular to spanwise direction in which moment analysis or crack control analysis is being made, in. (Crack Control)

u = beam width factor in ratio u/h_s , used in calculation of α_f ; $u = b$ for interior beam; $u = 2b$ for edge beam

v_n = nominal shear stress = $V_u/(\phi bd)$, psi

v_u = factored shear stress = $V_u/(bd)$, psi

V_c = nominal shear strength provided by concrete

V_u = factored perimeter shear force on critical shear section

V_w = factored shear force caused by wall supported by slab

w = pattern loading unit weight, psf

= width of crack

w_c = unit weight of concrete, pcf

w_d = dead load per unit area, psf (includes service dead load plus weight of slab unless otherwise defined)

w_l = live load per unit area, psf

w_{max} = maximum tolerable crack width for type of exposure, in. (Crack Control)

w_{mech} = mechanical load per unit area, psf

w_s = superimposed dead load, psf (total dead

load not including self weight of slab)

w_u = factored load per unit area, psf; = (typically) $1.4 w_d + 1.7 w_l$

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

$\bar{x}$ = distance between centroid of column and centroid of shear section, in.

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

α = relative beam stiffness; ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by center lines of adjacent panels (if any) on each side of beam.

= $(E_{cb}I_b)/(E_{cs}I_s)$

α_c = relative column stiffness; ratio of flexural stiffness of columns above and below the slab to combined flexural stiffness of the slabs and beams at a joint (stiffness taken in the direction of the span for which moments are being determined)

= $\Sigma K_c / \Sigma (K_s + K_b)$

α_f = shape coefficient which relates I of T-beam section to I of rectangular beam section; used in calculation of relative beam stiffness α

α_{min} = minimum α_c to satisfy ACI 318-89, Section 13.6.10(a)

α_s = constant used to compute V_c in nonprestressed slabs

α_{min} = minimum α_c to satisfy ACI 318-83, Section 13.6.10(a)

α_{ec} = relative stiffness of equivalent column, ratio of flexural stiffness of equivalent column to combined flexural stiffness of the slabs and beams at a joint (stiffness taken in the direction of the span for which moments are being determined)

= $K_{ec} / \Sigma (K_s + K_b)$

α_m = average value of α for all beams on edges of a slab panel

α_1 = α in direction of l_1

α_2 = α in direction of l_2

β = ratio of long to short clear spans (l_n) of a slab panel

β_a = ratio of dead load per unit area to live load per unit area (in each case without load factors)

β_c = ratio of long side to short side of concentrated load or reaction area

β_l = ratio of long to short center-to-center spans (l_1/l_s)

β_t = ratio of torsional stiffness of edge beam to flexural stiffness of a width of slab equal to span length of beam, center-to-center of supports

= $(E_{cb}C)/(2E_{cs}I_s)$

γ = factor for calculating slab thickness re-

quired by deflection; = the value of the denominator of Eq. (9-11), (9-12), or (9-13) divided by 1000.

γ_f = fraction of unbalanced moment transferred to column by flexure, Eq. (13-1)

γ_v = fraction of unbalanced moment transferred by eccentricity of shear at slab-column connection; = $1 - \gamma_f$; see ACI 318-89, Section 11.12.6.1

δ_s = positive moment increasing pattern-loading factor defined by Eq. (13-5); see ACI 318-89, Section 13.6.10(b)

λ = factor relating concrete strength f'_c to shear strength or to splitting tensile strength f_{ct} of lightweight concrete

ϕ = strength reduction factor

ρ = ratio of steel cross section area A_s to effective area bd of concrete

ρ_{tl} = active steel ratio = $A_{s1}/(24d_c)$ (Crack Control)

μ_e ,

μ_{ie} ,

μ_m = factors used in distribution of moment in an exterior span

ω = coefficient indicating relative strength of reinforcement and concrete in a member
= $\rho f_y / f'_c$

DESIGN PROCEDURES AND EXAMPLES

SLABS DESIGN PROCEDURE 1 for two-way slabs *without beams*,* designed by ACI 318-89 Direct Design Method

*Beams permitted at discontinuous edges.

For application of this Design Procedure, see Slabs Example 1.

ACI 318-89 REFERENCE	DESCRIPTION	HANDBOOK REFERENCE
13.6.1	Step 1 —Determine whether slab geometry and loading satisfy conditions for use of Direct Design Method:	
13.6.1.1	A) Minimum of three continuous spans in each direction.	
13.6.1.2	B) Panels shall be rectangular with ratio of longer to shorter span within a panel not greater than 2.0.	
13.6.1.3	C) Successive span lengths in each direction must not differ by more than one-third of longer span.	
13.6.1.4	D) No column offset more than 10 percent of span in the direction of offset.	
13.6.1.5	E) All loads are due to gravity and the service live load is not greater than three times total dead load. (Estimate dead load, including weight of slab.)	
	Step 2 —Select slab thickness The slab thickness chosen must satisfy both deflection and shear requirements. Such calculations require a knowledge of the support dimensions. If these dimensions are unknown, reasonable values for c_1 and c_2 are for columns $0.08 l_a$ to $0.15 l_a$ for capitals (if used) $0.15 l_a$ to $0.25 l_a$ when l_a is the average of l_1 and l_2 .	
9.5.3.1	A) Deflection requirements Locate the panel requiring the greatest thickness. (If spans are equal, the critical panel will usually be a corner panel since it has the least edge restraint. If spans are not equal, the critical panel may be the panel having the longest span.)	
9.5.3.2	Determine minimum slab thickness according to SLABS 1.1.	SLABS 1.1
	Note: Minimum slab thickness for slabs without interior beams may also be determined by using Eq. (9-11) through (9-13) in Section 9.5.3.3 and the applicable modification factors in Sections 9.5.3.4 and 9.5.3.5. However, this method requires more calculations and is therefore not used for slabs without interior beams in this volume. Slab thickness less than that required by SLABS 1.1 may be used if it can be shown by computation that deflection—including additional long-time deflection computed in accordance with Section 9.5.2.5—will not exceed the limits stipulated in Table 9.5(c) of Section 9.5.3.6, which table is SLABS 1.2 in this volume.	
11.12.1.2	B) Shear requirements Calculate the perimeter shear at the critical section of each support. The critical section is located at a distance of $d/2$ from the face of the support. <i>Note 1</i> —When choosing slab thickness from shear requirements, sufficient increase of calculated thickness is required to provide for moment-shear transfer at columns, particularly exterior columns. A final calculation for the moment-shear transfer is made in Step 9 after the moments to be transferred have been evaluated. <i>Note 2</i> —The beam action shear specified in Section 11.12.1.1. will not be critical for slabs without beams meeting the requirements of the Direct Design Method.	
	C) Commonly, the thickness determined for the critical panel is used for the slab thickness of all panels; hence it is essential that sufficient panels be checked to insure that the critical panel has been determined.	
13.6.2	Step 3 —Divide structure into design frames bounded by center lines of panels on each side of a line of columns. These frames constitute design strips which for interior frames have a width equal to the average of the l_2 on each side, and for exterior frames have a width equal to one-half exterior panel width plus distance from center line of exterior column to exterior edge.	

CODE REFERENCE	DESCRIPTION	HANDBOOK REFERENCE																									
Eq. (13-3)	Step 4—Compute total factored static moment M_o for the slab in each span within each frame. $\text{where } M_o = \frac{w_u l_2 l_n^2}{8}$																										
9.2	<i>Note 1</i>—w_u is factored load per unit area; = sum of factored service live load and factored dead load, which included service dead load plus slab weight.																										
13.6.2.3																											
13.6.2.5	<i>Note 2</i>—l_2 is taken as average of adjacent spans transverse to l_1. Clear span l_n is defined in Section 13.6.2.5 and may not be taken less than $0.65l_1$.																										
13.6.3.3	Step 5—Compute distribution of negative and positive moments to exterior and interior columns and spans. A) Exterior span moments (see sketch below) $-M_{ie} = \mu_{ie} M_o$ at interior support $+M_m = \mu_m M_o$ at midspan $-M_e = \mu_e M_o$ at exterior support <table><tr><th></th><th>Exterior edge unrestrained</th><th>Exterior edge fully restrained</th><th colspan="2">Other edge restraint</th></tr><tr><th></th><th></th><th></th><th>Without edge beam</th><th>With edge beam</th></tr><tr><td>μ_{ie}</td><td>0.75</td><td>0.65</td><td>0.70</td><td>0.70</td></tr><tr><td>μ_m</td><td>0.63</td><td>0.35</td><td>0.52</td><td>0.50</td></tr><tr><td>μ_e</td><td>0.00</td><td>0.65</td><td>0.26</td><td>0.30</td></tr></table> NEGATIVE AND POSITIVE DESIGN MOMENTS *For slabs with edge beam, the factors 0.26 and 0.52 are as shown in the right-hand column of the table above.		Exterior edge unrestrained	Exterior edge fully restrained	Other edge restraint					Without edge beam	With edge beam	μ_{ie}	0.75	0.65	0.70	0.70	μ_m	0.63	0.35	0.52	0.50	μ_e	0.00	0.65	0.26	0.30	
	Exterior edge unrestrained	Exterior edge fully restrained	Other edge restraint																								
			Without edge beam	With edge beam																							
μ_{ie}	0.75	0.65	0.70	0.70																							
μ_m	0.63	0.35	0.52	0.50																							
μ_e	0.00	0.65	0.26	0.30																							
13.6.3.2	B) Interior span moments $-M_i = 0.65 M_o$ $+M_i = 0.35 M_o$ <i>Note</i>—Distributed factored moments may be modified by 10 percent provided that the sum of the positive and the average negative moment in each span is not less than the total factored static moment M_o for that span.																										

CODE REFERENCE	DESCRIPTION	HANDBOOK REFERENCE
13.6.10 Table 13.6.10 Eq. (13-5)	Step 6—Consider whether interior columns are adequate for effects of pattern loading and, if necessary, increase design positive bending moments in panels. Compute $\beta_a = \frac{\text{unit unfactored dead load}}{\text{unit unfactored live load}}$ If $\beta_a \geq 2.0$, pattern loadings do not govern design; proceed to Step 7. If $\beta_a < 2.0$, compute $\alpha_c = \frac{\sum K_c}{\sum K_s}$ and compare α_c with α_{min} in Table 13.6.10 of ACI 318-83. If $\alpha_c \geq \alpha_{min}$, proceed to Step 7. If $\alpha_c < \alpha_{min}$, multiply positive bending moments obtained in Step 5 in spans supported by this column, by δ_s $\text{where } \delta_s = 1 + \frac{2 - \beta_a}{4 + \beta_a} \left(1 - \frac{\alpha_c}{\alpha_{min}} \right)$ <i>Note</i>—If $\alpha_c < \alpha_{min}$, in lieu of increasing positive moments in the spans supported by the column for which α_c was computed, the column dimensions may be increased such that $\alpha_c \geq \alpha_{min}$.	
13.6.4 13.2.1 13.2.2 13.6.4.2	Step 7—Distribute factored design frame moments computed in Step 5 (including modifications from Step 6 if any) to column and middle strips. For structures having equal spans l_1 and equal spans l_2, Column strip width: $0.5 l_s$ Half middle strip width: $0.5 [l_2 - 0.5 l_s]$ For irregular slabs, the center line of the columns will not be the center of the column strip. For that case, the width of the portion of the column strip on each side of the column center line must be determined separately, and the "half" middle strip on each side will be the remainder of the distance to the panel center line lying outside the column strip. A) Exterior negative moment Distribution depends on presence of torsionally stiff transverse edge beam (relative to flexural stiffness of slab) as measured by β_t $\text{where } \beta_t = \frac{E_b C}{2E_s I_s}$ If $\beta_t = 0$ (no edge beam) to column strip, 100 percent to middle strip, 0 percent If $\beta_t \geq 2.5$ (edge beam with extreme torsional stiffness) to column strip, 75 percent to middle strip, 25 percent Interpolate when $0 \leq \beta_t \leq 2.5$ <i>Note 1</i>—When exterior support consists of a wall or columns extending along the edge for a distance greater than three-fourths of the length, the exterior negative moment may be distributed uniformly across the panel. <i>Note 2</i>—The 75-25 distribution percentages in Steps 7A and 7B and the 60-40 percentages in Step 7C assume that $\alpha_1 = 0$ (i.e., there is no longitudinal beam within the design strip). For an edge panel with an edge beam in the same direction as l_1 and $\alpha_1 l_2 / l_1 > 0$, distribution percentages must be interpolated from the tables in Section 13.6.4 of ACI 318-89.	DESIGN Procedure 1, Step 10A
13.6.4.1	B) Interior negative moments (Use the larger negative moment from either side of the column unless a moment distribution is performed.) With no longitudinal beam in design strip (i.e., $\alpha_1 = 0$), to column strip, 75 percent to middle strip, 25 percent	
13.6.4.4	C) Positive moments With no longitudinal beam in design strip (i.e., $\alpha_1 = 0$), to column strip, 60 percent to middle strip, 40 percent	

CODE REFERENCE	DESCRIPTION	HANDBOOK REFERENCE
	Step 8 —Repeat Steps 4 to 7 for all other column lines, including exterior column lines.	
13.6.3 13.6.9.2 Eq. (13-4) 11.11 11.12.6.1 Eq. (11-42)	Step 9—Determine whether trial slab thickness chosen is adequate for moment-shear transfer. A) Compute moments to be transferred to interior columns as the greater of —Unbalanced interior negative panel moments from Step 5 —From pattern loading of dead plus one-half live load on longer of adjacent spans and dead load only on shorter span as given by $M = 0.07[(w_d + 0.5w_l)l_2l_n^2 - w_d'l_2'(l_n')^2]$ B) Compute portion of moment for all columns that is to be considered transferred to the slab by eccentricity of the shear, and the properties of the critical shear section. <i>Note</i>—Step 9B is satisfied automatically if the Design Aids in this handbook are used. B1) Compute portion of moment considered to be transferred by shear: $\gamma_v = 1 - \gamma_f = 1 - \frac{1}{1 + (2/3)\sqrt{b_1/b_2}}$ where $b_1 = (c_1 + d)$ is the length of the critical shear section in the direction of the moment and $b_2 = (c_2 + d)$ is the length of the critical shear section about which the moment acts in interior columns. In practice, for edge columns the dimension $(c_1 + d)$ is taken as $(c_1 = d/2)$, and for corner columns, the two dimensions are taken as $(c_1 + d/2)$ and $(c_2 + d/2)$. B2) Compute properties of critical shear section at distance $d/2$ from face of column or capital. (Expressions for the required properties J and A are given in ACI 318-89, Section 11.12.6.2 for interior columns only.)	
13.6.3.6 13.3.3.2 13.3.3.3	C) If $-M_e$ was determined using the moment coefficient $\mu_e = 0.26$, compute the full column strip nominal moment strength M_n provided. The total reinforcement provided in the column strip includes the additional reinforcement concentrated over the column to resist the fraction of the unbalanced moment $\gamma_f(0.26 M_o)$ transferred by flexure.	
11.11 Eq. (11-1) Eq. (11-2) Eq. (11-36) or 11.12.2	D) Compute value of v_n (ACI 318R-89, Section 11.12.6.2)	
	$v_n = \frac{V_u}{\phi A_c} + \frac{\gamma_v M_1 c_{A1}}{\phi J_1} + \frac{\gamma_v M_2 c_{A2}}{\phi J_2}$ $v_n = k_1 V_u + k_2 M_1 + k_3 M_2$	SLABS 9
13.6.3.6	Anytime $-M_e$ is determined using the moment coefficient $\mu_e = 0.26$, the full nominal moment strength M_n provided in the column strip must be used in the above equations. Otherwise the factored moment obtained directly from exact slab analysis should be used.	
	The required nominal shear strength V_n should not exceed the available nominal shear strength V_c .	
11.12.2.1	If no special shear reinforcement is provided, the maximum allowable nominal shear strength V_c of the section is the smallest of the values from ACI 318-89 Eq. (11-36), Eq. (11-37), and Eq. (11-38):	
	$V_c = \left(2 + \frac{4}{\beta_c}\right) \sqrt{f'_c} b_o d$ Eq. (11-36)	
	where	
	β_c is the ratio of the long side to the short side of the reaction areas	
	and	

CODE REFERENCE	DESCRIPTION	HANDBOOK REFERENCE
	b_o is the perimeter of the critical section $V_c = \left(\frac{\alpha_s d}{b_o} + 2 \right) \sqrt{f'_c} b_o d \quad \text{Eq. (11-37)}$ where α_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns $V_c = 4\sqrt{f'_c} b_o d \quad \text{Eq. (11-38)}$ It is clear from Eq. (11-38) that the shear strength provided by the plain concrete must not exceed $4\sqrt{f'_c}$, which is almost double the shear strength allowed in one-way members such as beams and one-way slabs. If special shear reinforcement is provided, the maximum nominal shear strength V_n must not exceed $6\sqrt{f'_c} b_o d$, provided that the value used for V_c to determine $V_s = V_n - V_c$ does not exceed $2\sqrt{f'_c} b_o d$. If the nominal shear strength V_n does exceed the available shear strength V_c, take one or more of the following actions: —Increase slab thickness —Increase column dimensions —Add column capital —Add shear reinforcing —Add drop panels.	
13.2.1 13.2.2 13.4 13.3.3.2 13.6.7 7.12 13.4 12.0	Step 10—Choose slab reinforcement. A) Compute widths of column strips and middle strips: A1) Width of half column strip on either side of column center line $0.25 l_2$ but not greater than $0.25 l_1$. A2) Width of half middle strip on each side is that portion of the design frame width l_2 not included in column strip. B) Compute the area of flexural reinforcement required to resist the factored moments from Step 7. <i>Note 1</i>—Reinforcement for moment considered to be transferred by flexure must be placed in a width not exceeding the column width plus one and one-half times the slab thicknesses on each side of the column. This requirement frequently controls placement of flexural reinforcement at exterior columns. <i>Note 2</i>—Distributed factored moments may be modified by 10 percent provided that the sum of the positive and the average negative moment in each span is not less than the total factored static moment M_o for that span. C) Choose bar diameters and spacing, considering: —Shrinkage and temperature stresses —Crack control —Bar development requirements	
	Step 11—Design edge beams where specified. See examples of beam design in Design Handbook Volume 1. <i>Note</i>—Torsional stresses in spandrel beam must be compatible with distribution used in design of slabs.	

SLABS EXAMPLE 1—Two-way slab *without beams*, designed according to Slabs Design Procedure I

Design the two-way slab without beams shown in the sketch by the Direct Design Method.

Given: Loading

- Live load, 125 psf
- Mechanical load, 15 psf
- Exterior wall, 400 plf

Materials

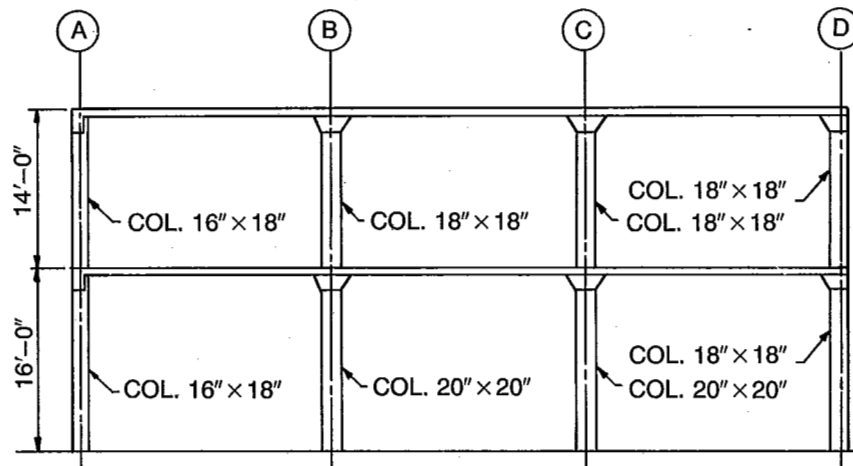
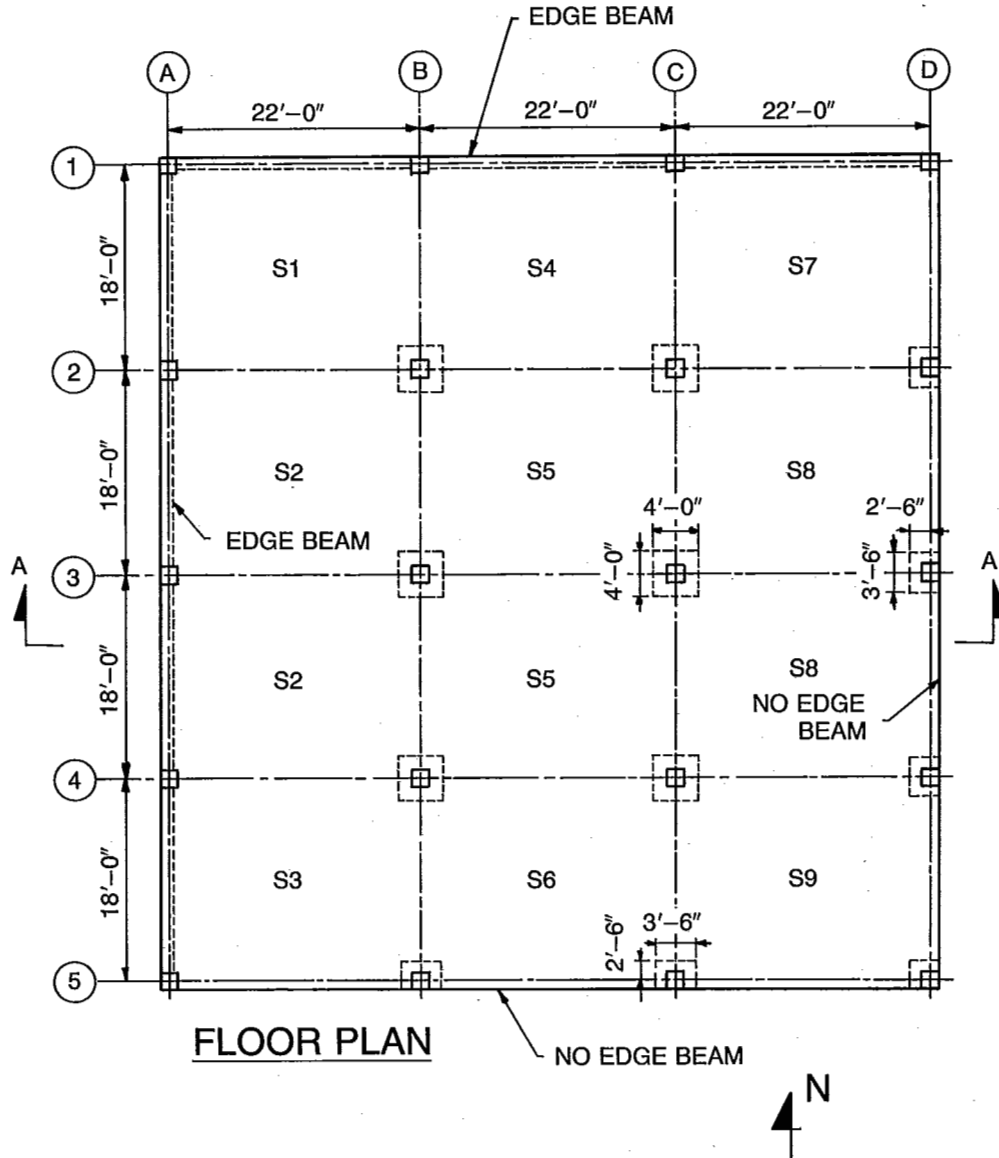
- Concrete strength, 3000 psi for slab and columns
- Yield strength of reinforcement, 40,000 psi
- Weight of concrete, 150 pcf

Design conditions

- Interior columns below slab, 20×20 in.
- Interior columns above slab, 18×18 in.
- Corner column A1, 16×16 in.; other exterior columns where edge beams are used, 16×18 in., with longer dimension parallel to the edge of slab.
- Exterior columns without beams, 18×18 in.
- Column capitals may be used.
- Edge beams extending not more than 16 in. below the soffit of the slab are permitted along building Lines (A) and (I). No beam is provided along Lines (5) and (D).
- Floor to floor height below slab, 16 ft 0 in.
- Floor to floor height above slab, 14 ft 0 in.

Note—To facilitate comparison of slabs design by Procedures I and III, loading, materials, and design conditions are the same for Slabs Examples 1 and 3.

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
	Step 1—Determine whether slab geometry and loading satisfy conditions for use of Direct Design Method.		
13.6.1.1		A) At least three continuous spans in each direction	
13.6.1.2		B) Panels are rectangular, and ratio of longer span to shorter span is $22.0/18.0 = 1.22 < 2.0$	
13.6.1.3		C) Successive span lengths in each direction do not differ	
13.6.1.4		D) Columns are not offset	
13.6.1.5	E) To verify that w_l does not exceed $3w_d$, estimate minimum likely w_d on basis of minimum allowable slab thickness of 5 in., and compare unfactored w_l and unfactored $3w_d$	E) $w_d = w_{mech} + \frac{h}{12} w_c$ $= 15 + \frac{5}{12} (150)$ $= 77.5 \text{ psf}$ $3w_d = 232.5$	
9.5.3.2			
13.6.1.5	<i>Note</i> —All loads considered are due to gravity only.	Given $w_l = 125 \text{ psf}$ $w_l < 3w_d$	
13.6.1.5		$\therefore$ Direct Design Method may be used.	



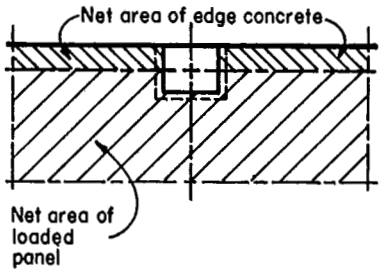
Two-way slab without beams

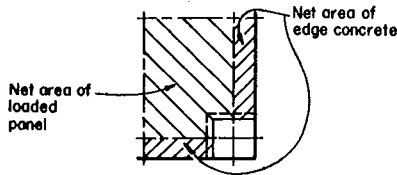
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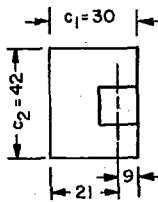
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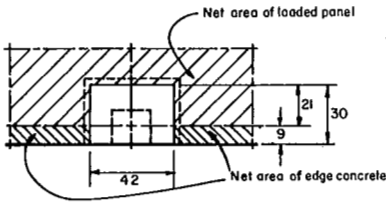
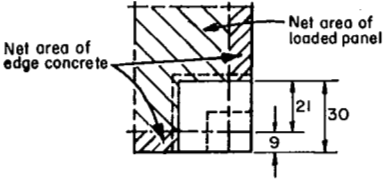
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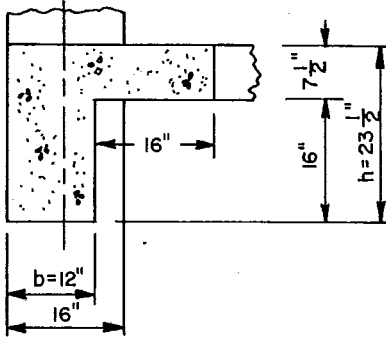
CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
9.0 13.6.2.5	Step 2—Select trial thickness on basis of deflection and shear requirements for critical panel. A) Consider Panel S9, compute trial slab thickness adequate for deflection requirements. Assume no column capitals.	A) For deflection (Panel S9): $l_{n(S)} = 22 \times 12 - (20/2) - (18/2) = 245 \text{ in.}$ $l_{n(D)} = 18 \times 12 - (20/2) - (18/2) = 197 \text{ in.}$ $h_{s(\min)} = \frac{245}{33} = 7.42 \text{ in.}$ Try $h_s = 7.5 \text{ in.}$	SLABS 1.1
9.5.3.1	<i>Note</i>—If no capitals are used and the successive spans in a given direction are equal, the critical panel for a constant-thickness slab is a corner panel with the least edge restraint. B) Check trial slab thickness for shear capacity. <i>Note</i>—At this stage, since moments are not known, only a preliminary check for shear can be made. <i>Note</i>—To provide some reserve shear capacity for transfer of unbalanced moments at column-slab connections, the depths required for shear alone must be increased. Suggested amounts for this increase are as follows: Interior columns 10 percent Exterior columns 40 percent Corner columns 70 percent Cover to centroid of steel layers assumed to be 1.25 in. (i.e., 3/4 in. cover with #4 bars). Perimeter shear load = V_u = total factored unit loading $w_u \times$ net slab area. B1) For perimeter shear load at interior column, net slab area = span length $\times$ design frame width less area of one column section. $V_u = (l_1 \times l_2 - c_1 \times c_2) w_u$	B) For $h_s = 7.5 \text{ in.}$ $w_d = \left(15 + \frac{7.5}{12} \times 150 \right) 1.4 = 21 + 131 = 152 \text{ psf}$ $w_l = 125 \times 1.7 = 213 \text{ psf}$ $w_u = 365 \text{ psf} = 0.365 \text{ ksf}$ B1) For interior column $V_u = \left(22 \times 18 - \left(\frac{20}{12} \right)^2 \right) \times 0.365 = 143.5 \text{ kips}$ $2(c_1 + c_2) = 2(20 + 20) = 80 \text{ in.}$ Entering chart at $f'_c = 3000 \text{ psi}$, $V_u = 143.5 \text{ kips}$, and $2(c_1 + c_2) = 80 \text{ in.}$, read $d = 7.2 \text{ in.}$ Allowing 10% for unbalanced moment: $d = 1.1 \times 7.2 = 7.9 \text{ in.}$	SLABS 9.1

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
		Checking whether values from chart can be used without modification: $\beta_c = 20 / 20 = 1 < 2$ and $b_o/d = \frac{2(c_1 + c_2) + 4d}{d}$ $= \frac{80 + 4(7.9)}{7.9} = 14 < 20$ $\therefore$ chart can be used without modification; use $d = 7.9$ in. $h_{s(s)} = 7.9 + 1.25 = 9.2 \text{ in.} > 7.5 \text{ in.}$ indicating trial slab thickness of 7.5 in. is not adequate in shear.	
	B2) Exterior column shear is calculated for load on net half-panel area plus weight of edge concrete and upper exterior wall with dead load factor. 	B2) For exterior edge column $V_u = \left(\frac{22 \times 18}{2} - \frac{9 \times 18}{12 \times 12} \right) \times 0.3652$ $+ \frac{9}{12} (22 - 1.5) \times 0.131$ $+ (22 - 1.5) \times 0.400 \times 1.4 = 85.3 \text{ kips}$ $2c_1 + c_2 = (2 \times 18) + 18 = 54 \text{ in.}$ Entering the chart at $f'_c = 3000$ psi, $V_u = 85.3$ kips, and $2c_1 + c_2 = 54$ in., read $d = 6.8$ in. Allowing 40% for unbalanced moment: $d = 1.4 \times 6.8 = 9.5 \text{ in.}$ Checking whether values from chart can be used without modification: $\beta_c = 18 / 18 = 1 < 2$ and $b_o/d = \frac{2(c_1 + c_2) + 2d}{d}$ $= \frac{54 + 2(9.5)}{9.5} = 7.7 < 15$ $\therefore$ chart can be used without modification; use $d = 9.5$ in. $h_{s(s)} = 9.5 + 1.25 = 10.8 \text{ in.} > 7.5 \text{ in.}$ indicating trial slab thickness of 7.5 in. is not adequate in shear.	SLABS 9.4

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
	B3) Calculation of shear at a corner column is included here as an example although this would not be necessary since it is already obvious, from the interior column and edge column calculations, that thickness required for shear greatly exceeds that required for deflection—and therefore column capitals should be used. Corner column shear is calculated for load on net quarter-panel area plus weight of edge concrete and exterior wall on two edges with dead load factor. 	B3) For corner column $V_u = \left(\frac{22 \times 18}{4} - \left(\frac{9}{12} \right)^2 \right) \times 0.365$ $+ \left[\frac{9}{12} \left(\frac{22}{2} - \frac{9}{12} \right) + \frac{9}{12} \left(\frac{18}{2} - \frac{9}{12} \right) \right] \times 0.131$ $+ \left(\frac{22 + 18}{2} - \frac{18}{12} \right) \times 0.400 \times 1.4$ $= 48.7 \text{ kips}$ $c_1 + c_2 = 18 + 18 = 36 \text{ in.}$ Entering the chart at $f'_c = 3000 \text{ psi}$, $V_u = 48.7 \text{ kips}$, and $c_1 + c_2 = 36$ read $d = 6.3 \text{ in.}$ Allowing 70% for unbalanced moment: $d = 1.7 \times 6.3 = 10.7 \text{ in.}$ Checking whether values from chart can be used without modification: $\beta_c = 18 / 18 = 1 < 2$ and $b_o/d = \frac{(c_1 + c_2) + d}{d}$ $= \frac{36 + 10.7}{10.7} = 4.4 < 10$ $\therefore$ chart can be used without modification; use $d = 10.7 \text{ in.}$ $h_{s(s)} = 10.7 + 1.25 = 11.9 \text{ in.} > 7.5 \text{ in.}$ indicating trial slab thickness of 7.5 in. is not adequate in shear. At all columns, thickness of the slab required for shear is greater than that required for deflection. $\therefore$ Use column capitals C) Select size of column capitals. Design Procedure says reasonable assumption for interior capitals is $0.15l_a \leq c \leq 0.25l_a$ where l_a is average of longer and shorter spans.	SLABS 9.8
		C) Estimate capital size. For interior columns, capital size may be $0.15l_a = 0.15 \left(\frac{18 + 22}{2} \right) = 3.0 \text{ ft}$ $0.25l_a = 0.25 \left(\frac{18 + 22}{2} \right) = 5.0 \text{ ft}$	

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
	<i>Note 1</i>—Considerable freedom is given to the designer in selecting capital dimensions. Designer should consider dimensions easily formed with plywood or use of standard forms, as well as shear requirements. <i>Note 2</i>—No capitals are used for the exterior columns in Line (A) and Line (1) since there are beams along these lines.	Choosing increments of 6 in., select square interior capitals: $c_1 = c_2 = 4 \text{ ft } 0 \text{ in.}$ For edge columns, select capital size: 2 ft 6 in. perpendicular to edge 3 ft 6 in. parallel to edge. (In SLABS 9.4, dimension perpendicular to discontinuous edge is labeled c_1 and parallel dimension c_2. Therefore, $c_1 = 2 \text{ ft } 6 \text{ in.}$ and $c_2 = 3 \text{ ft } 6 \text{ in.}$)  For corner column D5, select square capitals: $c_1 = c_2 = 2 \text{ ft } 6 \text{ in.}$	
	Repeat Step 2A and 2B with capitals. A') Consider Panel S9 for deflection requirements after capitals added. No edge beam $\therefore h_{s(d)} = \frac{l}{33}$ B1') Recheck slab thickness required by shear with column capitals. Obtain value of d required to provide shear capacity and compare with d provided. Compute b_o/d based on trial dimensions. If $b_o/d > 20$ modify k_1. Increase by 10% for allowance for unbalanced moment.	A') For deflection (Panel S9) $l_{n(s)} = 22 \times 12 - 21 - 21 = 222 \text{ in.}$ $l_{n(D)} = 18 \times 12 - 21 - 21 = 174 \text{ in.}$ $h_{s(d)} = \frac{222}{33} = 6.7 \text{ in.}$ Try 7.5 in. B1') For interior column with capital $V_u = \left[22 \times 18 - \left(\frac{48}{12} \right)^2 \right] \times 0.365$ $= 138.7 \text{ kips}$ $2(c_1 + c_2) = 2(48 + 48) = 192 \text{ in.}$ For h 7.5 in., $d = 7.5 - 1.25 = 6.25 \text{ in.}$ $b_o/d = 4(48 + 6.25)/6.25 = 217/6.25$ $= 34.7 > 20;$ Modification of k_1 required. Enter SLABS 9.1 at $f'_c = 3000 \text{ psi}$, $V_u = 138.7^k$, and read $k_1 = 1.60$. Mod. $k_1 = 1.60(10 \times 6.25/217 \text{ to } .5)$ $= 1.26$ Enter SLABS 9.1 with k_1 1.26, $2(c_1 + c_2) = 192$ and read $d = 4.5 \text{ in.}$ Increase d for unbalanced moment, $d = 1.10 \times 4.5 = 4.95 \text{ in.}$ $d < d(\text{provided}) = 6.25 \text{ in.}$ O.K.	SLABS 9.1

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
	 Net area of loaded panel Net area of edge concrete Compute b_o/d based on trial dimensions. If $b_o/d > 15$, modify k_1. Increase d by 40% for allowance for unbalanced moment. Compare $d_{(req'd)}$ to $d_{(prov.)}$	B2') For exterior (edge) column with capital $V_u = \left[\frac{22 \times 18}{2} - \frac{21 \times 42}{12 \times 12} \right] \times 0.365$ $+ \frac{9}{12} (22 - 3.5) \times 0.131$ $+ (22 - 3.5) \times 0.400 \times 1.4$ $= 82.2 \text{ kips}$ $b_o/d = [2(30) + 42 + 2(6.25)] / 6.25$ $= 18.32 > 15$, modify k_1 Enter SLABS 9.4 at $f'_c = 3000$ psi $V_u = 82.2^k$, read $k_1 = 2.70$. Mod. $k_1 = 2.70 (7.5 / 18.32 \text{ to } .5)$ $= 2.46$ Enter SLABS 9.4 with $k_1 2.46$, $2(c_1 + c_2) = 102$, read $d = 4.2$ in. $d_{req} = 1.4 (4.2) = 5.9$ in. $< d$ (provided) O.K.	SLABS 9.4
	 Net area of edge concrete Net area of loaded panel Compute b_o/d based on trial dimensions. If $b_o/d > 10$, modify k_1. Increase d by 70% for unbalanced moment. Since deflection thickness requirement exceeds shear requirement with capitals, use thickness calculated for deflection requirements.	B3') For corner column with capital $V_u = \left[\frac{22 \times 18}{4} - \left(\frac{21}{12} \right)^2 \right] \times 0.365$ $+ \left[\frac{9}{12} \left(\frac{22}{2} - \frac{21}{12} \right) + \frac{9}{12} \left(\frac{18}{2} - \frac{21}{12} \right) \right]$ $\times 0.131$ $+ \left[\frac{22 \times 18}{2} - \frac{2 \times 21}{12} \right] \times 0.400 \times 1.4$ $= 46 \text{ kips}$ $c_1 + c_2 = 30 + 30 = 60$ in. $b_o/d = (30 + 30 + 6.25) / 6.25$ $= 10.6 > 10$ Enter SLABS 9.8 at $f'_c = 3000$ psi $V_u = 46^k$, read $k_1 = 4.8$. Mod. $k_1 = 4.8 (5 / 10.6 + 0.5) = 4.66$ Enter SLABS 9.8 with $k_1 4.66$, $c_1 + c_2 = 60$, read $d = 4.0$ $d_{req} = 1.7 \times 4.0 = 5.6$ in. $< d$ (provided) O.K. Use $h_{s(d)} = 7.5$ in. Confirm loading: $w_u = 213 + \left(15 + \frac{7.5}{12} \times 150 \right) \times 1.4$ $= 365 \text{ psf} = 0.365 \text{ ksf}$	SLABS 9.8

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
	D) Check adequacy of slab thickness for other panels, as in Steps 2A and 2B. <i>Note 1</i>—In slabs where not all spans are the same, it may be necessary to check several panels to determine which panel is critical for deflection. While Panel S2 is not critical by inspection for this slab, a check is made to illustrate the procedure when an edge beam is present.	D) Check Panel S2 $l_{n(A)} = 18 \times 12 - \frac{18}{2} - \frac{18}{2} = 198 \text{ in.}$ $l_{n(B)} = 18 \times 12 - \frac{48}{2} - \frac{48}{2} = 168 \text{ in.}$	
13.6.2.5	<i>Note 2</i>—Clear span l_n is measured from face of capital where there is a capital and from face of column where there is no capital [Lines (A) and (1)]—except where there are beams, in which case l_n is measured to the face of beams for the slab thickness formulas. Minimum l_n is $0.65 l_1$.	First it is necessary to calculate α for edge beam.	SLABS 1
13.2.4	D1 To obtain α for the beam, compute from the cross section of the beam and slab the values h/h_s, u/h_s, and l, and read α_f from SLABS 2; then find α from	D1) Compute α  $\alpha = \frac{E_{cb} b}{E_{cs} l} \left(\frac{h}{h_s} \right)^3 \alpha_f$ $h/h_s = 23.5/7.5 = 3.13$ $u/h_s = (2 \times 12)/7.5 = 3.2$ $l = 0.5l_2 + 0.5 (\text{column dimension})$ $= 0.5(22.0 \times 12) + 0.5(16) = 140 \text{ in.}$	SLABS 2
13.2.4	<i>Note 3</i>—In an edge beam, for the purpose of computing u/h_s, $u = 2b$. If concrete is the same in beam and slab, E_{cb} and E_{cs} are the same so that $\alpha = \frac{b}{l} \left(\frac{h}{h_s} \right)^3 \alpha_f$	Reading the graph for $h/h_s = 3.13$ and $u/h_s = 3.2$, find $\alpha_f = 1.46$ $\alpha = \frac{12}{140} \times 3.13^3 \times 1.46$ $= 3.84 > 0.80$ $h_{min.} = \frac{l_n}{36} = \frac{198}{36} = 5.50 \text{ in} < 7.5 \text{ in.}$	SLABS 2 SLABS 1.1

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
9.5.3.3	D3) Compute slab thickness for Panel S7 for deflection requirements. This panel contains an exterior edge [along line (D)] without edge beam, thus indicating a 10 percent thickness increase. The other exterior edge [along line (1)] does have the edge qualifying for a waiver of the 10 percent increase. For α, It is recommended that the 10 percent increase be applied for this case. The large value of α along one edge does not seem a sufficient requirement to neglect the effect of an edge having no beam at all.	D3) Check Panel S7. $l_{n(1)} = 22(12) - 18 = 246 \text{ in.}$ $u/h_s = 3.2; h/h_s = 3.13$ $l = \frac{18(12)}{2} + 8 = 116 \text{ in.}$ find $\alpha_r = 1.45$ $\alpha = \frac{12}{116} (3.13)^3 (1.45)$ $= 4.6 > 0.8$ $h_s = \frac{246}{33} = 7.45 \text{ in.}$	SLABS 2 SLABS 1.1
	Step 3—Divide the structure into design frames along the column lines.	Interior frames along Lines (B) and (C) have a frame width $l_2 = 22$ ft; interior frames along Lines (2), (3) and (4) have a frame width $l_2 = 18$ ft; exterior frames along Lines (A) and (D) have a frame width $l_2 = 11$ ft 8 in. and 11 ft 9 in., respectively (from center line of panel to exterior face of slab); exterior frames along Lines (1) and (5) have a frame width $l_2 = 9$ ft 8 in. and 9 ft 9 in., respectively (from center line of panel to exterior face of slab).	
13.6.2.1 13.6.2.2 Eq. (13-3)	Step 4—Compute total static moment M_o for each span in a design frame centered on one column line (and for similar frames centered on similar column lines). $\text{where } M_o = \frac{w_u l_2 l_n^2}{8}$	Step 4—Considering the design frame along Line (3), compute moments in east-west direction: For End Span A-B on column Line (3), $l_2 = 18.0 \text{ ft}$ $l_n = 22 - 0.67 - 2.0 = 19.33 \text{ ft}$ $M_o = (0.365)(18.0)(19.33)^2 / 8$ $= 307 \text{ kip-ft}$	

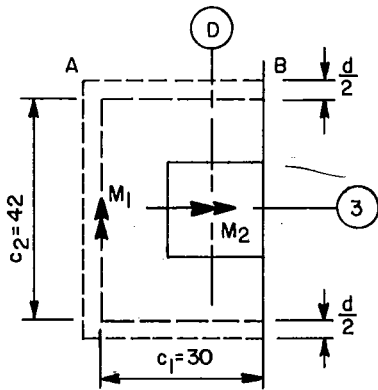
CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
	<i>Note</i>—Steps 4 through 7 are carried out for column Line (3); then Steps 4 through 7 are repeated as Step 8 for each other column line in north-south and east-west directions.	For Interior Span B-C on column Line (3), $l_2 = 18.0 \text{ ft}$ $l_n = 22 - 2.0 - 2.0 = 18.0 \text{ ft}$ $M_o = (0.365)(18.0)(18)^2/8$ $= 266 \text{ kip-ft}$ For End Span C-D on column Line (3), $l_2 = 18.0 \text{ ft}$ $l_n = 22 - 2.0 - 21/12 = 18.25 \text{ ft}$ $M_o = (0.365)(18.0)(18.25)^2/8$ $= 273 \text{ kip-ft}$	
	Step 5—Compute distribution of factored moments for east-west design frame centered on Column Line (3). <i>Note</i>—Because of symmetry of the design, the results will be the same for adjacent design frames centered on Lines (2) and (4). A similar procedure is used to provide moment distribution for north-south design frames centered on Lines (B) and (C).		
13.6.3.3	Moments for an <i>end</i> span Moment at exterior column $-M_e = 0.30M_o$ Positive moment $+M_e = 0.50M_o$ Moment at first interior column $-M_{ie} = 0.70M_o$	For design span centered on Line (3) dividing Panels S2, where $M_o = 307 \text{ kip-ft}$ (from Step 4): $-M_e = 0.30 \times 307$ $= 92 \text{ kip-ft}$ $+M_e = 0.50 \times 307$ $= 154 \text{ kip-ft}$ $-M_{ie} = 0.70 \times 307$ $= 215 \text{ kip-ft}$	
13.6.3.2	Moments for an <i>interior</i> span $-M = 0.65M_o$ $+M = 0.35M_o$	For design span centered on Line (3) dividing Panels S5, where $M_o = 266 \text{ kip-ft}$ (from Step 4): $-M = 0.65 \times 266$ $= 173 \text{ kip-ft}$ $+M = 0.35 \times 266$ $= 93 \text{ kip-ft}$	
13.6.3.3	Moments for the <i>end</i> span <i>Note</i>—These calculation results will be the same for design frames centered on lines (2) or (4).	For design span centered on Line (3) dividing Panels S8 where $M_o = 273 \text{ kip-ft}$ $-M_e = 0.26 \times 273$ $= 71 \text{ kip-ft}$ $+M_e = 0.52 \times 273$ $= 142 \text{ kip-ft}$ $-M_{ie} = 0.70 \times 273$ $= 191 \text{ kip-ft}$	

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
13.6.10	Step 6—Consider whether interior columns are adequate for effects of pattern loading and, if necessary, increase design positive bending moments in panels. A) Determine whether $\beta_a = \frac{w_d^*}{w_l^*}$ is greater or less than 2.0 and, if less, satisfy Sections 13.6.10(a) or 13.6.10(b). *unfactored B) To check whether columns satisfy Section 13.6.10(a), compute $\alpha_c = \frac{\Sigma K_c/E_c}{\Sigma K_s/E_c}$ and compare with α_{min}. B1) Compute $\Sigma \frac{K_c}{E_c} = \Sigma \frac{k_c I_c}{l_c} = \Sigma \frac{4I_c}{l_c}$ Note 1—For $\Sigma K_c/E_c$ to be in units of in.³, the length l_c must be in inch units. Note 2—When calculating column or slab flexural stiffness (K_c or K_s) in the Direct Design Method, the coefficient 4 is equivalent to the stiffness coefficients k_c and k_s used in the Equivalent Frame Method. The value 4 used in the Direct Design Method assumes uniform cross section of a column for the full length, l_c, or uniform cross section of a slab span for the full length, l_1. In the Equivalent Frame Method, values of k_c or k_s exceed 4 to account for the added stiffness of the capital and slab-column joint. The Direct Design Method simplification is valid as long as it is applied to both column and slab. B2) Compute $\Sigma K_s/E_c = 2K_s/E_c$ $K_s/E_c = \frac{k_s I_s}{l_1} = \frac{4I_s}{l_1}$ $I_s = \frac{l_2 h_s^3}{12}$ B3) $\alpha_c = \frac{\Sigma K_c/E_c}{\Sigma K_s/E_c}$ B4) Read α_{min}	A) $\beta_a = \frac{w_d^*}{w_l^*} = \frac{15 + (7.5/12)(150)}{125} = 0.87 < 2.0$ ∴ Check α_c against minimum required α_c, i.e., α_{min}. B1) For 20 × 20 in. column below slab: $I_c = \frac{20 \times 20^3}{12} = 13,333 \text{ in.}^4$ and for 18 × 18 in. column above slab: $I_c = \frac{18 \times 18^3}{12} = 8748 \text{ in.}^4$ $\Sigma K_c/E_c = \frac{4 \times 13,333}{16 \times 12} + \frac{4 \times 8748}{14 \times 12} = 486 \text{ in.}^3$ B2) For $h = 7.5$ in., $l_2 = 18$ ft, and $l_1 = 22$ ft, $I_s = \frac{7.5^3 (18 \times 12)}{12} = 7594 \text{ in.}^4$ $K_s/E_c = \frac{4 \times 7594}{22 \times 12} = 115 \text{ in.}^3$ $\Sigma K_s/E_c = 2K_s/E_c = 230 \text{ in.}^3$ B3) $\alpha_c = 486/230 = 2.11$ B4) Reading Table 13.6.10 for $\beta_a = 0.87$, $l_2/l_1 = 18/22 = 0.82$, and $\alpha = 0$, find $\alpha_{min} \approx 0.9$	GENERAL 2 GENERAL 2 GENERAL 1
13.6.10			

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
13.6.10	B5) Compare α_c and α_{min}	B5) $\alpha_c = 2.11 > \alpha_{min} \approx 0.9$ $\therefore$ Section 13.6.10(a) is satisfied and no modification of positive bending moments is required.	
13.6.4	Step 7—Distribute frame factored moments computed in Step 6 to column and middle strips.		
13.6.4.2	A) For exterior frames, find proportion of exterior negative factored moment to be apportioned to column strip from table in Section 13.6.4.2. Consider east-west design Frames (2), (3), (4) framing into exterior columns in Lines (A) and (D). <i>Note</i> —Distribution of frame moment at discontinuous edge depends on relative torsional stiffness of edge beam transverse to direction in which moments are determined as measured by β_t , the ratio of torsional stiffness of edge beam to flexural stiffness of slab strip. Also depends on relative flexural stiffness of beam parallel to direction in which moments are determined—as measured by α_1 , the ratio of beam flexural stiffness to slab flexural stiffness. A1) Column D3 Where there is no torsional edge beam, as in Lines (5) and (D), $\beta_t = 0$. Where there is no flexural beam, as in design frames along Lines (2), (3), (4), (5), and (B), (C), (D), $\alpha_1 = 0$. If $\beta_t = 0$, distribute factored negative moment 100 percent to column strip. If $\beta_t \geq 2.5$, and $\alpha_1 = 0$, distribute 75 percent of factored negative moment to column strip. If $0 \leq \beta_t \leq 2.5$, interpolate linearly If $\alpha_1 > 0$, interpolate linearly A2) Column A3	A) Design Frame (3) and Columns A3 and D3 A1) Column Line (D) has no edge beam $\beta_t = 0$ 100 percent of negative moment in column strip = 71 kip-ft. A2) Column Line (A) contains an edge beam. There is no flexural beam in the design strip in the direction in which moments are determined. $\therefore \alpha_1 = 0$ and $\alpha_1 l_2 = 0$ (See sketch in Step 2D1) for $x_1 = 12$ in. and $y_1 = 23.5$ in. $C_1 = 9181 \text{ in.}^4$ for $x_2 = 7.5$ in. and $y_2 = 16$ in. $C_2 = 1586 \text{ in.}^4$ $C = C_1 + C_2 = 9181 + 1586$ $= 10,767 \text{ in.}^4$	
13.7.5.3	Compute C for the edge beam in Column Line (A)		SLABS 3

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
	Compute I_s for slab strip cross section at Column A3	For $h_s = 7.5$ in., $l_2 = 18$ ft. and $l_1 = 22$ ft	
	$I_s = \frac{l_2 h_s^3}{12}$	$I_s = \frac{7.5^3(18 \times 12)}{12} = 7594 \text{ in.}^4$	
13.0	$\beta_t = \frac{E_{cb} C}{2E_{cs} I_s} = \frac{C}{2I_s}$	$\beta_t = \frac{C}{2I_s} = \frac{10,767}{2(7594)} = 0.709$	
13.6.4.2		Interpolating in the table of Section 13.6.4.2 between $\beta_t = 0$ and $\beta_t \geq 2.5$ for $\beta_t = 0.709$: Percent to column strip = $100\% - \frac{0.709}{2.5} (100\% - 75\%)$ = 93% of exterior negative moment - M_{ie} at Line (A) to be assigned to column strip: $= 0.93 \times 92 = 86 \text{ kip-ft}$ Moments assigned to each half middle strip equal $\frac{92 - 86}{2} = 3 \text{ kip-ft}$	
13.6.4.1	B) For exterior and interior panels, find proportion of <i>interior negative</i> design moment to be apportioned to column strip from table in Section 13.6.4.1	B) Reading the table in Section 13.6.4.1 for $\alpha_1 l_2 / l_1 = 0$, find 75% of - M_{ie} to be apportioned to column strip. For Column B3 (east-west) Column Strip Negative Moment $= 0.75 \times 215 = 161 \text{ kip-ft}$ For each half middle strip, $\text{Negative Moment} = \frac{215 - 161}{2} = 27 \text{ kip-ft}$ For Column C3 (east-west) Column Strip Negative Moment $= 0.75 \times 191 = 143 \text{ kip-ft}$ For each half middle strip, $\text{Negative Moment} = \frac{191 - 143}{2} = 24 \text{ kip-ft}$	
13.6.4.4	C) For exterior and interior panels, find proportion of <i>positive</i> design moment to be apportioned to column strip from Table in Section 13.6.4.4.	C) Reading the table in Section 13.6.4.4 for $\alpha_1 l_2 / l_1 = 0$, find 60% of + M to be apportioned to column strip. For Span A3-B3 (Panels S2) Column Strip Positive Moment $= 0.6 \times 154 = 92 \text{ kip-ft}$ Half Middle Strip Positive Moment $= 0.2 \times 154 = 31 \text{ kip-ft}$ For Span B3-C3 (Panels S5) Column Strip Positive Moment $= 0.6 \times 93 = 56 \text{ kip-ft}$	

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE																																							
		Half Middle Strip Positive Moment = 0.2 × 93 = 19 kip-ft For Span C3–D3 (Panels S8) Column Strip Positive Moment = 0.6 × 142 = 85 kip-ft Half Middle Strip Positive Moment = 0.2 × 142 = 28 kip-ft D) Summarize by span the distribution of factored moments to the column strip and the two half middle strips. For interior span, use the larger of the two negative moments at the column. D) Factored moment, kip-ft																																								
		<table><tr><th>Panel and moment</th><th>Distribution to column strip</th><th>Total to two half middle strips</th></tr><tr><td colspan="3">Panel S2</td></tr><tr><td>−M_e</td><td>93% (92) = 86</td><td>6</td></tr><tr><td>+M_e</td><td>60% (154) = 92</td><td>62</td></tr><tr><td>−M_{ie}</td><td>75% (215) = 161</td><td>54</td></tr><tr><td colspan="3">Panel S5</td></tr><tr><td>−M</td><td>75% (215) = 161</td><td>54</td></tr><tr><td>+M</td><td>60% (93) = 56</td><td>37</td></tr><tr><td>−M</td><td>75% (191) = 143</td><td>48</td></tr><tr><td colspan="3">Panel S8</td></tr><tr><td>−M_{ie}</td><td>75% (191) = 143</td><td>48</td></tr><tr><td>+M_e</td><td>60% (142) = 85</td><td>57</td></tr><tr><td>−M_e</td><td>100% (71) = 71</td><td>0</td></tr></table>	Panel and moment	Distribution to column strip	Total to two half middle strips	Panel S2			−M _e	93% (92) = 86	6	+M _e	60% (154) = 92	62	−M _{ie}	75% (215) = 161	54	Panel S5			−M	75% (215) = 161	54	+M	60% (93) = 56	37	−M	75% (191) = 143	48	Panel S8			−M _{ie}	75% (191) = 143	48	+M _e	60% (142) = 85	57	−M _e	100% (71) = 71	0	
Panel and moment	Distribution to column strip	Total to two half middle strips																																								
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−M _e	100% (71) = 71	0																																								
		<i>Note 1</i>—These calculation results will be the same for design frames centered on Lines (2) or (4). Distribution of north-south moments for columns in Line (5) will be 100 percent in column strips, as already shown for distribution of east-west moments to columns in Line (D). Distribution for columns in Line (1) will be calculated in the same manner as Column A3. Note that frames along Lines (A) and (1) have flexural edge beams so that α₁ > 0 when taking moments in direction along Line (A) or Line (1). <i>Note 2</i>—Minimum reinforcement steel is required in middle strips even when moment is distributed 100 percent to column strip.																																								
	Step 8—Repeat Steps 4 through 7 for all other column lines.	Step 8 omitted in this example.																																								
13.6.3 13.6.9.2	Step 9—Determine whether trial slab thickness chosen is adequate for moment-shear transfer. Consider all possible critical columns. A) For interior columns compute moments	A) Consideration of interior columns omitted in this example																																								

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
11.12.6.2	B) For all columns, compute portion of moment that is to be considered transferred to the slab by eccentricity of the shear, and the properties of the critical shear section. Compute dimensions for use in SLABS 9. SLABS 9.5 assumes $c_1 = c_2$; SLABS 9.6 corrects for $c_1 \neq c_2$	B) For Column D3  $d = h_s - \text{cover} - d_b$ $= 7.5 - 0.75 - 0.500 = 6.25 \text{ in.}$ $2c_1 + c_2 = 2 \times 30 + 42$ $= 102 \text{ in.}$ $c_1 - c_2 = 30 - 42 = -12 \text{ in.}$ To find k_1, enter the graph at $d = 6.25$; proceed horizontally to $2c_1 + c_2 = 102$, drop to k_1 scale, and read $k_1 = 1.65$ To find k_2, first find k_2' and then correct for $c_1 \neq c_2$. Find k_2' by entering SLABS 9.5 at $c_2 = 42 \text{ in.}$, proceeding horizontally to $d = 6.25 \text{ in.}$, dropping to the k_2' scale, and reading $k_2' = 0.42$. Then find k_2 by entering SLABS 9.6 at $d =$ 6.25 and $k_2' = 0.41$, proceeding horizontally to $c_1 - c_2 = -12 \text{ in.}$, dropping vertically to $d = 6.25$, and reading $k_2 = 0.56$.	SLABS 9.4 SLABS 9.5 SLABS 9.6
13.6.3.6	C) For moment transfer between slab and an edge column Section 13.6.3.6 requires that the fraction of unbalanced moment transferred by eccentricity of shear must be based on the full column strip nominal moment strength M_n provided. Determine reinforcement required for column strip negative moment. Estimate j at 0.95 and compute $A_s = \frac{M_u}{\phi f_y j d}$ Check validity of estimated value of j by $a = \frac{A_s f_y}{0.85 f'_c b}$	C) For Column D3 $M_u = -M_e = 71 \text{ kip-ft}$ $A_s = \frac{71(12 \text{ in./ft})(1000 \text{ lb/kip})}{0.90(40,000)(0.95)(6.25)}$ $= 3.99 \text{ in.}^2$ $a = \frac{3.99(40,000)}{0.85(3000)(108)} = 0.58 \text{ in.}$	FLEXURE 1 of Handbook Vol. 1

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
	and $j = 1 - \left(\frac{a}{2d}\right)$	$j = 1 - \frac{0.58}{2(6.25)} = 0.95$	FLEXURE 1 of Handbook Vol. 1
	Using #5 bars, for which $A_b = 0.31$, determine number of bars required	$\therefore$ assumed j is OK $\frac{3.99}{0.31} = 12.9$ bars $\therefore$ 13 bars required	
	Determine bar spacing	$\frac{108}{13} = 8.3$ in.	
13.4.2	Check whether this spacing exceeds twice slab thickness	$2h_s = 2(7.5) = 15$ in. 8.3 in. < 15 in., so spacing is OK. $\therefore$ try 13 #5 bars	
13.3.3.2	Determine unbalanced moment $\gamma_f M_u$ transferred by flexure.		
	$\gamma_f = \frac{1}{1 + \frac{2}{3} \sqrt{\frac{c_1 + d/2}{c_2 + d}}}$	$\gamma_f = \frac{1}{1 + \frac{2}{3} \sqrt{\frac{30 + 6.25/2}{42 + 6.25}}}$ $= 0.64$	
	Note—Where there is a capital, c_1 and c_2 refer to the capital dimensions as shown in the sketch in Step 9B, rather than to the column dimensions.	$M_u = 71$ kip-ft (from Step 9C) $\gamma_f M_u = 0.64(71) = 45$ kip-ft	
	Calculate width of moment transfer section = $c_1 + 2(1.5h_s)$	$42 + [2(1.5)(7.5)] = 64.5$ in.	
	Determine reinforcement needed for unbalanced moment, again estimating j at 0.95		
	$A_s = \frac{\gamma_f M_u}{\phi f_y j d}$	$A_s = \frac{(45)(12 \text{ in./ft})}{(0.90)(40)(0.95)(6.25)}$ $= 2.51 \text{ in.}^2$	
	Check validity of estimated value of j by		
	$a = \frac{A_s f_y}{0.85 f'_c b}$ and $j = 1 - \left(\frac{a}{2d}\right)$	$a = \frac{(2.51)(40,000)}{(0.85)(3000)(64.5)} = 0.61$ $j = 1 - \left(\frac{0.61}{2(6.25)}\right) = 0.95$ $\therefore$ assumed j is OK	FLEXURE 1 of Handbook Vol. 1 FLEXURE 1 of Handbook Vol. 1
	Determine number of #5 bars needed in moment transfer section	$\frac{2.51}{0.31} = 8.1$ bars $\therefore$ 9 #5 bars are needed	
	Compute nominal moment strength	Use 9 #5 bars within 64.5 in. moment transfer section and two bars on each side for a total of 13 bars	
	$M_n = \frac{A_s f_y j d}{(12 \text{ in./ft})(1000 \text{ lb/kip})}$	$M_n = \frac{(3.99)(40,000)(0.95)(6.25)}{(12)(1000)}$ $= 79$ kip-ft	

CODE REFERENCE	EXPLANATION	CALCULATION	HANDBOOK REFERENCE
11.12	D) Compute total nominal shear stress v_n, and compare with maximum allowable value. As represented in SLABS 9, $v_n = \frac{1}{\phi A} V_u + \frac{\gamma_{v1} c_{a1}}{\phi J_1} M_1 + \frac{\gamma_{v2} c_{a2}}{\phi J_2} M_2$ or $v_n = k_1 V_u + k_2 M_1 + k_3 M_2$ where values of k_1, k_2, and k_3 depend only on values of c_1, c_2, and d, and are given in SLABS 9. D1) Initial check neglecting possible unbalanced north-south moment M_2 (which will be small for Column D3 for which spans on either side are of equal length and weight of wall).	D1) For Column D3 $w_u = 0.365 \text{ ksf (from Step 2)}$ $M_n = 79 \text{ kip-ft from Step 9C}$ $-M_{ie} = 191 \text{ kip-ft}$ $V_u = \frac{0.365}{0.85} \left[(11 \times 18) - \frac{24.125 \times 48.25}{144} \right]$ $+ \frac{9}{12} \left(18 - \frac{48.25}{12} \right) \frac{0.131}{0.85} - \frac{191}{22 - 3.75}$ $= 75.9 \text{ kips}$ Obtain equivalent shear and moment at centroid of critical section. $c_a = \frac{2(0.5)(33.125)^2}{2 \times 33.125 + 48.25} = 9.58 \text{ in.}$ $g = 30.0 - (33.125 - 9.58) = 6.46 \text{ in.}$ $M_1 = M_n + V_u g$ $= 79 + 75.9 \left(\frac{6.46}{12} \right) = 120 \text{ kip-ft}$ With $k_1 = 1.65$ and $k_2 = 0.56$, the shear stress due to perimeter shear and effect of moment M_1 is $\frac{v_u}{\phi} = 1.65(75.9) + 0.56(120) = 192 \text{ psi}$ Note—The limiting shear stress v_u/ϕ can increase, necessitating a thicker slab if excessive reinforcement is used in the shear-moment transfer zone, raising the nominal moment strength M_n used in the calculation.	

