Material Properties

$E_c = 0.043 w^{1.5} \sqrt{f_c'}$, $E_c = 4700\sqrt{f_c'}$

$f_r = 0.62\sqrt{f_c'}$

Load Factors and Strength Reduction Factors

$U = 1.4(D + F)$

$U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$

$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W)$

$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R)$

$U = 1.2D + 1.0E + 1.0L + 0.2S$

$U = 0.9D + 1.6W + 1.6H$

$U = 0.9D + 1.0E + 1.6H$

Strength Condition	ϕ Factors
1. Flexure (with or without axial force)	
Tension-controlled sections	0.90
Compression-controlled sections	
Spirally reinforced	0.75
Others	0.65
2. Shear and torsion	0.75
3. Bearing on concrete	0.65
4. Post-tensioned anchorage zones	0.85
5. Struts, ties, nodal zones, and bearing areas in strut-and-tie models	0.75

$$\varphi = 0.65 + (\varepsilon_s - 0.002)\frac{250}{3}$$

Balanced, Maximum and Minimum Steel Percentages

$$\rho_b = 0.85\frac{f_c'}{f_y}\beta_1\left(\frac{600}{f_y + 600}\right), \qquad \beta_1 = 0.85 - 0.007(f_c' - 28)$$

The maximum reinforcement ratio ρ_{max}

$$\rho\ (\varepsilon_t = 0.004) = \frac{0.003 + \varepsilon_y}{0.007}\rho_b = \rho_{max}$$

A minimum steel area,

$$A_{s,min} = 0.25\frac{\sqrt{f_c'}}{f_y}b_w d$$

and not less than

$$A_{s,min} = \frac{1.4}{f_y}b_w d$$

The above requirements of $A_{s,min}$ need not be applied if, ($A_{s,provided} \geq 1.33A_{s,required}$).

Maximum Spacing

$$s = 380\left(\frac{280}{f_s}\right) - 2.5C_c \qquad but \qquad s \leq 300\left(\frac{280}{f_s}\right)$$

Analysis and Design of Singly Reinforced Concrete Rectangular Sections for Flexure

$$T = A_s f_y \qquad C = 0.85 f_c' ab \qquad a = \frac{A_s f_y}{0.85 f_c' b} \qquad a = \beta_1 c$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) \qquad R_n = \rho f_y\left(1 - \frac{\rho m}{2}\right) \qquad \rho = \frac{A_s}{bd} \qquad m = \frac{f_y}{0.85 f_c'}$$

$$M_n = A_s f_y\left(d - \frac{a}{2}\right) \qquad M_n = 0.85 f_c' ab\left(d - \frac{a}{2}\right) \qquad M_n = R_n bd^2$$

Analysis and Design of Doubly Reinforced Concrete Sections

$$\bar{\rho}_{cy} = 0.85\frac{f_c' d'}{f_y d}\beta_1\left(\frac{600}{600 - f_y}\right) + \rho' \qquad \rho' = \frac{A_s'}{bd}$$

$$T = A_s f_y = C_c + C_s, \qquad C_c = 0.85 f_c' ab, \qquad C_s = A_s'(f_s' - 0.85 f_c')$$

$$a = \frac{A_s f_y - A_s'(f_s' - 0.85 f_c')}{0.85 f_c' b},$$

$$M_n = \left(A_s f_y - A_s'(f_s' - 0.85 f_c')\right)\left(d - \frac{a}{2}\right) + A_s'(f_s' - 0.85 f_c')(d - d'),$$

$$or \qquad M_n = 0.85 f_c' ab\left(d - \frac{a}{2}\right) + A_s'(f_s' - 0.85 f_c')(d - d')$$

Reinforced Concrete T- and L- Sections

The effective width b_e for interior T-section must be the smallest value of:

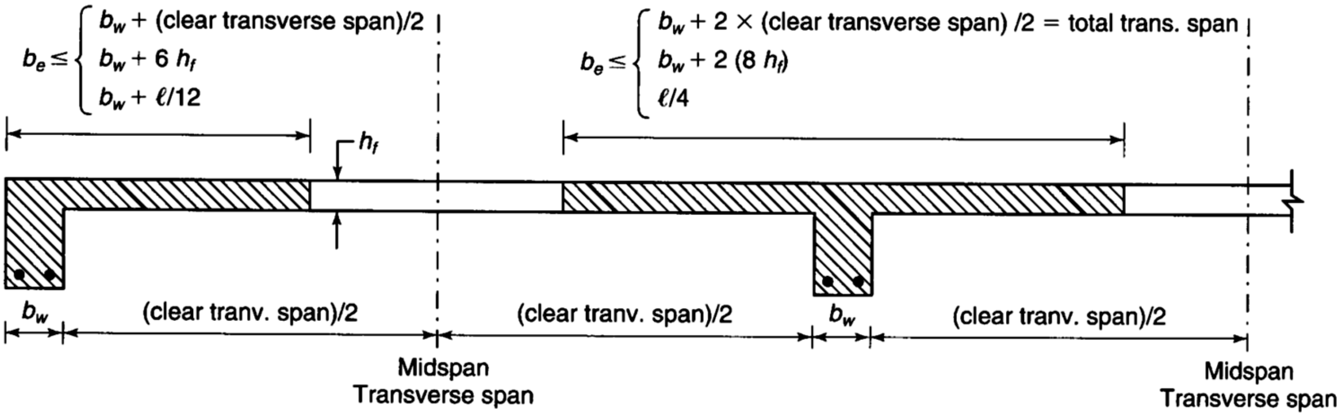
$$b_e = \frac{L}{4} \qquad b_e = b_w + 16t \qquad b_e = center\ to\ center\ spacing\ of\ beams$$

The effective width b_e for exterior (edge) L-section must be the smallest value of:

$$b_e = b_w + \frac{L}{12} \qquad b_e = b_w + 6t \qquad b_e = b_w + \frac{l_n}{2}$$

For isolated T-section beams:

$$b_e \le 4b_w \qquad and \qquad t \ge \frac{b_w}{2}$$



ℓ = length of beam span (longitudinal span)

$$T_f = A_{sf} f_y \qquad C_f = 0.85 f_c' h_f (b_e - b_w) \qquad T_w = A_{sw} f_y \qquad C_w = 0.85 f_c' a b_w$$

$$\bar{M}_{nf} = 0.85 f_c' b_e h_f \left(d - \frac{h_f}{2}\right) \qquad M_n = M_{nf} + M_{nw}$$

$$M_{nf} = 0.85 f_c' h_f (b_e - b_w)\left(d - \frac{h_f}{2}\right) = A_{sf} f_y \left(d - \frac{h_f}{2}\right) \qquad M_{nw} = A_{sw} f_y \left(d - \frac{a}{2}\right)$$

$$A_{s,min} = \frac{0.5\sqrt{f_c'}}{f_y}\, b_w d, \qquad A_{s,min} = \frac{0.25\sqrt{f_c'}}{f_y}\, bd,$$

Shear in Beams

$$V_s = \frac{A_v f_{yt} d}{s}, \qquad V_c = \frac{1}{6}\sqrt{f_c'} b_w d, \qquad V_s' = \frac{1}{3}\sqrt{f_c'} b_w d, \qquad V_{s,max} = \frac{2}{3}\sqrt{f_c'} b_w d$$

$So, \quad if \;\; V_s > V_{s,max} - The\ section\ must\ be\ enlarged\ (Dimenstions\ are\ not\ enough)$

Case I:

$$V_u \le \frac{1}{2}\phi V_c \quad - No\ shear\ reinforcement\ is\ required$$

Case II:

$$\frac{1}{2}\phi V_c < V_u \le \phi V_c \quad - Minimum\ shear\ reinforcement\ is\ required\ \left(A_{v,min}\right)$$
$$but\ not\ for\ the\ exceptions.$$

Minimum shear reinforcement, $A_{v,min}$

$$A_{v,min} = \frac{1}{16}\sqrt{f_c'}\frac{b_w s}{f_{yt}} \quad \geq \quad \frac{1}{3}\frac{b_w s}{f_{yt}},$$

$$or\ in\ the\ form \qquad \left(\frac{A_{v,min}}{s}\right) \geq \frac{1}{3}\frac{b_w}{f_{yt}} \geq \frac{1}{16}\sqrt{f_c'}\frac{b_w}{f_{yt}}\ ,$$

$$Here \qquad s_{max} \leq \frac{d}{2} \qquad or \qquad s_{max} \leq 600\ mm$$

Case III:

$$\phi V_c < V_u \leq \phi\left(V_c + V_{s,min}\right)$$

$V_{s,min}$ is the maximum of

$$V_{s,min} = \frac{1}{16}\sqrt{f_c'}b_w d \qquad and \qquad V_{s,min} = \frac{1}{3}b_w d$$

Minimum shear reinforcement is provided $\left(A_{v,min}\right)$ with

$$s_{max} \leq \frac{d}{2} \qquad or \qquad s_{max} \leq 600\ mm$$

Case IV:

$$\phi\left(V_c + V_{s,min}\right) < V_u \leq \phi(V_c + V_s') \quad - stirrups\ are\ required$$

$$here \qquad s_{max} \leq \frac{d}{2} \qquad or \qquad s_{max} \leq 600\ mm$$

Case V:

$$\phi(V_c + V_s') < V_u \leq \phi\left(V_c + V_{s,max}\right) \quad - stirrups\ are\ required$$

$$here \qquad s_{max} \leq \frac{d}{4} \qquad or \qquad s_{max} \leq 300\ mm$$

Axially Loaded Short Columns

$$P_{n,max} = 0.8\left[0.85f_c'\left(A_g - A_{st}\right) + A_{st}f_y\right] \qquad P_{n,max} = 0.85\left[0.85f_c'\left(A_g - A_{st}\right) + A_{st}f_y\right]$$

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}, \qquad \rho_s = \frac{4a_s(D_{ch} - d_s)}{sD_{ch}^2}$$

Short Columns with Axial Load and Bending Moment

$$c_b = \left(\frac{600}{600 + f_y}\right) d \qquad f_s = 600\left(\frac{d - c}{c}\right) \qquad f_s' = 600\left(\frac{c - d'}{c}\right)$$

$$C_c = 0.85\, f_c' ab, \qquad C_s = A_s'(f_s' - 0.85\, f_c'), \qquad T = A_s f_s$$

$$P_n = \frac{bhf_c'}{\frac{3he}{d^2} + 1.18} + \frac{A_s' f_y}{\frac{e}{d - d'} + 0.5} \qquad P_n = A_g\left[\frac{f_c'}{\left(\frac{3}{\xi^2}\right)\left(\frac{e}{h}\right) + 1.18} + \frac{\rho_g f_y}{\left(\frac{2}{\gamma}\right)\left(\frac{e}{h}\right) + 1}\right]$$

$$P_n = 0.85 f_c' bd\left\{\rho'(m-1) - \rho m + \left(1 - \frac{e'}{d}\right) + \sqrt{\left(1 - \frac{e'}{d}\right)^2 + 2\left[\left(\frac{e'}{d}\right)(\rho m - \rho' m + \rho') + \rho'(m-1)\left(1 - \frac{d'}{d}\right)\right]}\right\}$$

When $\rho = \rho'$ then

$$P_n = 0.85 f_c' bd\left\{-\rho + 1 - \frac{e'}{d} + \sqrt{\left(1 - \frac{e'}{d}\right)^2 + 2\rho\left[(m-1)\left(1 - \frac{d'}{d}\right) + \frac{e'}{d}\right]}\right\}$$

$$where \qquad e' = e + \frac{d - d'}{2}$$

Biaxially Loaded Columns

$$\frac{1}{P_u} = \frac{1}{P_{ux}} + \frac{1}{P_{uy}} - \frac{1}{P_{uo}} \qquad or \qquad \frac{1}{P_n} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_{no}}$$

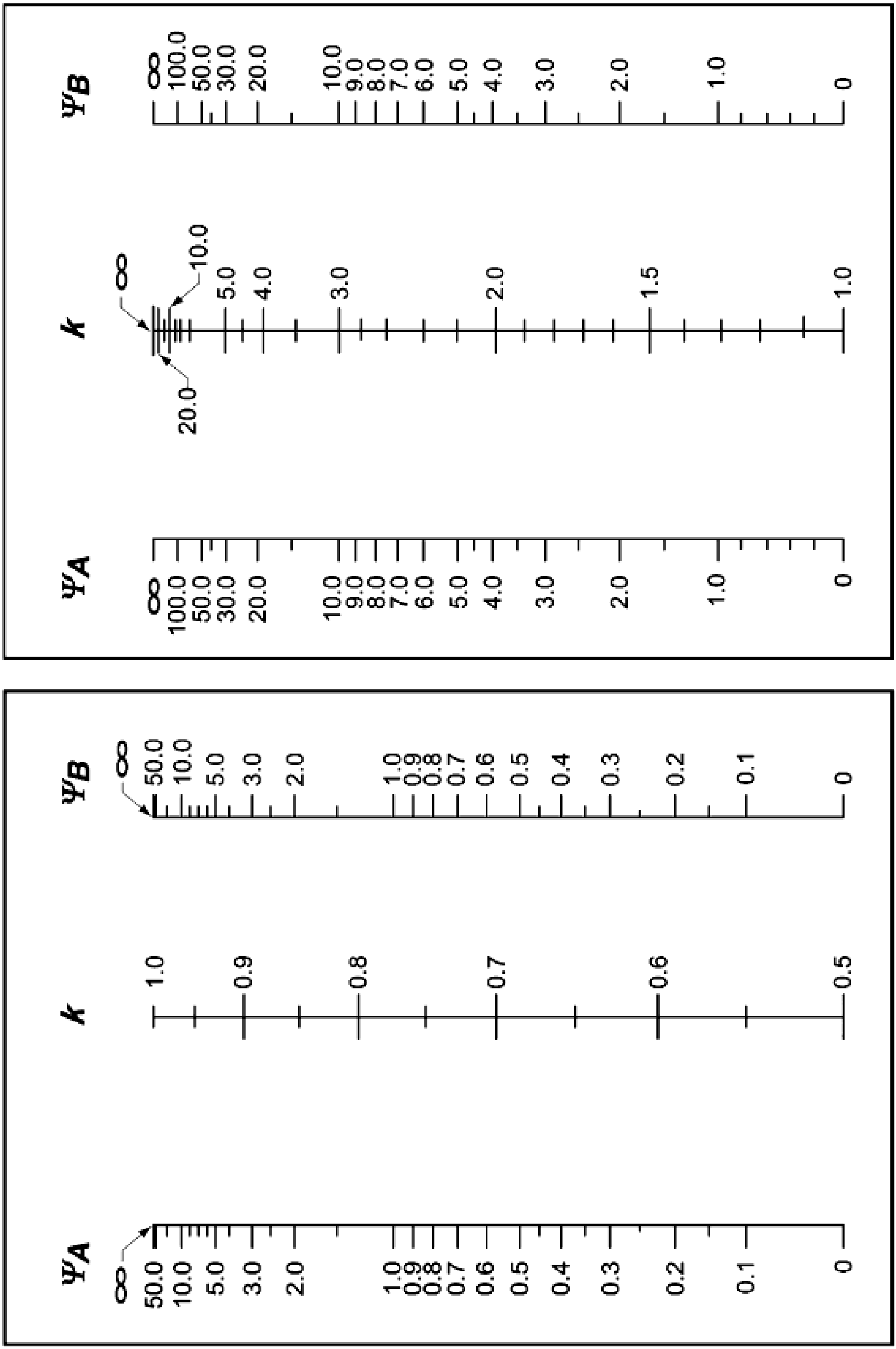
$$P_{no} = A_g\left[0.85 f_c'\left(1 - \rho_g\right) + \rho_g f_y\right]$$

Slender Columns

$$r = \sqrt{I/A} \qquad r_x = 0.3h \qquad r_y = 0.3b \qquad r = 0.25D$$

$$Q = \frac{\sum P_u\, \Delta_o}{V_{us} l_c} \le 0.05 \qquad P_c = \frac{\pi^2 EI}{(kl_u)^2} \qquad e_{min} = (15 + 0.03h)$$

$$\psi = \frac{\sum E_c I_c / l_c}{\sum E_b I_b / l_b} \qquad EI = \frac{\left(0.2 E_c I_g + E_s I_{se}\right)}{1 + \beta_{dns}}, \qquad EI = \frac{0.4 E_c I_g}{1 + \beta_{dns}}$$



(b) Sway Frames

(a) Nonsway Frames

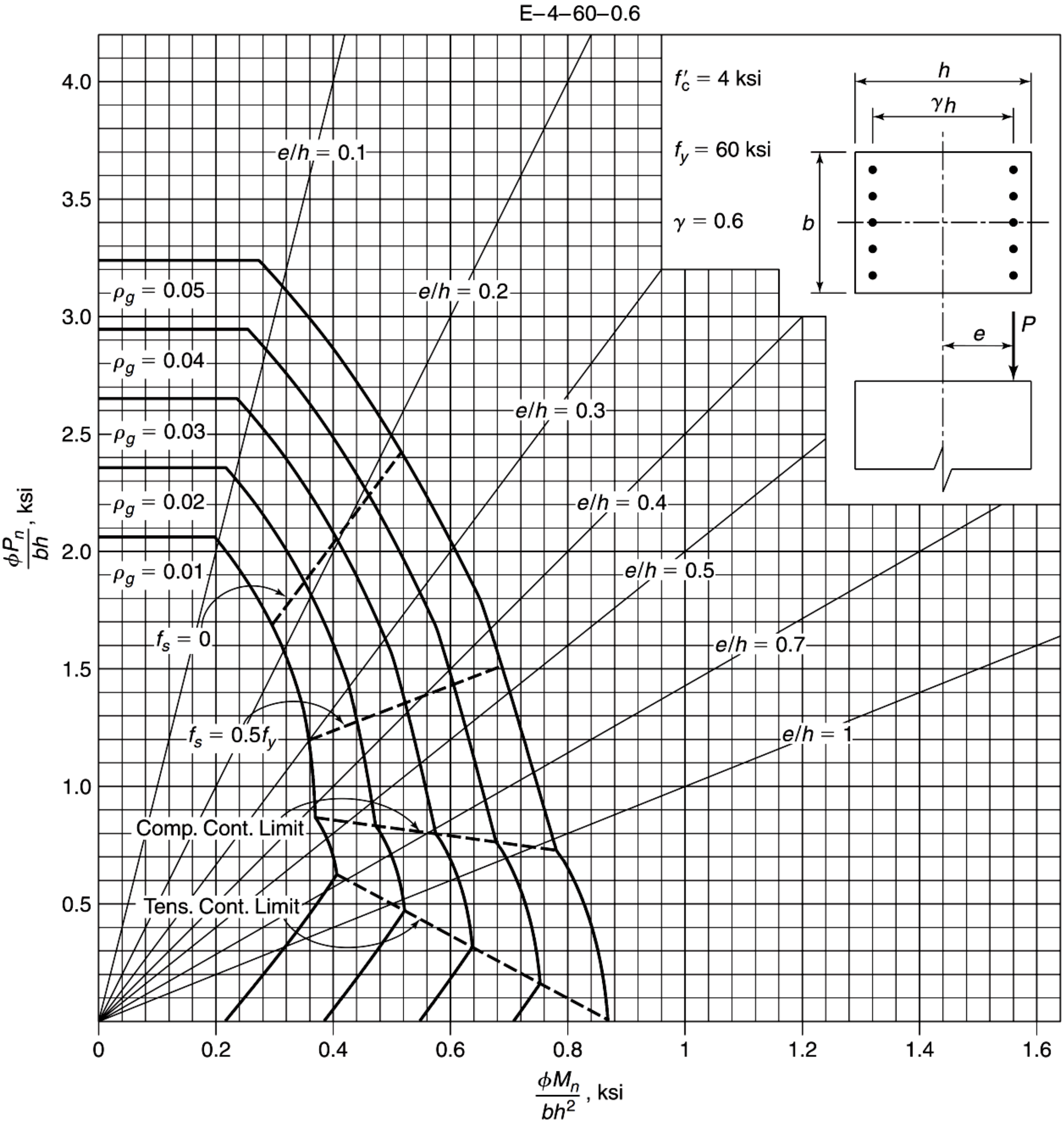


Fig. A-6a
Nondimensional interaction diagram for rectangular tied columns with bars in two faces: $f'_c = 4000$ psi and $\gamma = 0.60$.

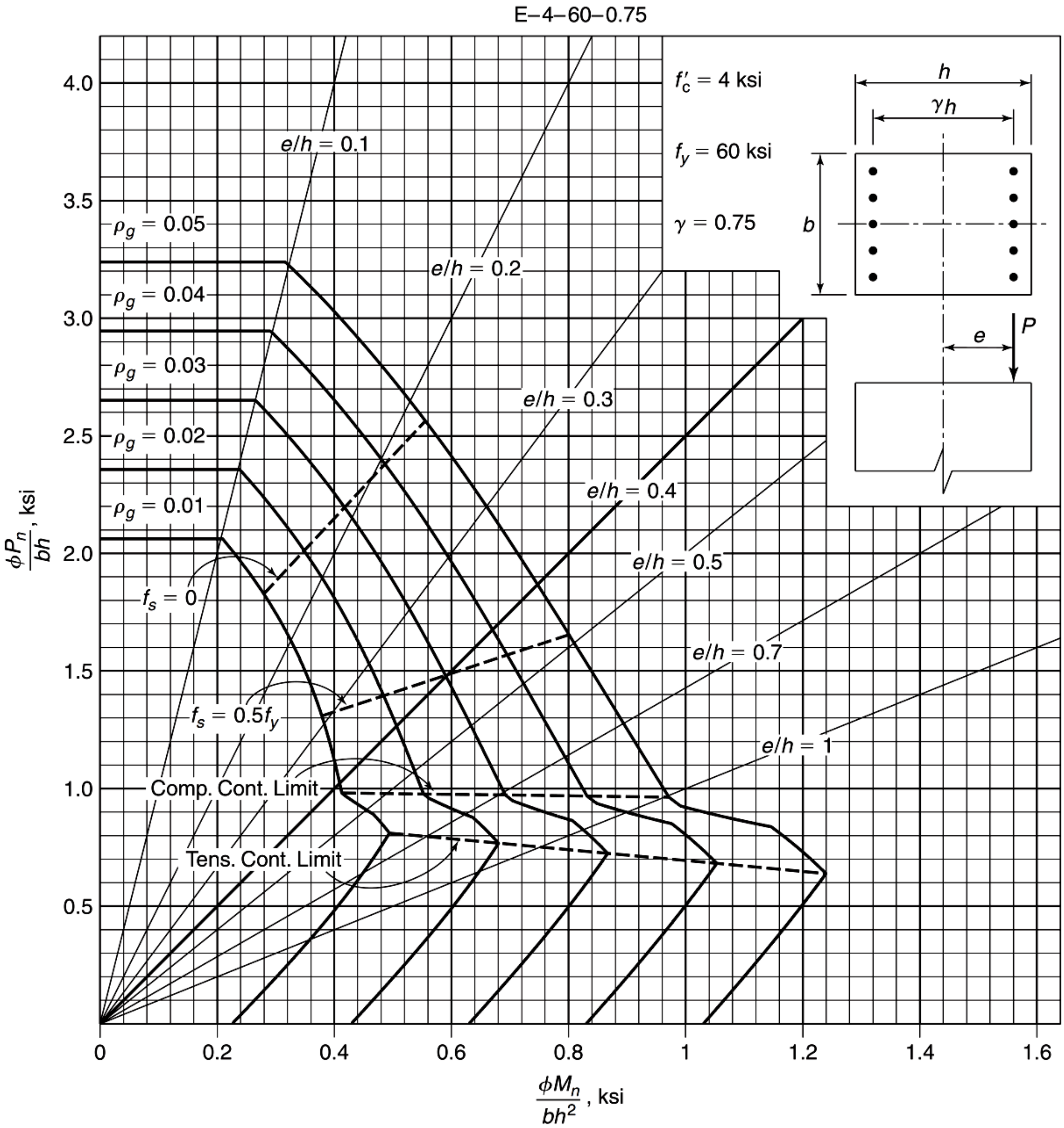


Fig. A-6b
Nondimensional interaction diagram for rectangular tied column with bars in two faces: f'_c = 4000 psi and γ = 0.75.

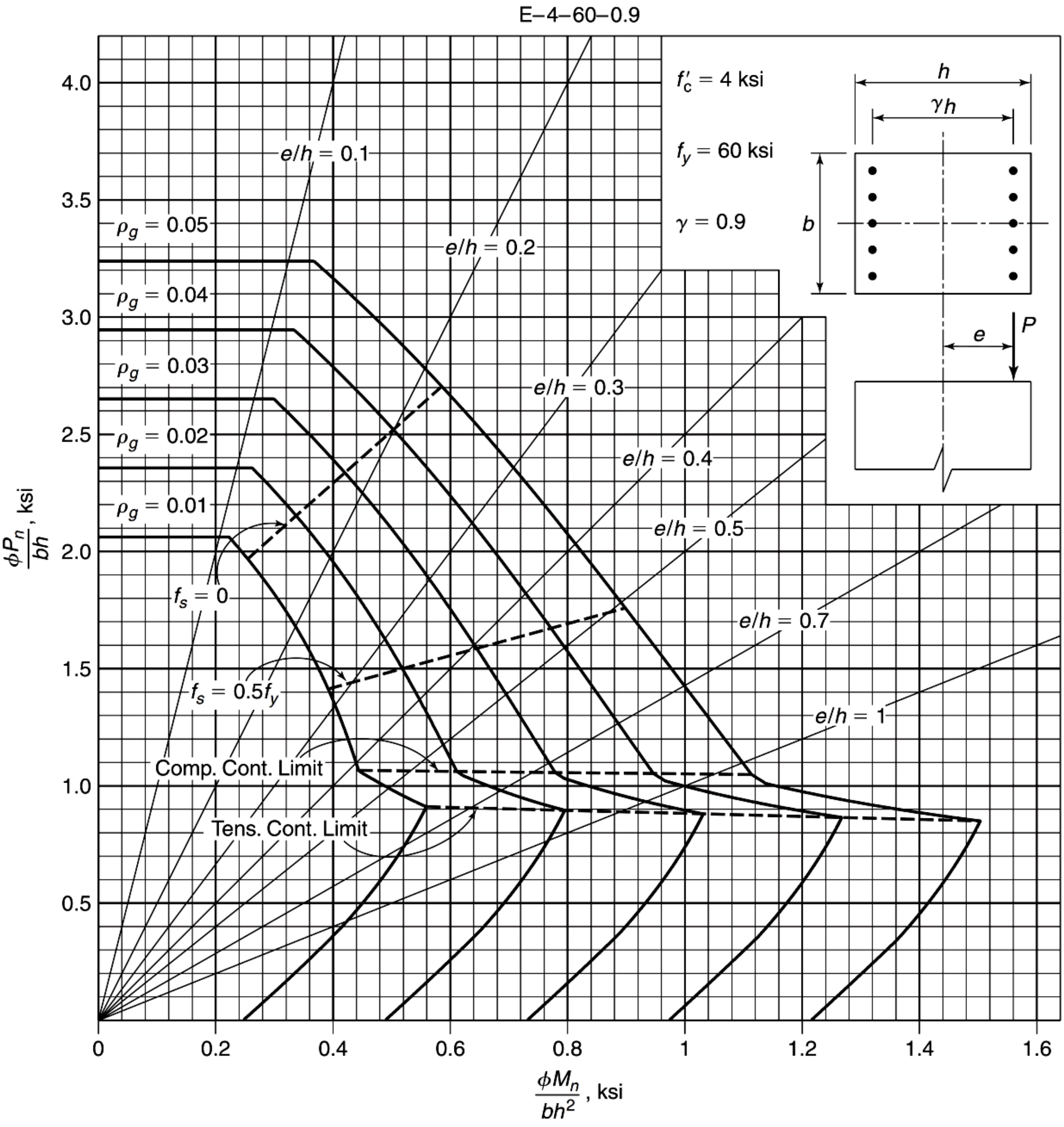


Fig. A-6c
Nondimensional interaction diagram for rectangular tied column with bars in two faces: f'_c = 4000 psi and γ = 0.90.

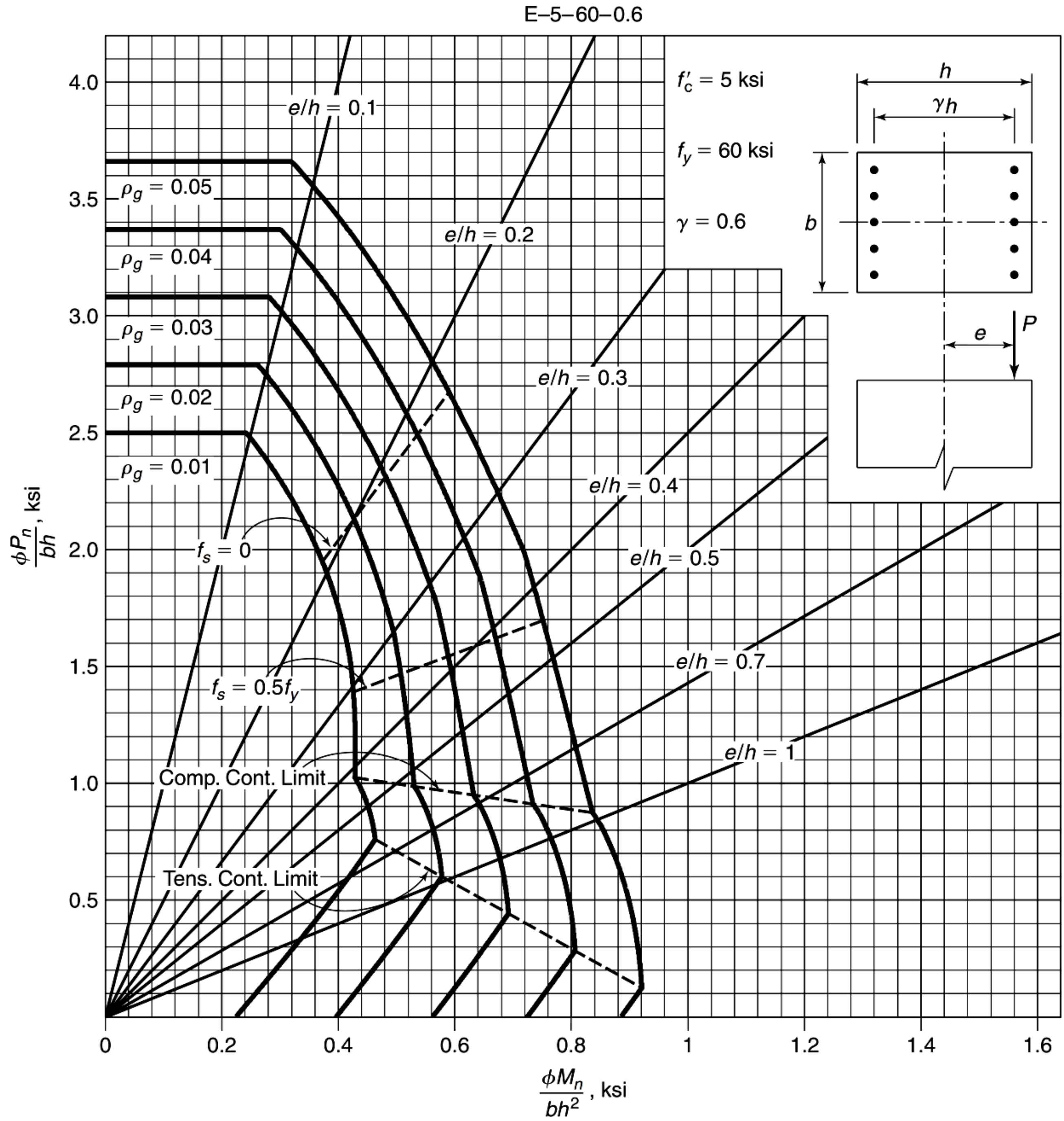


Fig. A-7a
Nondimensional interaction diagram for rectangular tied column with bars in two faces: $f_c' = 5000$ psi and $\gamma = 0.60$.

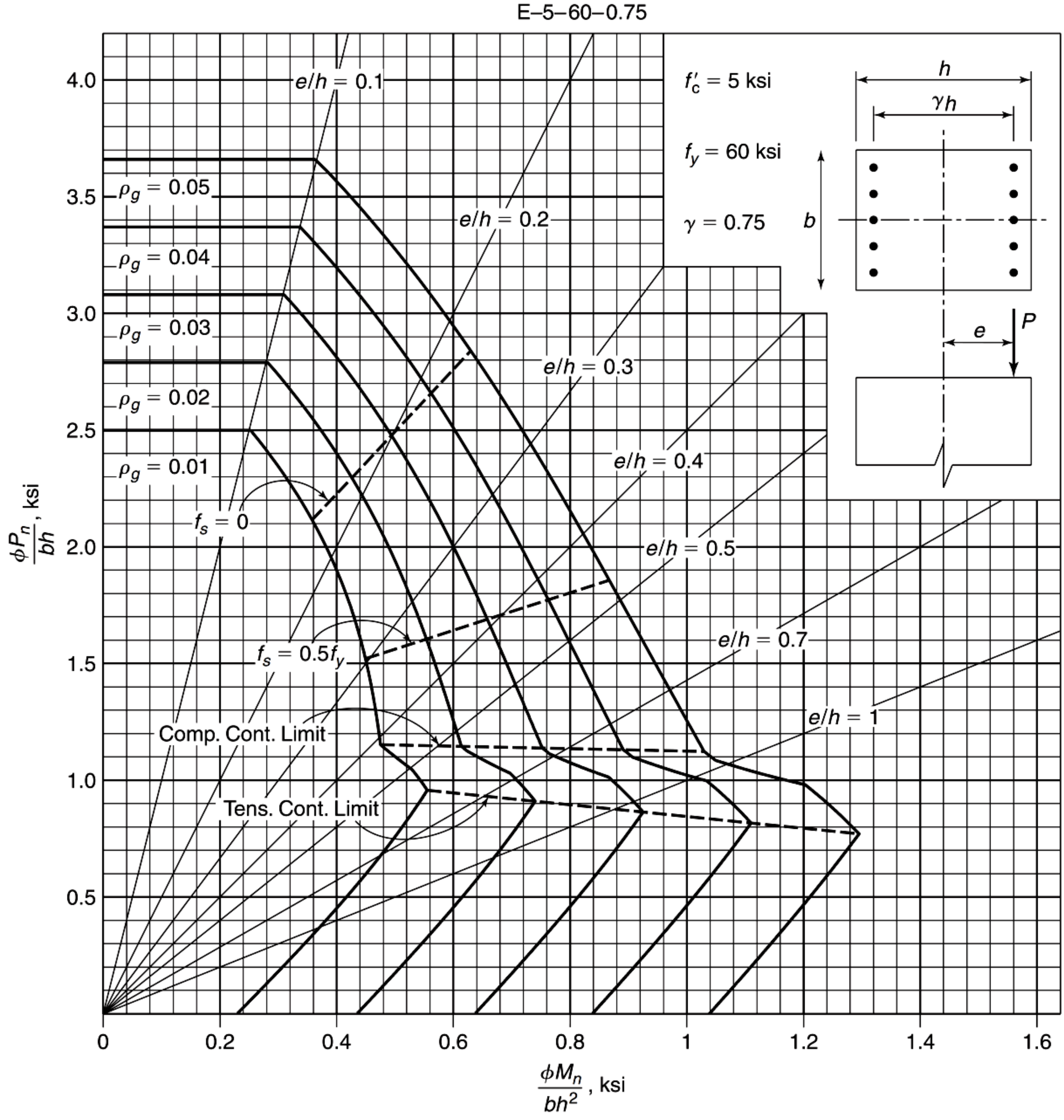


Fig. A-7b
Nondimensional interaction diagram for rectangular tied column with bars in two faces: $f'_c = 5000$ psi and $\gamma = 0.75$.

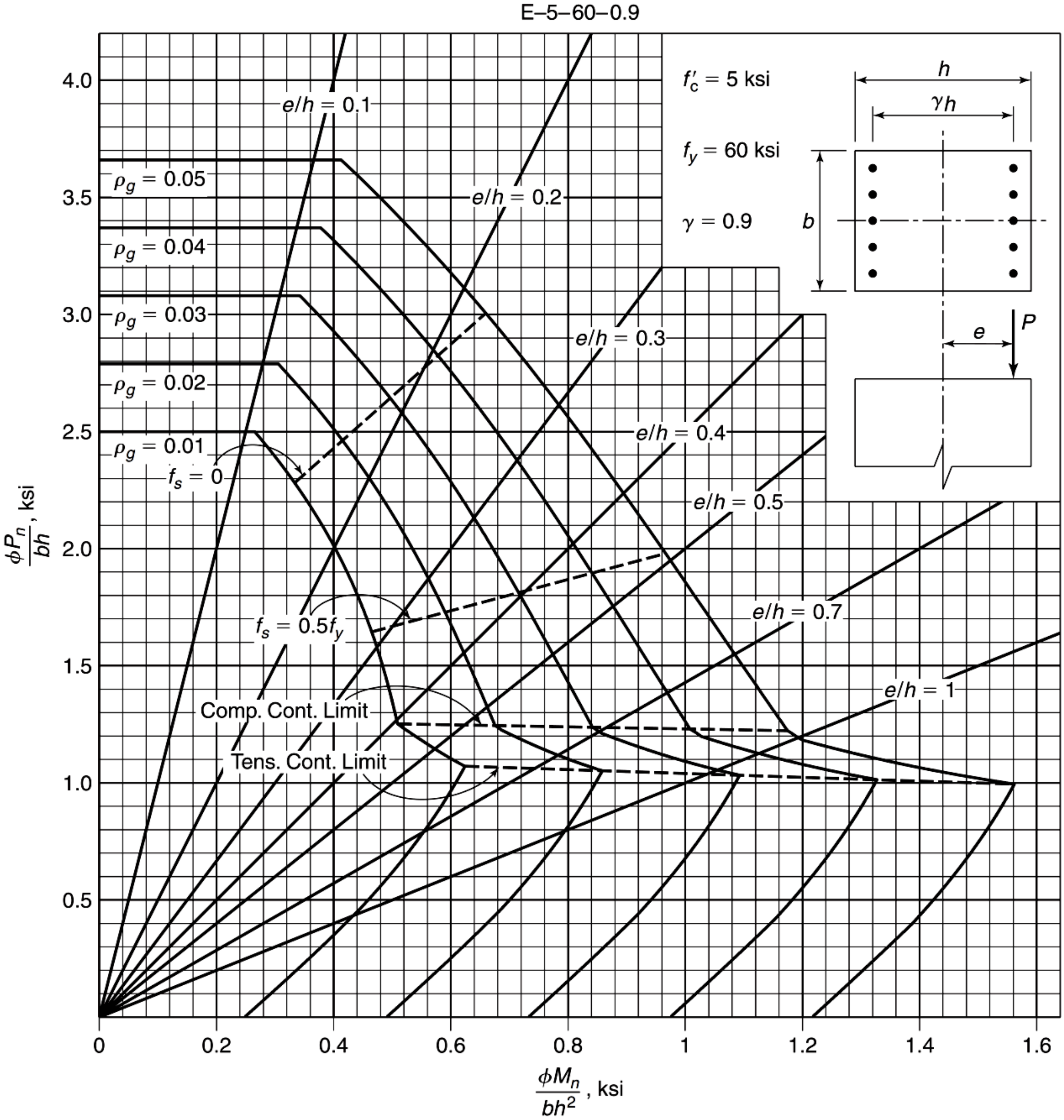


Fig. A-7c
Nondimensional interaction diagram for rectangular tied column with bars in two faces: $f'_c = 5000$ psi and $\gamma = 0.90$.

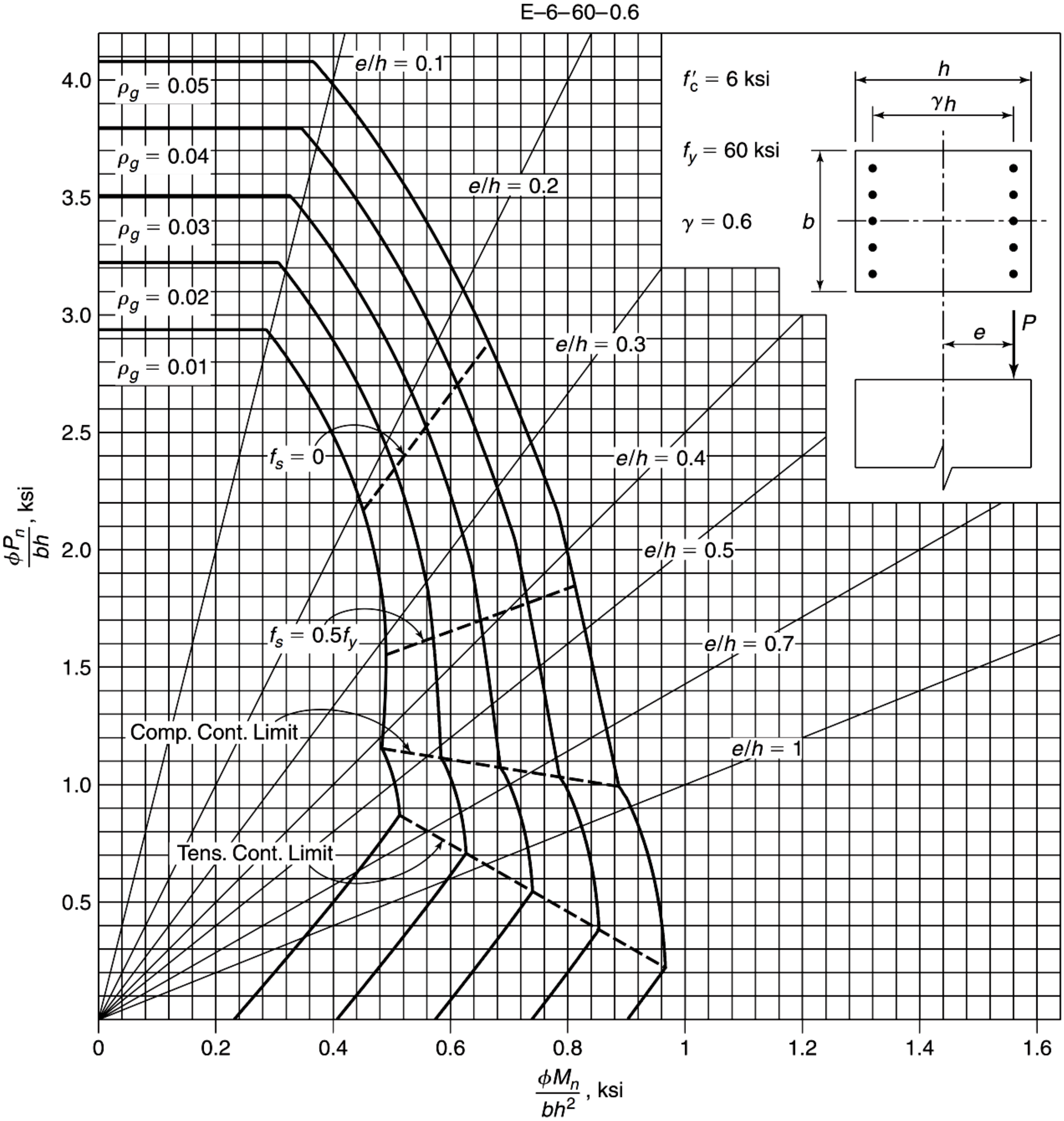


Fig. A-8a
Nondimensional interaction diagram for rectangular tied column with bars in two faces: $f'_c = 6000$ psi and $\gamma = 0.60$.

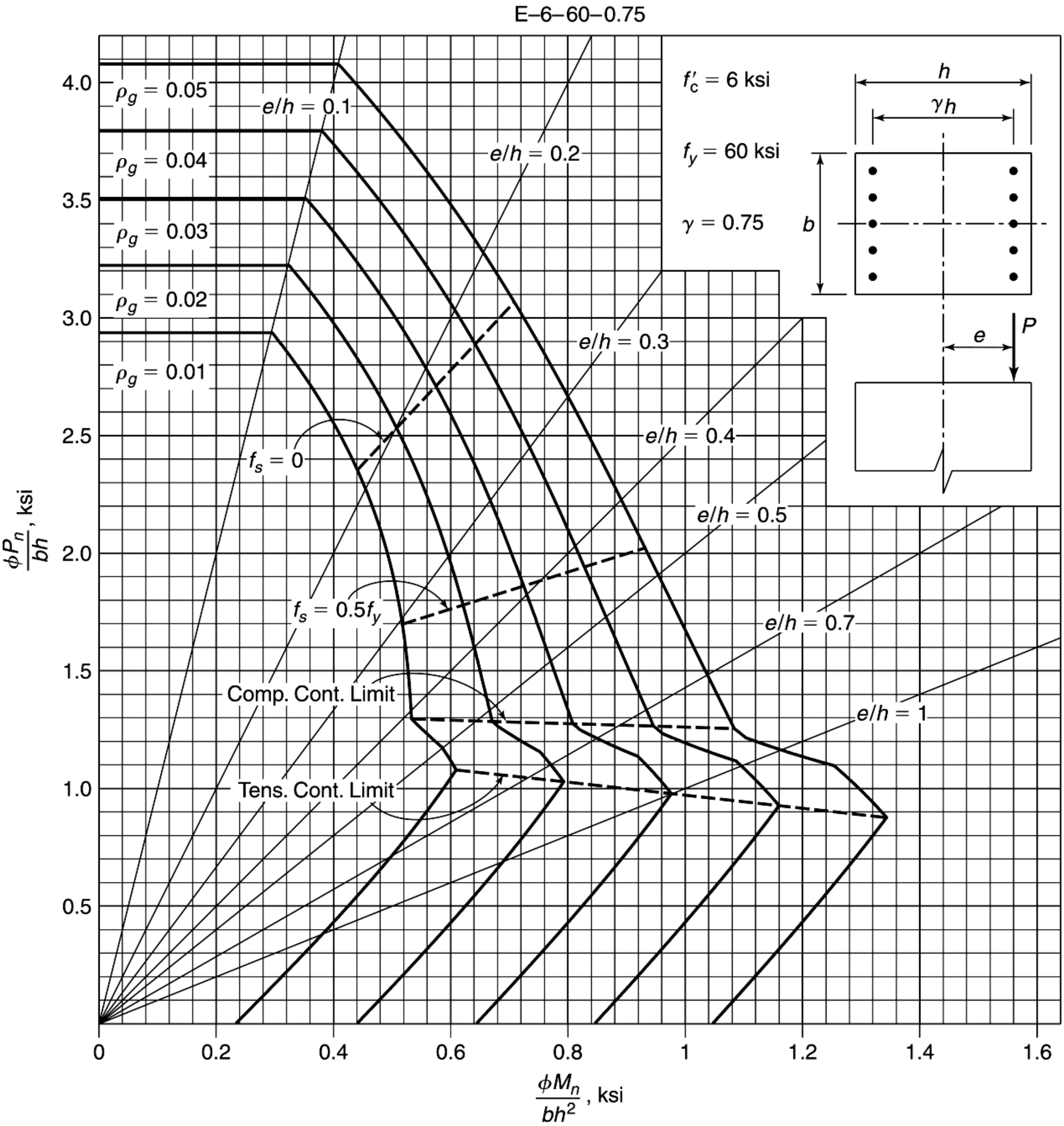


Fig. A-8b
Nondimensional interaction diagram for rectangular tied column with bars in two faces: $f'_c = 6000$ psi and $\gamma = 0.75$.

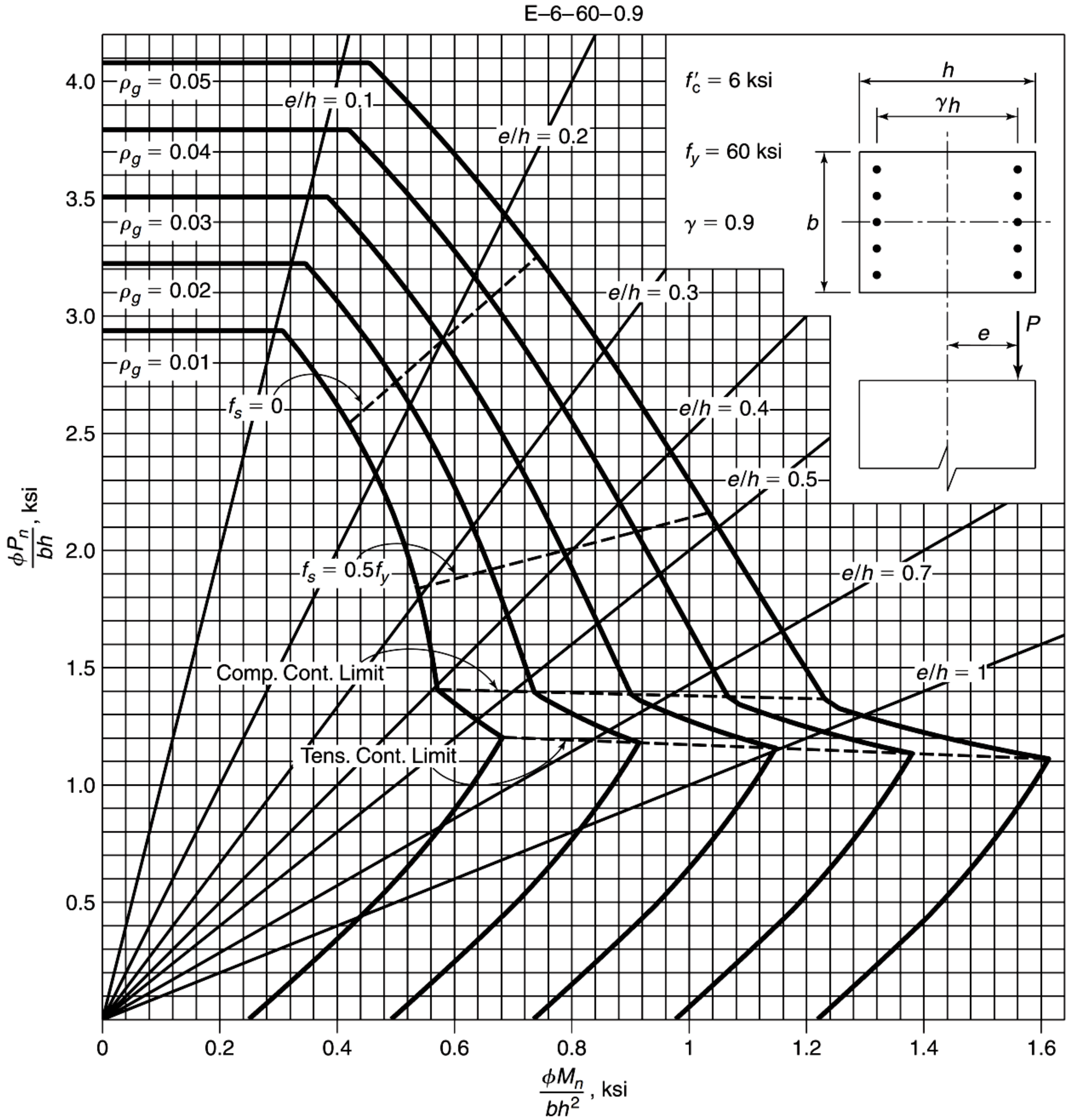


Fig. A-8c
Nondimensional interaction diagram for rectangular tied column with bars in two faces: $f'_c = 6000$ psi and $\gamma = 0.90$.

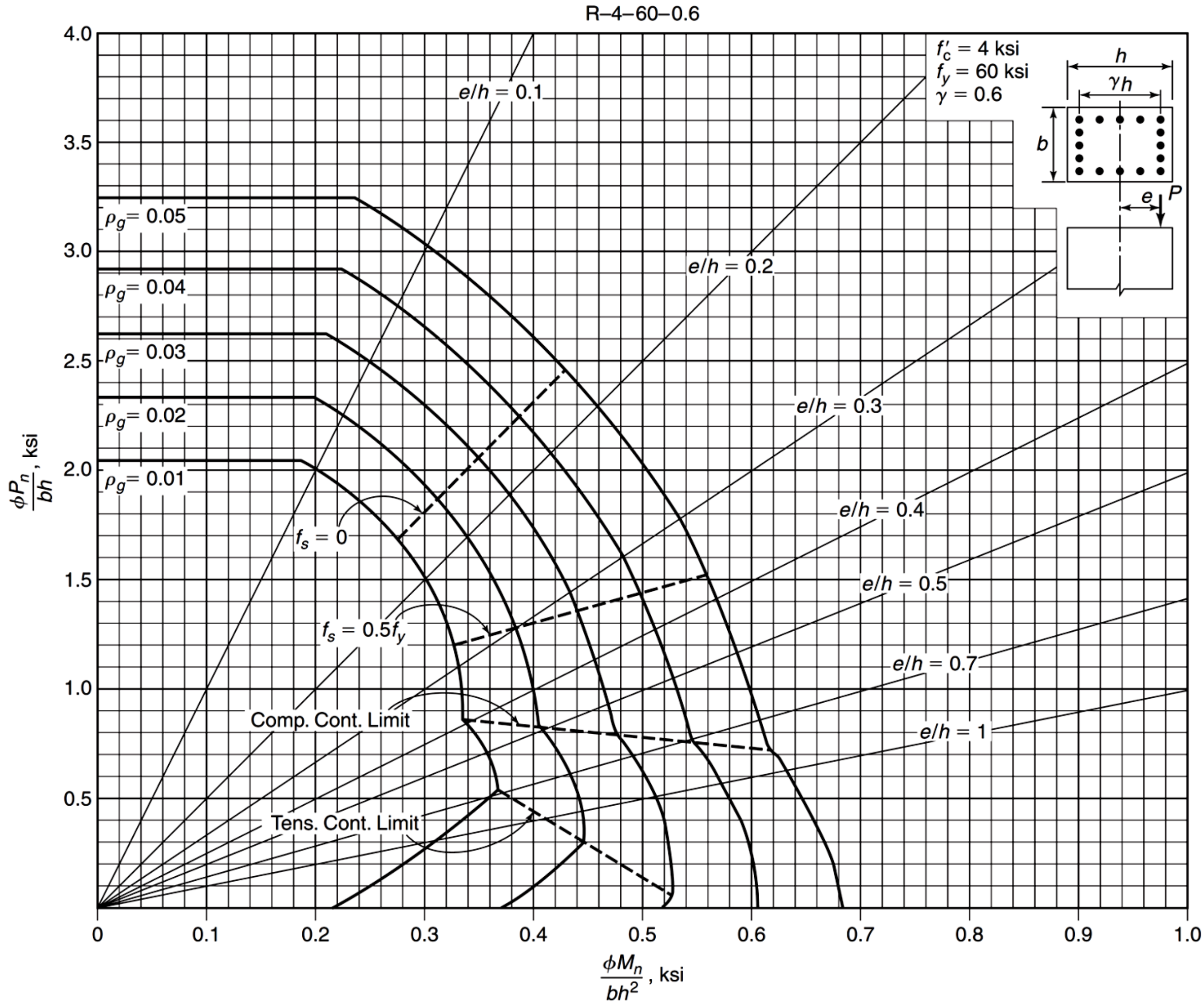


Fig. A-9a
Nondimensional interaction diagram for rectangular tied column with bars in four faces: $f'_c = 4000$ psi and $\gamma = 0.60$.

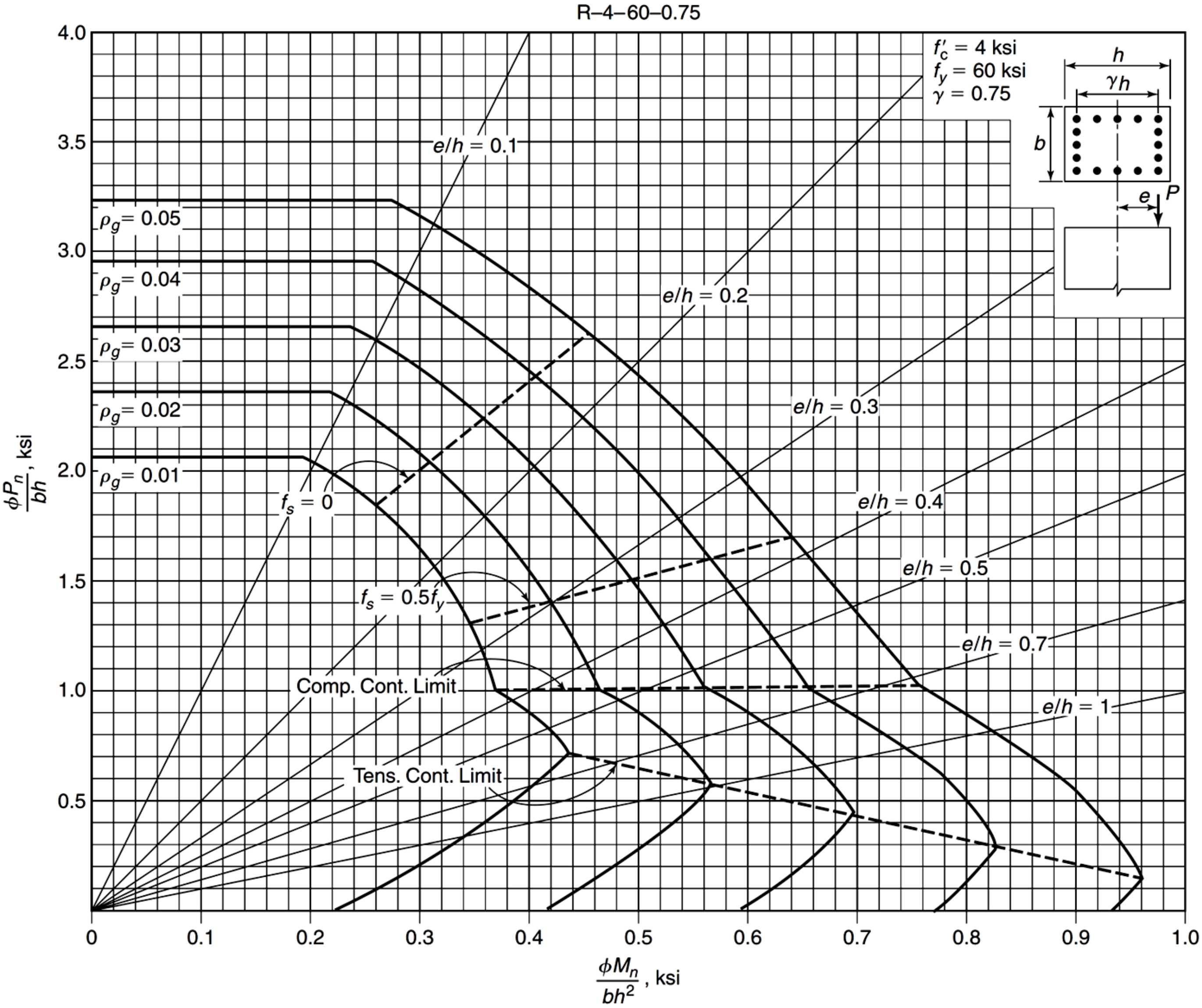


Fig. A-9b
Nondimensional interaction diagram rectangular for tied column with bars in four faces: f'_c = 4000 psi and γ = 0.75.

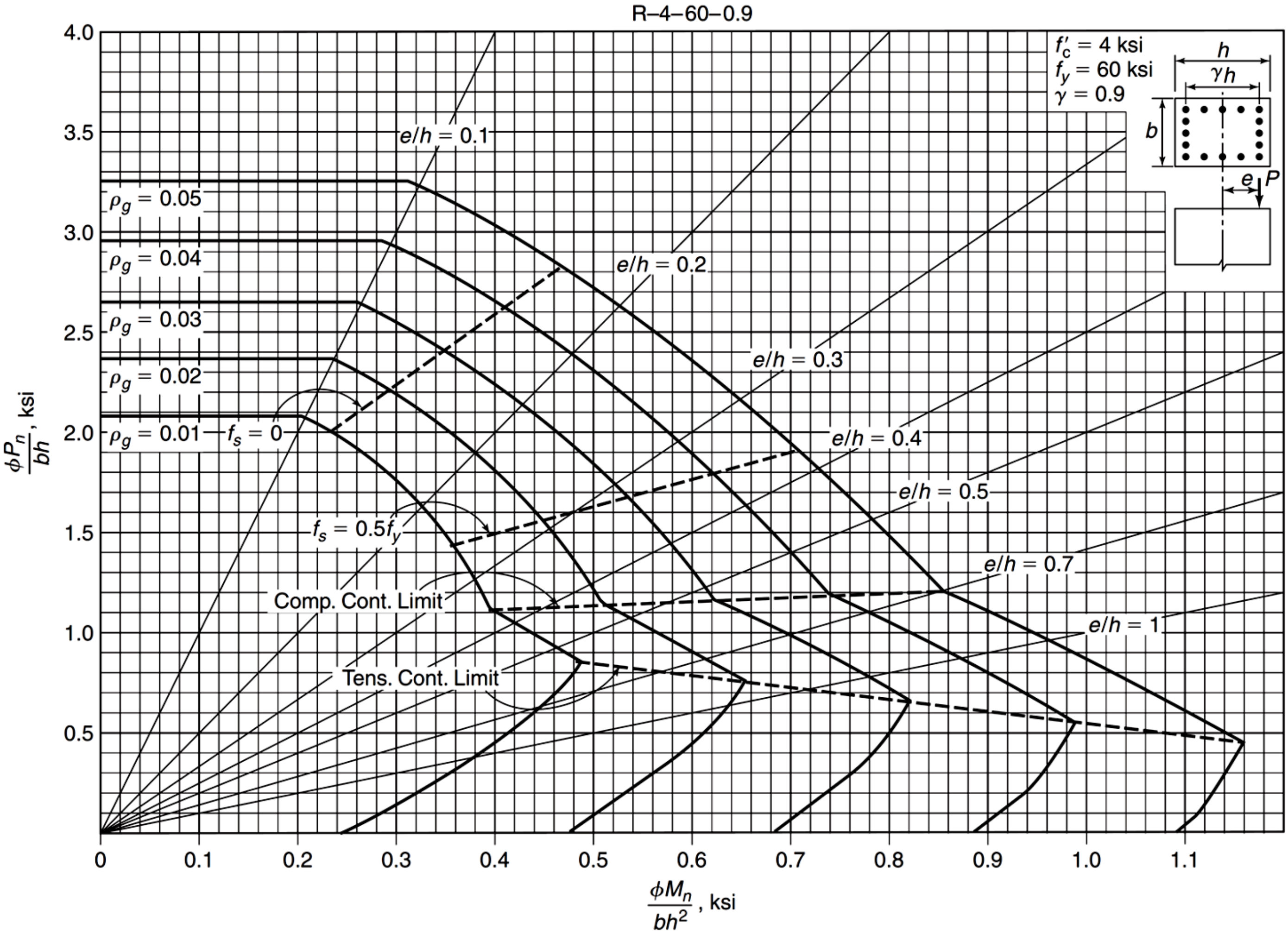


Fig. A-9c
Nondimensional interaction diagram for rectangular tied column with bars in four faces: f'_c = 4000 psi and γ = 0.90.

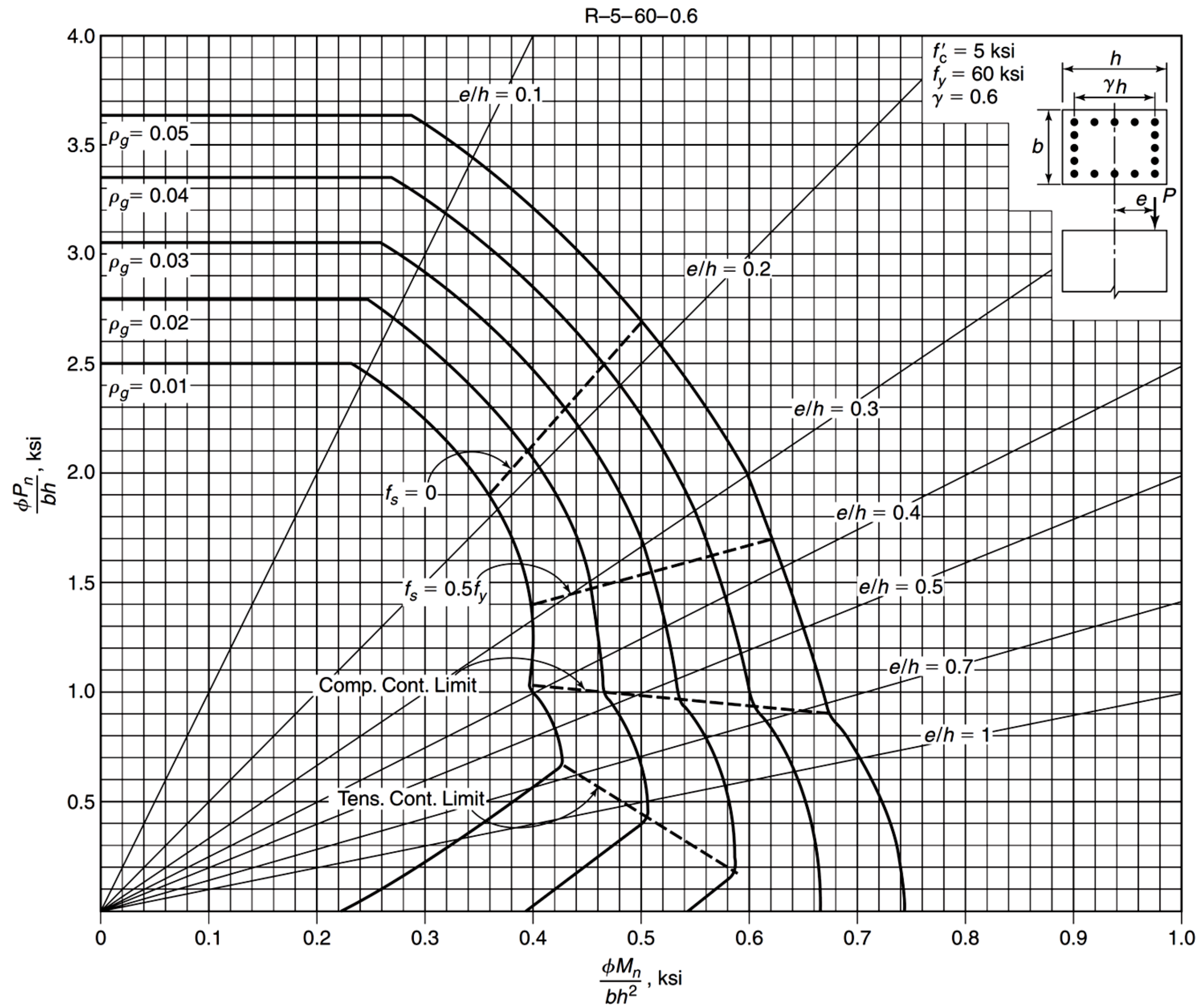


Fig. A-10a
Nondimensional interaction diagram for rectangular tied column with bars in four faces: f'_c = 5000 psi and γ = 0.60.

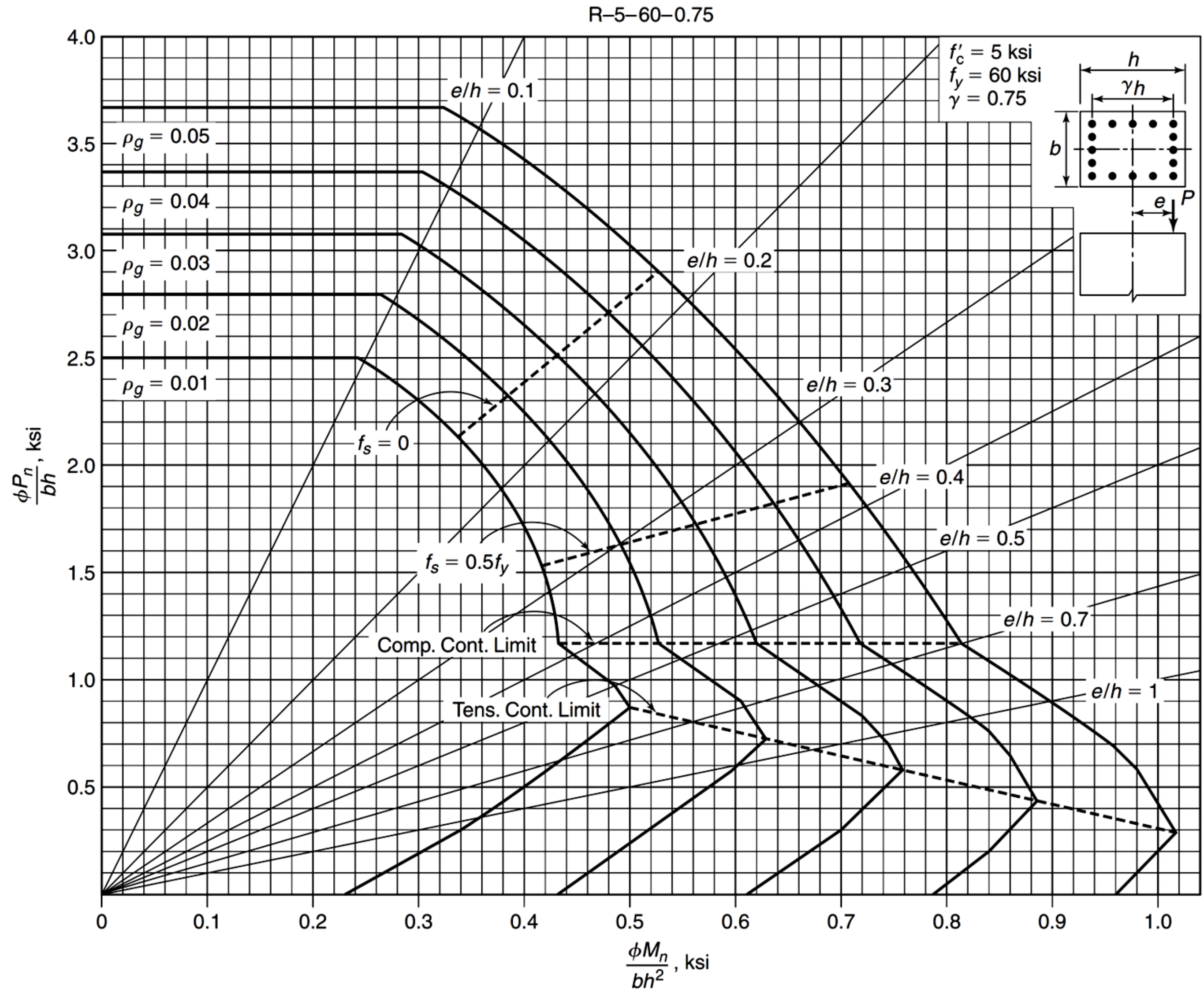


Fig. A-10b
Nondimensional interaction diagram for tied column with bars in four faces: f'_c = 5000 psi and γ = 0.75.

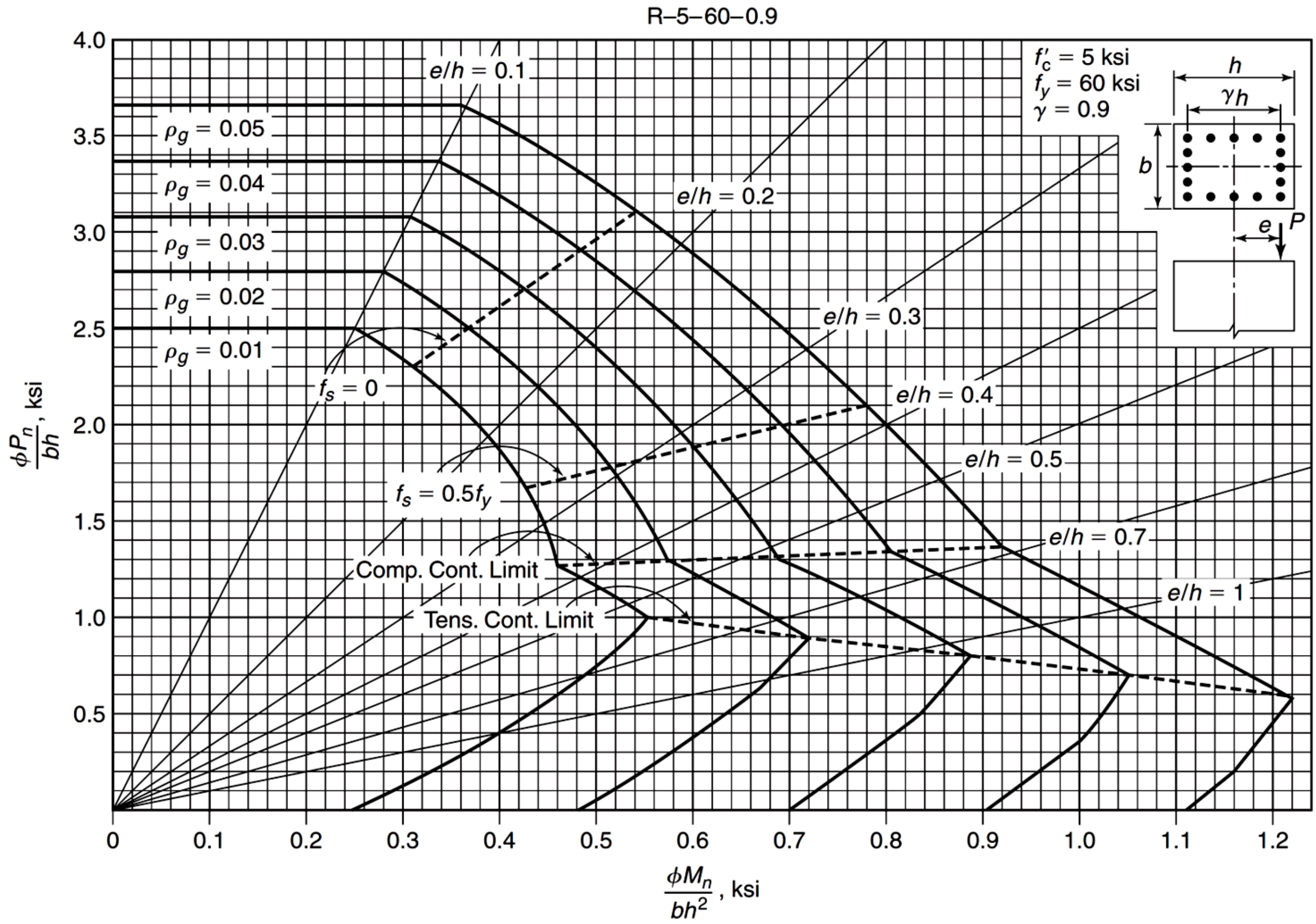


Fig. A-10c
Nondimensional interaction diagram for tied column with bars in four faces: $f'_c = 5000$ psi and $\gamma = 0.90$.

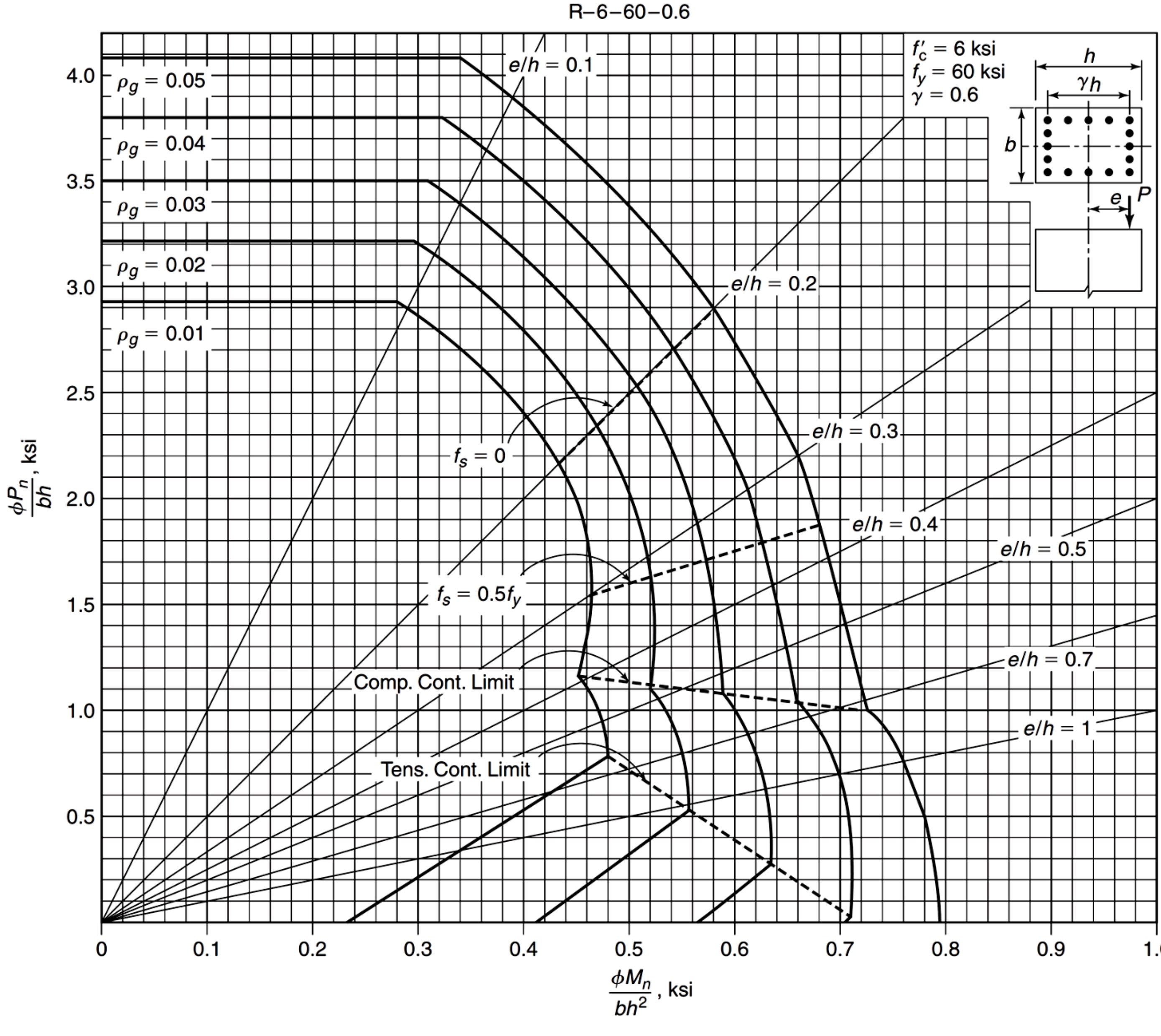


Fig. A-11a
Nondimensional interaction diagram for tied column with bars in four faces: f'_c = 6000 psi and γ = 0.60.

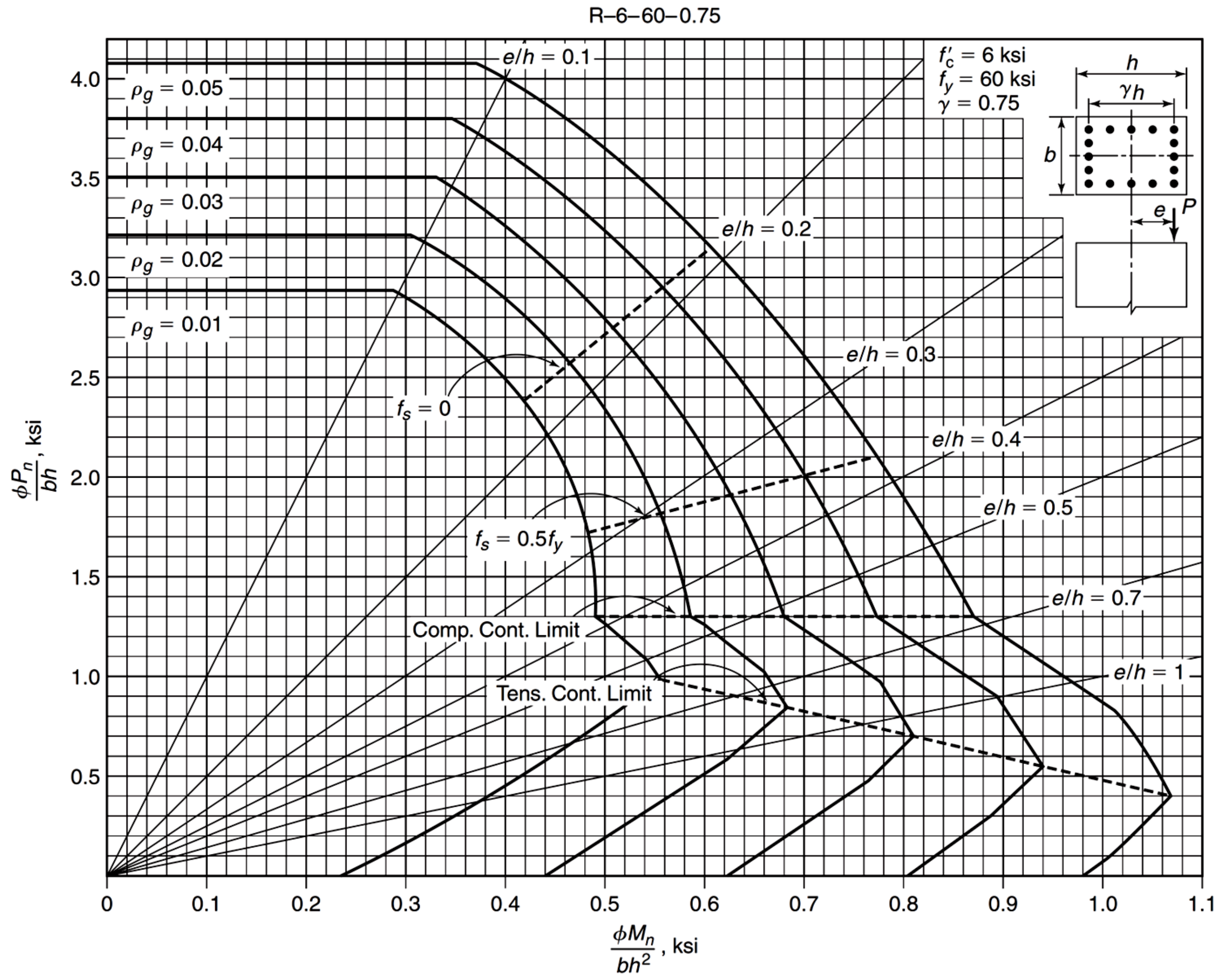


Fig. A-11b
Nondimensional interaction diagram for tied column with bars in four faces: $f_c' = 6000$ psi and $\gamma = 0.75$.

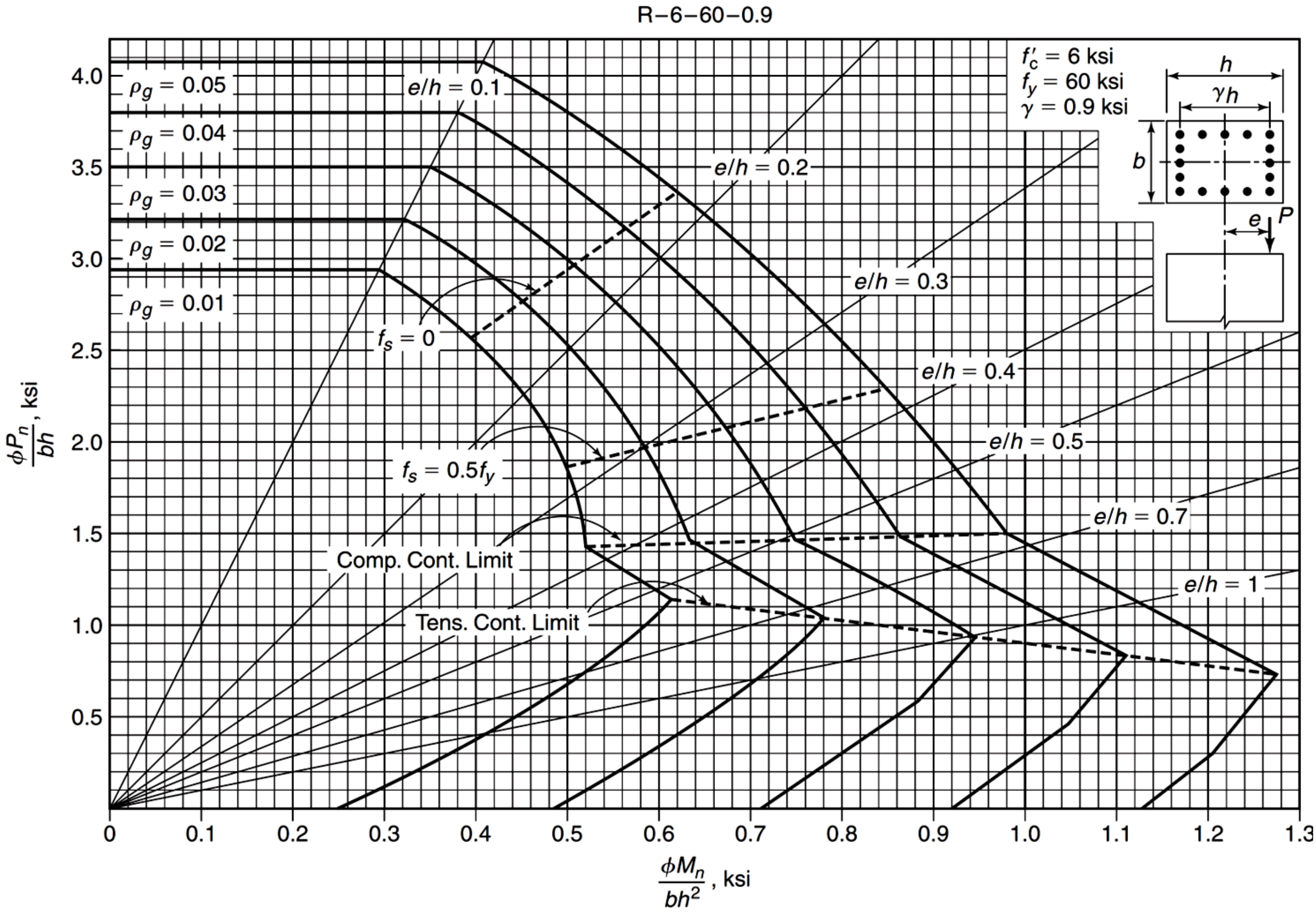


Fig. A-11c
Nondimensional interaction diagram for tied column with bars in four faces: $f'_c = 6000$ psi and $\gamma = 0.90$.

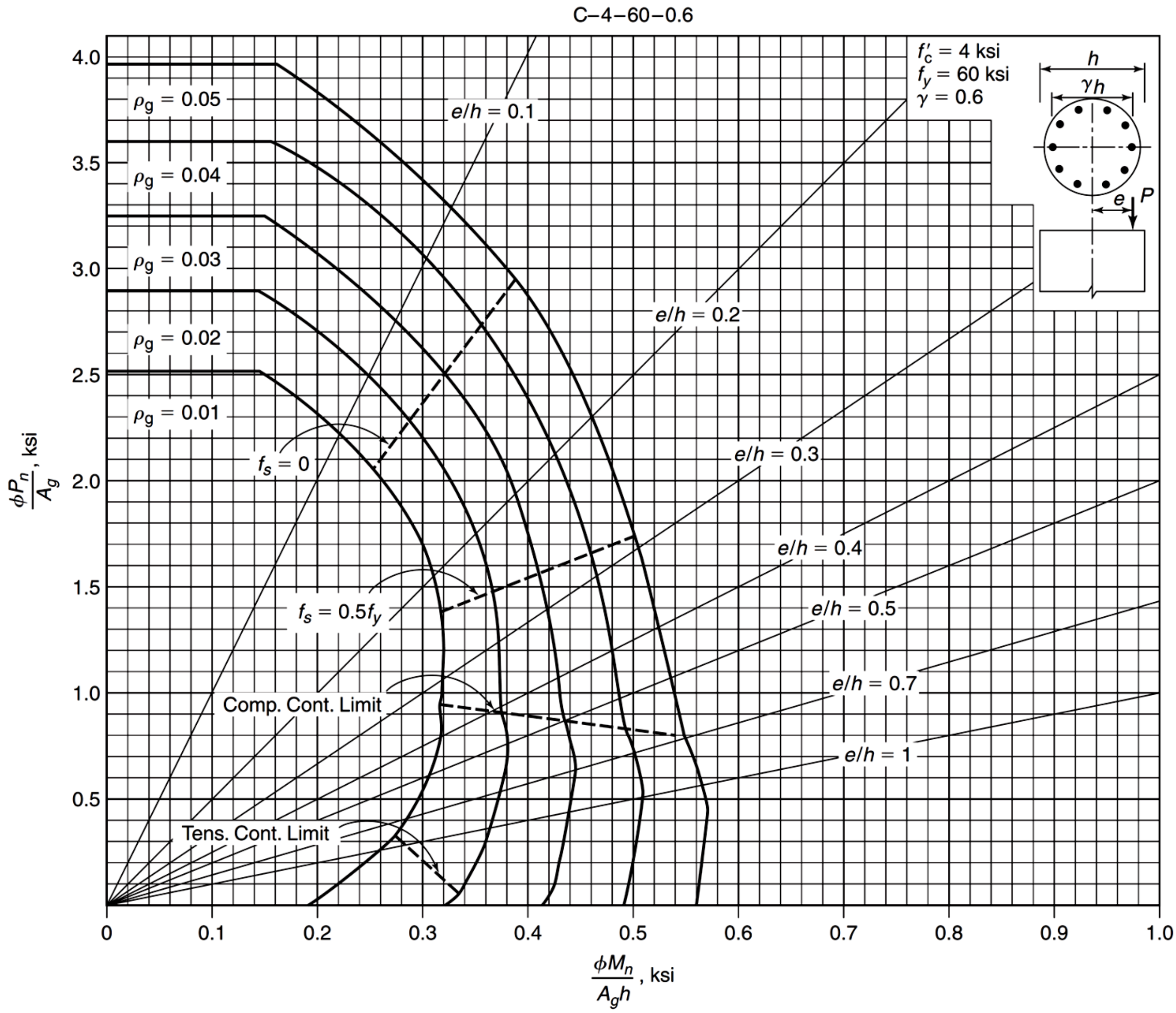


Fig. A-12a
Nondimensional interaction diagram for circular spiral column: $f'_c = 4000$ psi and $\gamma = 0.60$.

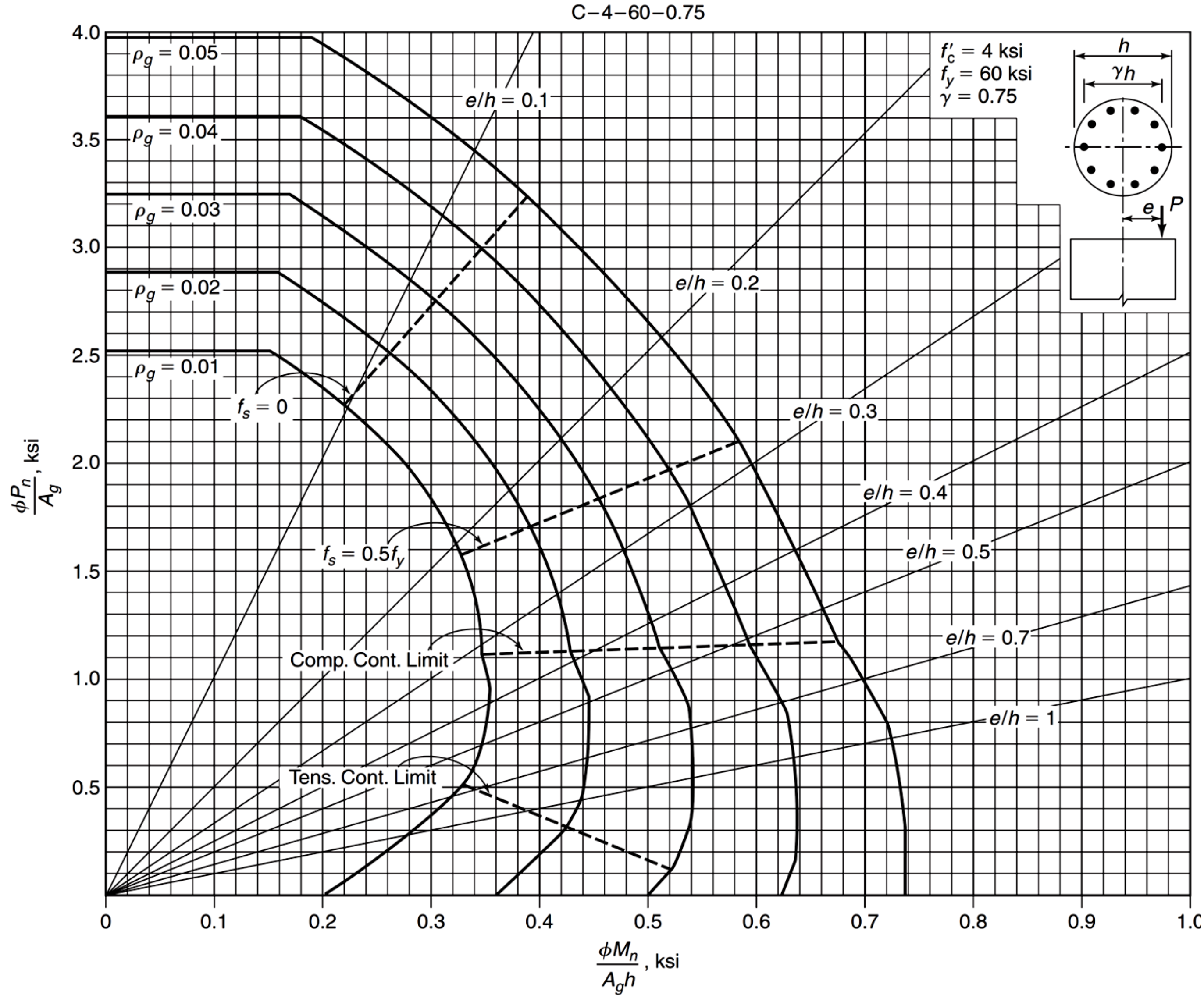


Fig. A-12b
Nondimensional interaction diagram for circular spiral column: f'_c = 4000 psi and γ = 0.75.

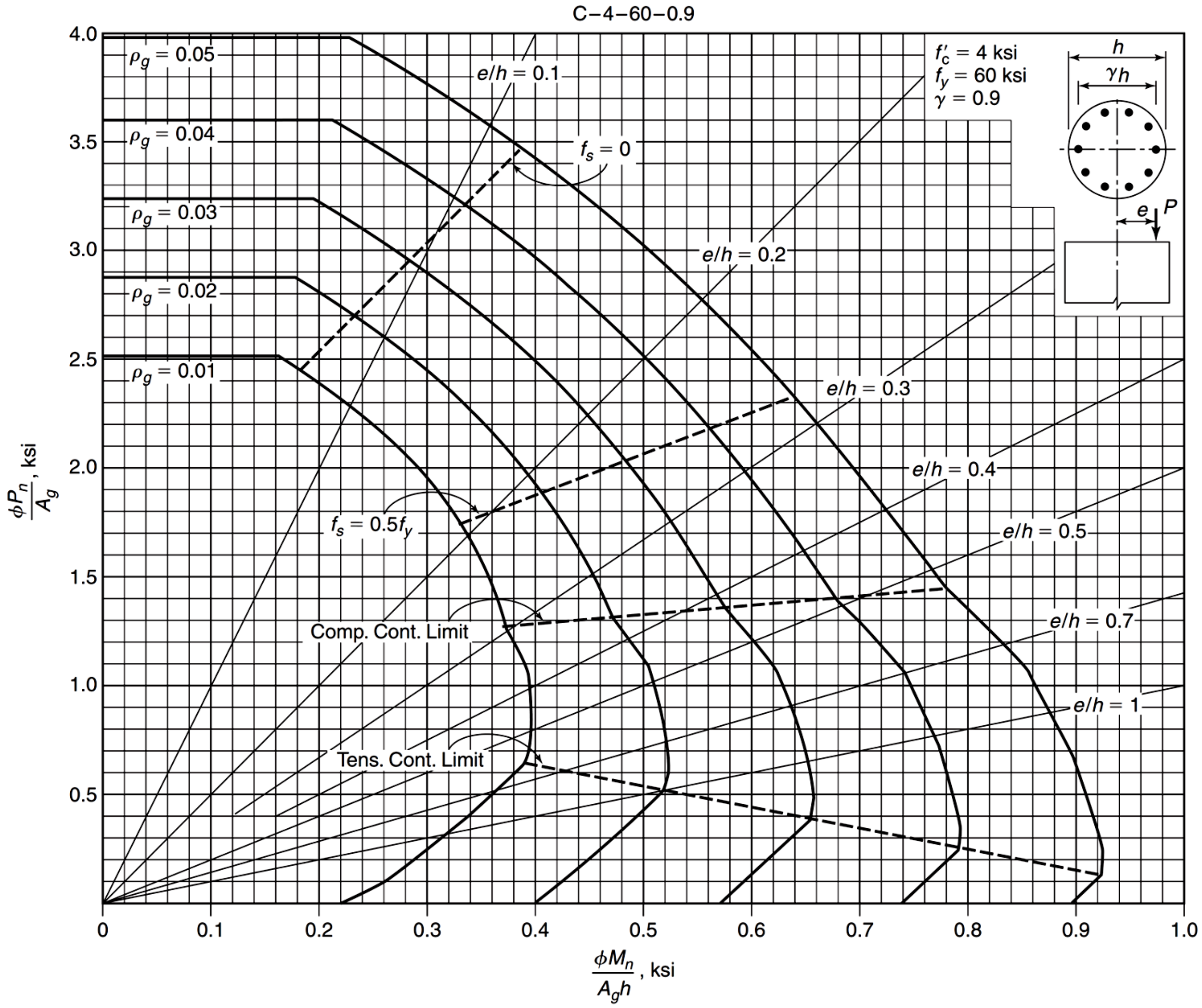


Fig. A-12c
Nondimensional interaction diagram for circular spiral column: f'_c = 4000 psi and γ = 0.90.

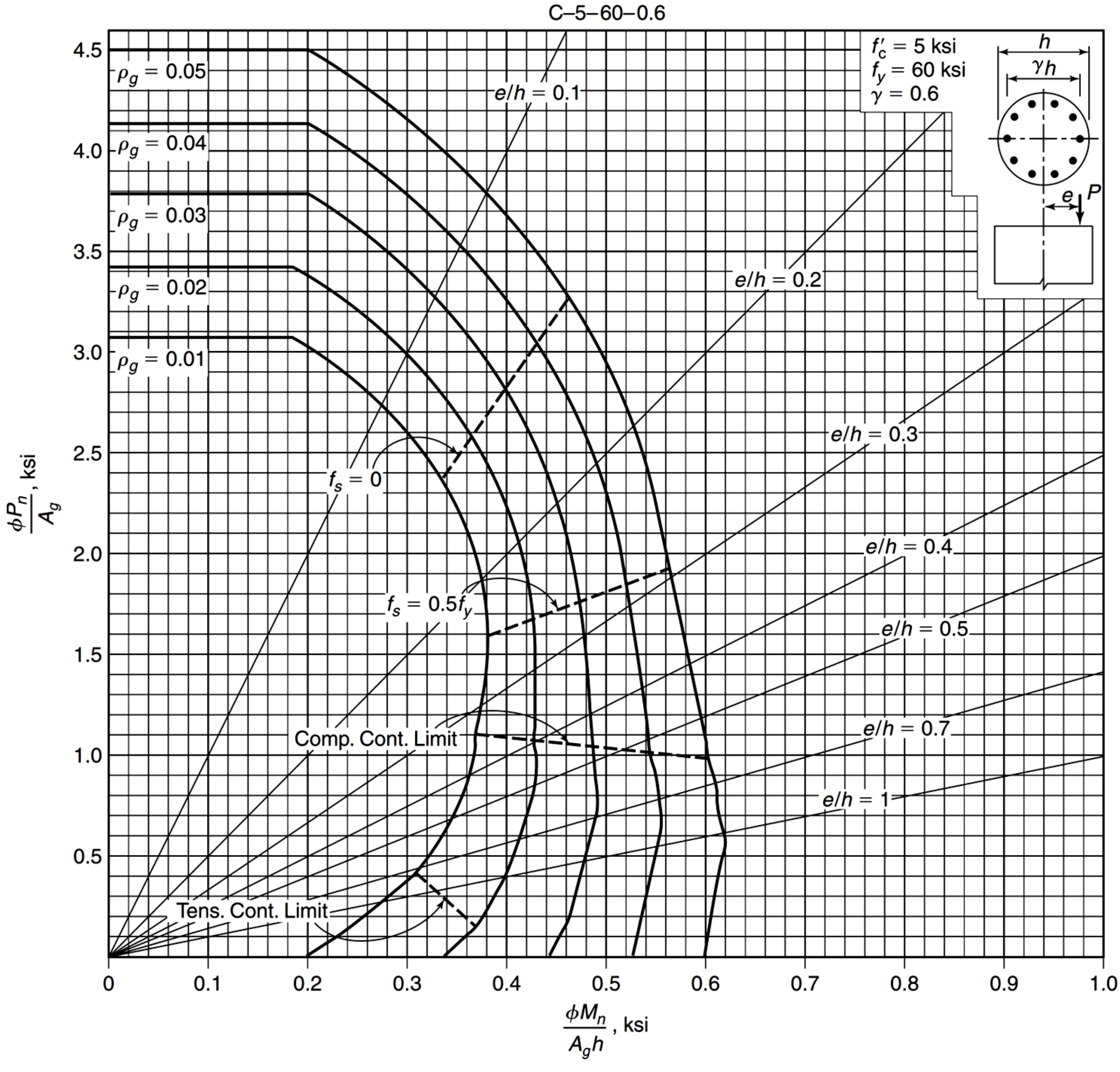


Fig. A-13a
Nondimensional interaction diagram for circular spiral column: $f'_c = 5000$ psi and $\gamma = 0.60$.

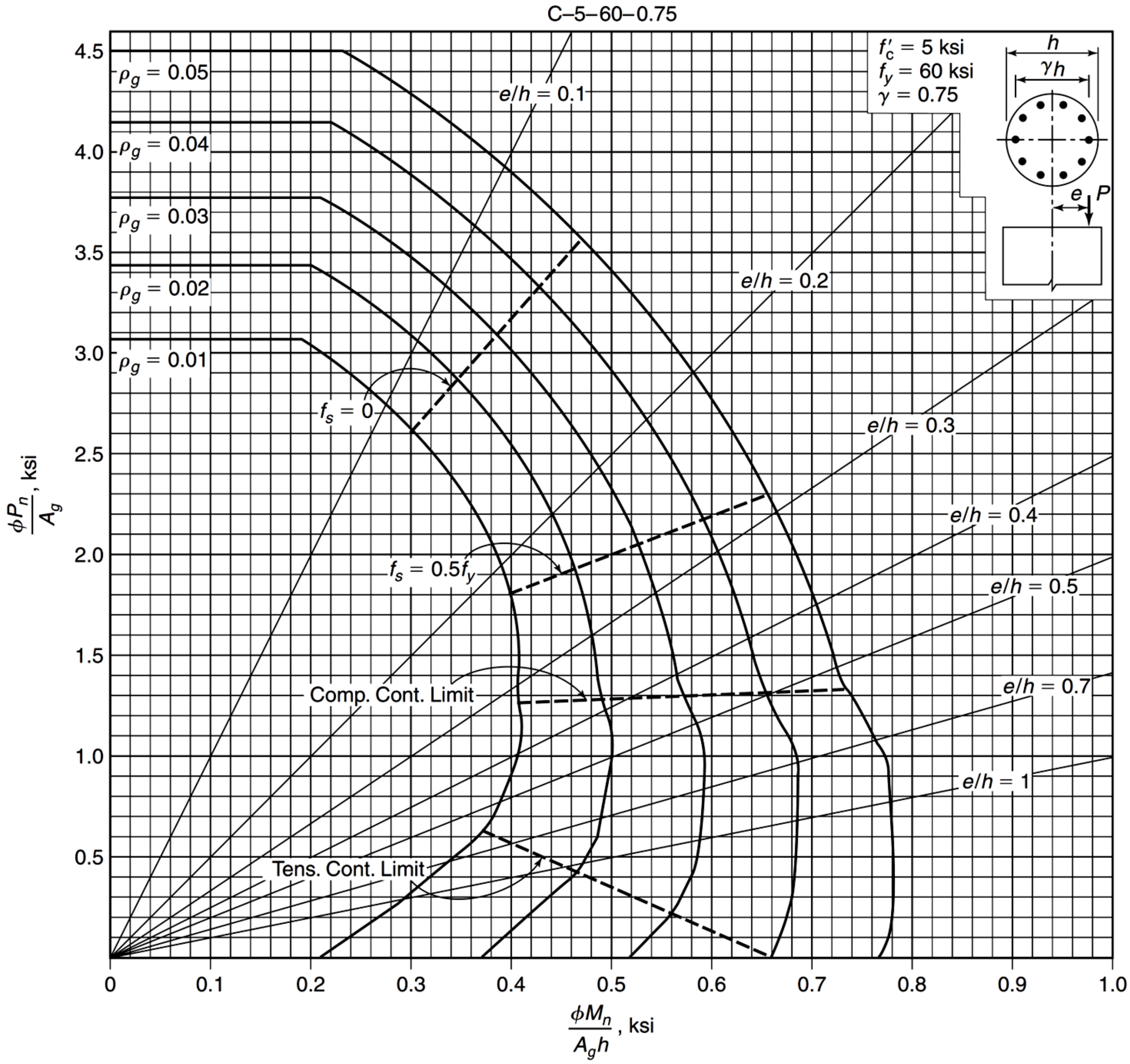


Fig. A-13b
Nondimensional interaction diagram for circular spiral column: $f'_c = 5000$ psi and $\gamma = 0.75$.

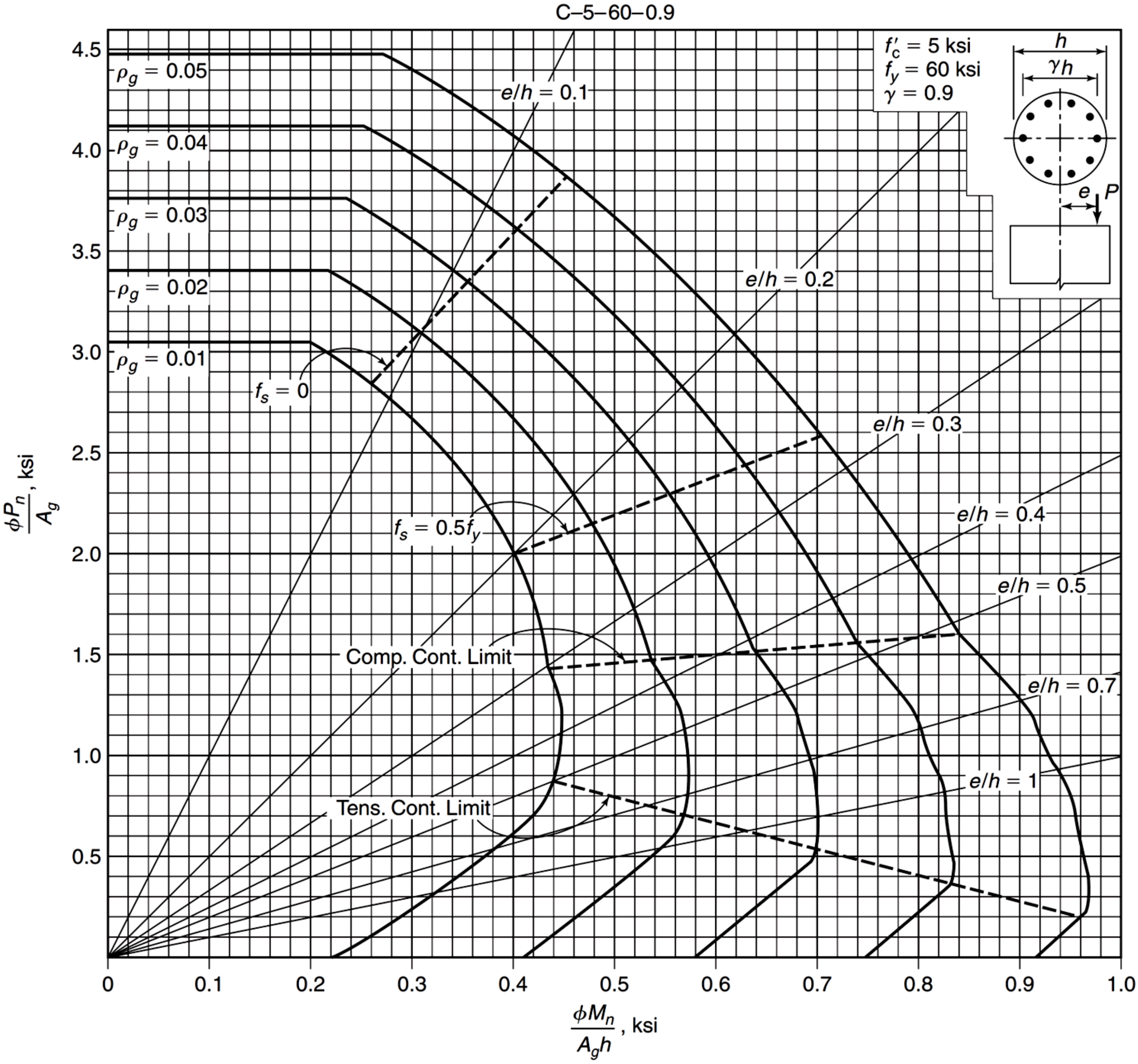


Fig. A-13c
Nondimensional interaction diagram for circular spiral column: $f'_c = 5000$ psi and $\gamma = 0.90$.

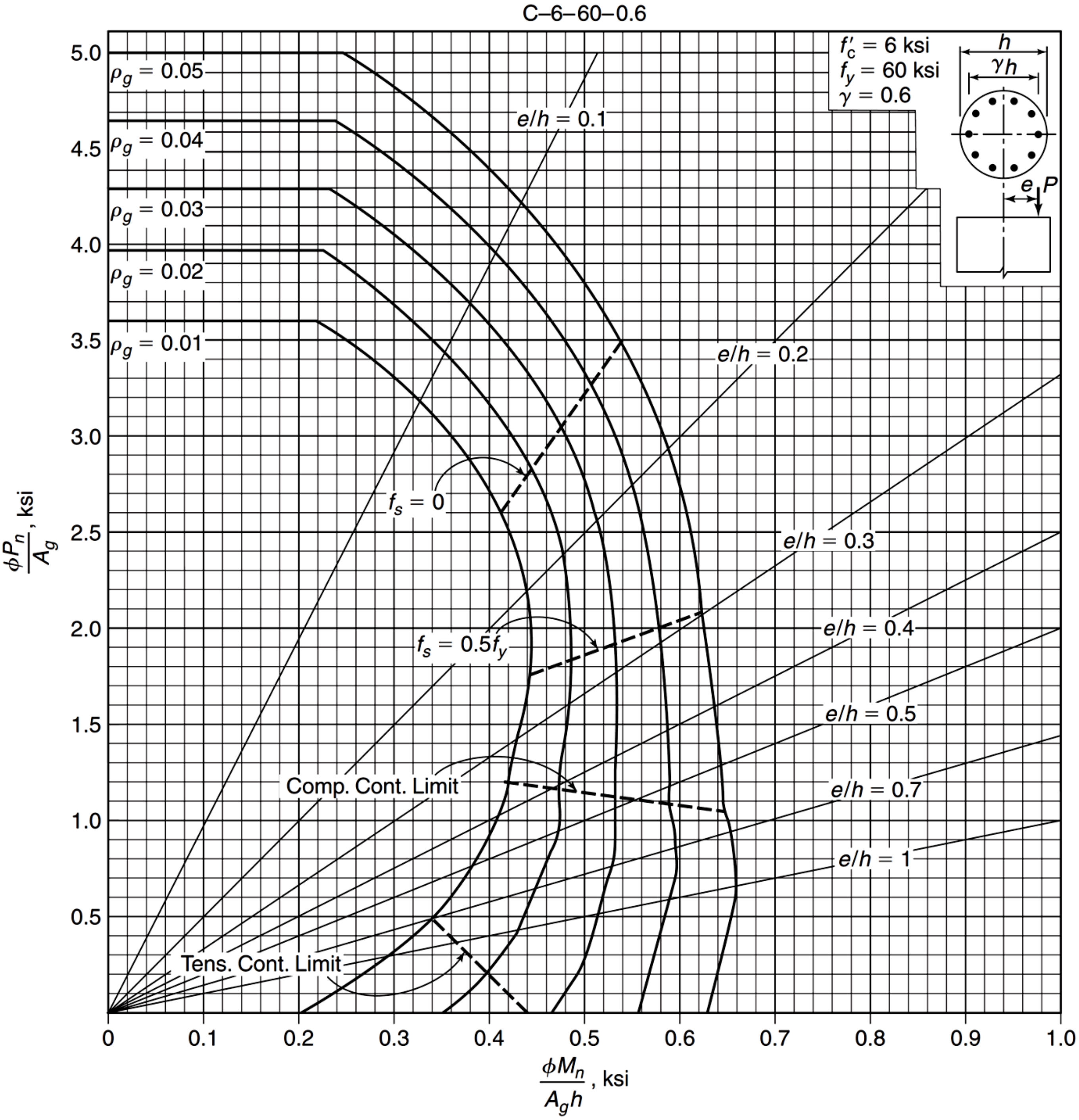


Fig. A-14a
Nondimensional interaction diagram for circular spiral column: $f'_c = 6000$ psi and $\gamma = 0.60$.

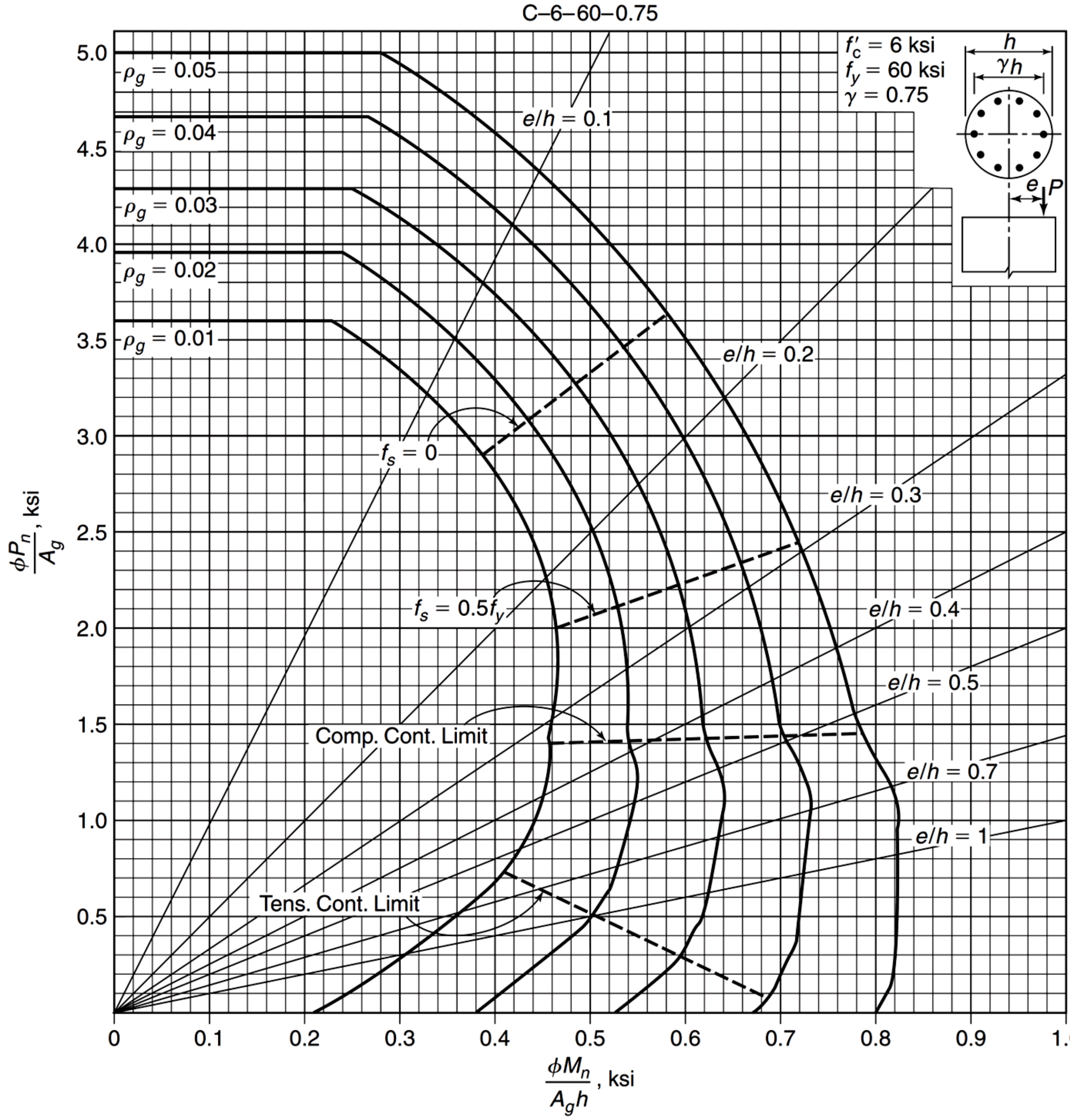


Fig. A-14b
Nondimensional interaction diagram for circular spiral column: $f'_c = 6000$ psi and $\gamma = 0.75$.

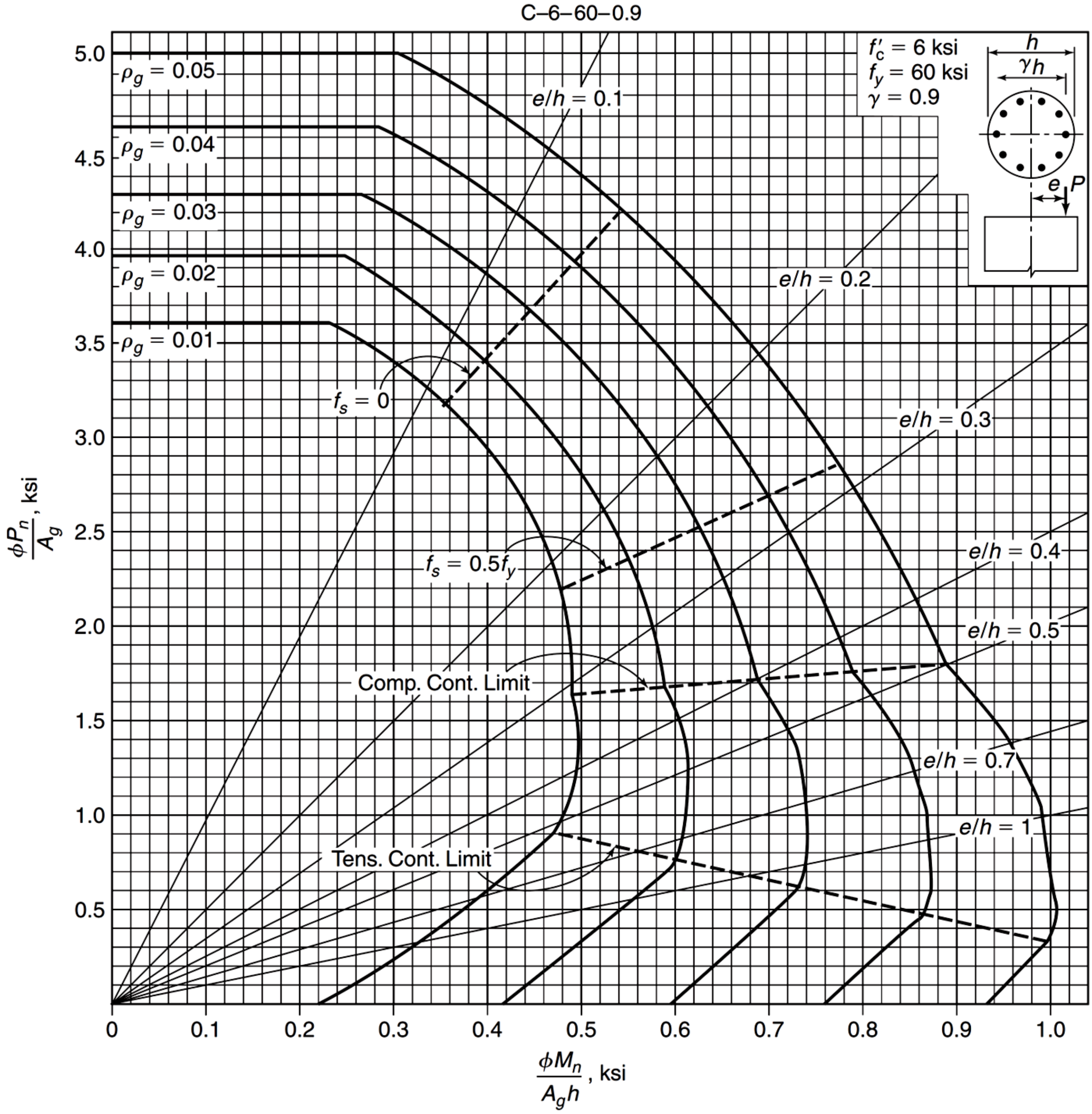


Fig. A-14c
Nondimensional interaction diagram for circular spiral column: $f'_c = 6000$ psi and $\gamma = 0.90$.

Compression members:

Columns	$0.70\ I_g$
Walls – Uncracked	$0.70\ I_g$
– Cracked	$0.35\ I_g$

Flexural members:

Beams	$0.35\ I_g$
Flat plates and flat slabs	$0.25\ I_g$

Compression members:

$$I = \left(0.8 + 25\frac{A_{st}}{A_g}\right)\left(1 - \frac{M_u}{P_u h} - 0.5\frac{P_u}{P_o}\right) I_g \le 0.875\ I_g$$

I need not be taken less than $0.35 I_g$.

Flexural members:

$$I = (0.1 + 25\rho)\left(1.2 - 0.2\frac{b_w}{d}\right) I_g \le 0.5\ I_g$$

$$\frac{kl_u}{r} \le 34 - 12\left(\frac{M_1}{M_2}\right) \le 40, \qquad \frac{kl_u}{r} \le 22$$

$$\beta_{dns} = \frac{1.2\ D\ (sustained)}{1.2\ D + 1.6\ L} \le 1 \qquad \beta_{ds} = \frac{maximum\ factored\ sustained\ shear\ in\ the\ story}{total\ factored\ shear\ in\ the\ story}$$

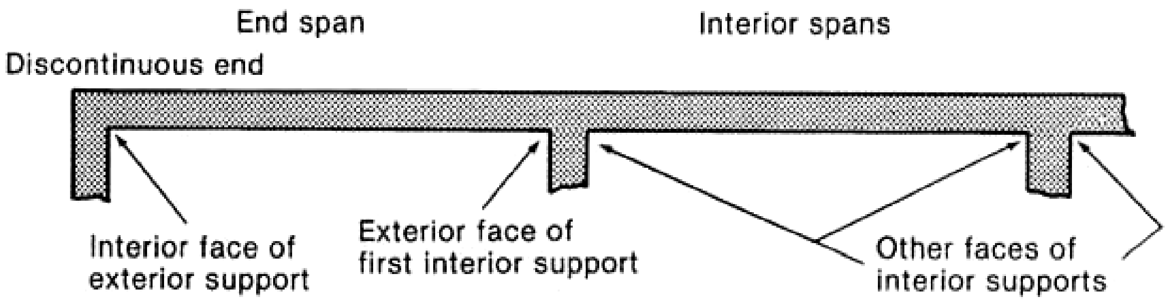
$$C_m = 0.6 + 0.4\left(\frac{M_1}{M_2}\right) \ge 0.4 \qquad \frac{l_u}{r} > \frac{35}{\sqrt{\dfrac{P_u}{f_c' A_g}}}$$

$$\delta_{ns} = \frac{C_m}{1 - \dfrac{P_u}{0.75 P_c}} \ge 1.0, \qquad \delta_s = \frac{1}{1 - Q} \ge 1.0, \qquad \delta_s = \frac{1}{1 - \dfrac{\sum P_u}{0.75 \sum P_c}} \ge 1.0$$

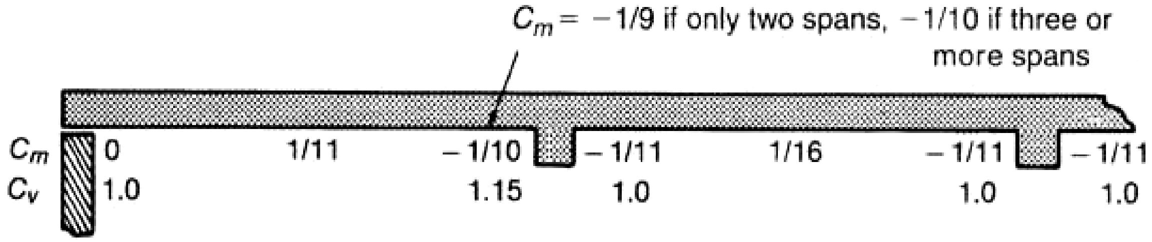
One-way Slabs

$$M_u = C_m(w_u l_n^2), \qquad V_u = C_v\left(\frac{w_u l_n}{2}\right)$$

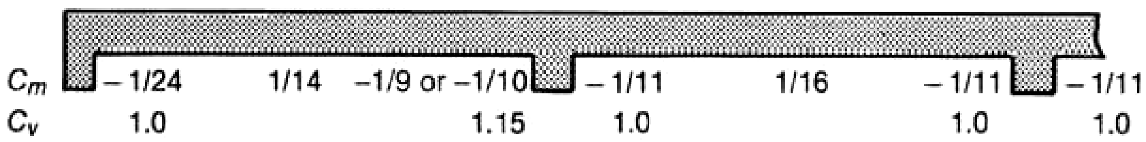
$$M_n = 0.42\sqrt{f_c'} S_m, \qquad S_m = \frac{bh^2}{6}$$



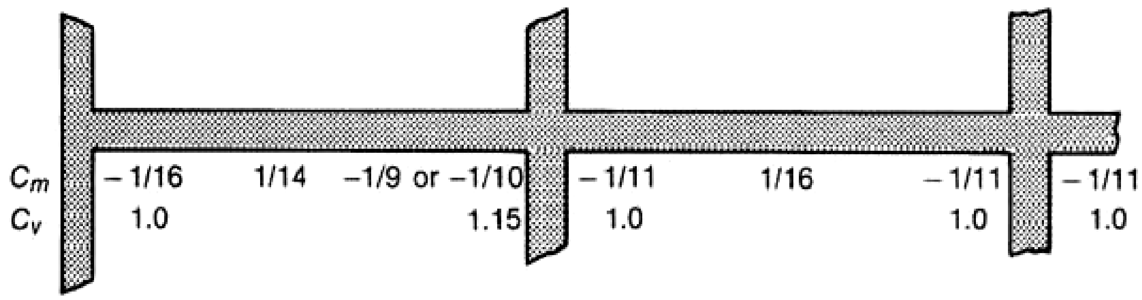
(a) Terminology.



(b) Moment and shear coefficients—Discontinuous end unrestrained.



(c) Moment and shear coefficients—Discontinuous end integral with support where support is a spandrel girder.



(d) Moment and shear coefficients—Discontinuous end integral with support where support is a column.

TABLE 9.5(a) — MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED

	Minimum thickness, ***h***			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one-way slabs	**ℓ/20**	**ℓ/24**	**ℓ/28**	**ℓ/10**
Beams or ribbed one-way slabs	**ℓ/16**	**ℓ/18.5**	**ℓ/21**	**ℓ/8**

Notes:
Values given shall be used directly for members with normalweight concrete and Grade 420 reinforcement. For other conditions, the values shall be modified as follows:
a) For lightweight concrete having equilibrium density, w_c, in the range of 1440 to 1840 kg/m^3, the values shall be multiplied by **(1.65 – 0.0003w_c)** but not less than 1.09.
b) For f_y other than 420 MPa, the values shall be multiplied by **(0.4 + f_y/700)**.

Two-way Slabs

$$\alpha_f = \frac{E_{cb} I_b}{E_{cs} I_s},$$

1. For $0.2 \leq \alpha_{fm} \leq 2$

$$h = \frac{l_n\left(0.8 + \frac{f_y}{1400}\right)}{36 + 5\beta\left(\alpha_{fm} - 0.2\right)} \geq 125\ mm$$

2. For $\alpha_{fm} > 2$

$$h = \frac{l_n\left(0.8 + \frac{f_y}{1400}\right)}{36 + 9\beta} \geq 90\ mm$$

3. For $\alpha_{fm} < 0.2$ $\quad h = minimum\ slab\ thickness\ without\ interior\ beams\ (Table\ 9.5(c))$

$$M_a = C_a w l_a^2, \qquad M_b = C_b w l_b^2 \qquad M_{a,rib} = C_a w l_a^2 b_f, \qquad M_b = C_b w l_b^2 b_f$$

$$V_{ud} = w_u\left(\frac{l_2}{2} - \frac{b_w}{2} - d\right)$$

$$Punching\ stirrups \qquad V_n = \frac{1}{2}\sqrt{f_c'}\, b_0 d, \qquad V_c = \frac{1}{6}\sqrt{f_c'}\, b_0 d$$

$$Punching\ studs \qquad V_n = \frac{2}{3}\sqrt{f_c'}\, b_0 d \qquad V_c = \frac{1}{4}\lambda\sqrt{f_c'}\, b_0 d \qquad v_u = \frac{V_u}{b_0 \times d} \leq \frac{1}{6}\phi\lambda\sqrt{f_c'}$$

Punching Shear (Slabs and Footings):

$$V_c = \frac{1}{6}\left(1 + \frac{2}{\beta}\right)\lambda\sqrt{f_c'}\, b_0 d, \qquad V_c = \frac{1}{12}\left(\frac{\alpha_s d}{b_0} + 2\right)\lambda\sqrt{f_c'}\, b_0 d, \qquad V_c = \frac{1}{3}\lambda\sqrt{f_c'}\, b_0 d$$

TABLE 9.5(c)—MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS*

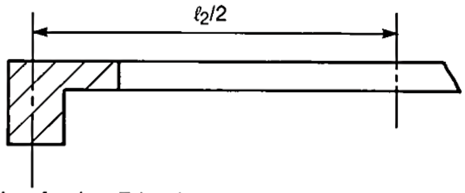
	Without drop panels‡			With drop panels‡		
	Exterior panels		Interior panels	Exterior panels		Interior panels
f_y, MPa†	Without edge beams	With edge beams§		Without edge beams	With edge beams§	
280	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
420	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
520	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$	$\ell_n/31$	$\ell_n/34$	$\ell_n/34$

*For two-way construction, ℓ_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.

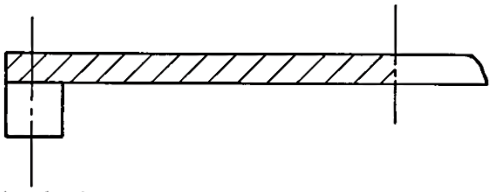
†For f_y between the values given in the table, minimum thickness shall be determined by linear interpolation.

‡Drop panels as defined in 13.2.5.

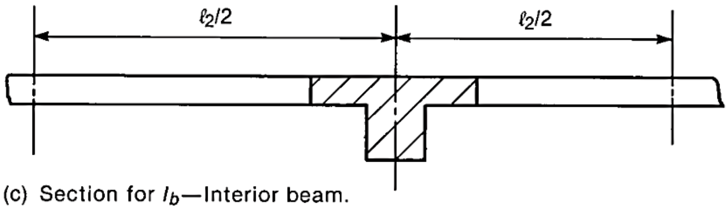
§Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall not be less than 0.8.



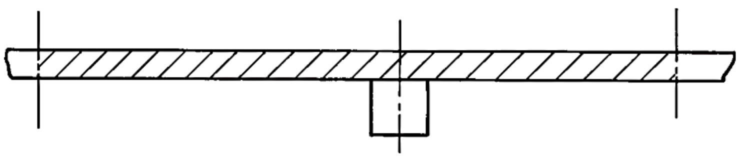
(a) Section for I_b—Edge beam.



(b) Section for I_s—Edge beam.

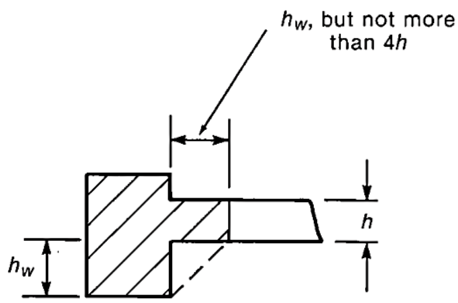


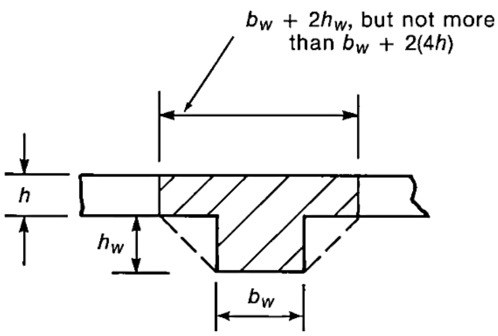
(c) Section for I_b—Interior beam.



(d) Section for I_s—Interior beam.

Beam and slab sections for calculations of α_f



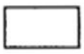
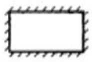
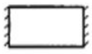
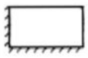
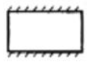
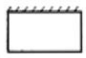
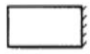
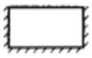
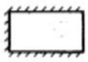


Cross section of beams as defined in ACI Code Section 13.2.4.

TABLE 1

Coefficients for negative moments in slabs[a]

$M_{a,neg} = C_{a,neg} w l_a^2$
$M_{b,neg} = C_{b,neg} w l_b^2$ where w = total uniform dead plus live load

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
										
1.00	$C_{a,neg}$		0.045		0.050	0.075	0.071		0.033	0.061
	$C_{b,neg}$		0.045	0.076	0.050			0.071	0.061	0.033
0.95	$C_{a,neg}$		0.050		0.055	0.079	0.075		0.038	0.065
	$C_{b,neg}$		0.041	0.072	0.045			0.067	0.056	0.029
0.90	$C_{a,neg}$		0.055		0.060	0.080	0.079		0.043	0.068
	$C_{b,neg}$		0.037	0.070	0.040			0.062	0.052	0.025
0.85	$C_{a,neg}$		0.060		0.066	0.082	0.083		0.049	0.072
	$C_{b,neg}$		0.031	0.065	0.034			0.057	0.046	0.021
0.80	$C_{a,neg}$		0.065		0.071	0.083	0.086		0.055	0.075
	$C_{b,neg}$		0.027	0.061	0.029			0.051	0.041	0.017
0.75	$C_{a,neg}$		0.069		0.076	0.085	0.088		0.061	0.078
	$C_{b,neg}$		0.022	0.056	0.024			0.044	0.036	0.014
0.70	$C_{a,neg}$		0.074		0.081	0.086	0.091		0.068	0.081
	$C_{b,neg}$		0.017	0.050	0.019			0.038	0.029	0.011
0.65	$C_{a,neg}$		0.077		0.085	0.087	0.093		0.074	0.083
	$C_{b,neg}$		0.014	0.043	0.015			0.031	0.024	0.008
0.60	$C_{a,neg}$		0.081		0.089	0.088	0.095		0.080	0.085
	$C_{b,neg}$		0.010	0.035	0.011			0.024	0.018	0.006
0.55	$C_{a,neg}$		0.084		0.092	0.089	0.096		0.085	0.086
	$C_{b,neg}$		0.007	0.028	0.008			0.019	0.014	0.005
0.50	$C_{a,neg}$		0.086		0.094	0.090	0.097		0.089	0.088
	$C_{b,neg}$		0.006	0.022	0.006			0.014	0.010	0.003

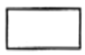
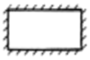
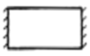
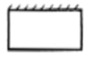
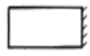
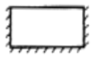
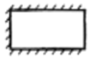
[a] A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

TABLE 2

Coefficients for dead load positive moments in slabs[a]

$$M_{a,pos,dl} = C_{a,dl} w l_a^2$$
$$M_{b,pos,dl} = C_{b,dl} w l_b^2$$

where w = total uniform dead load

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
										
1.00	$C_{a,dl}$	0.036	0.018	0.018	0.027	0.027	0.033	0.027	0.020	0.023
	$C_{b,dl}$	0.036	0.018	0.027	0.027	0.018	0.027	0.033	0.023	0.020
0.95	$C_{a,dl}$	0.040	0.020	0.021	0.030	0.028	0.036	0.031	0.022	0.024
	$C_{b,dl}$	0.033	0.016	0.025	0.024	0.015	0.024	0.031	0.021	0.017
0.90	$C_{a,dl}$	0.045	0.022	0.025	0.033	0.029	0.039	0.035	0.025	0.026
	$C_{b,dl}$	0.029	0.014	0.024	0.022	0.013	0.021	0.028	0.019	0.015
0.85	$C_{a,dl}$	0.050	0.024	0.029	0.036	0.031	0.042	0.040	0.029	0.028
	$C_{b,dl}$	0.026	0.012	0.022	0.019	0.011	0.017	0.025	0.017	0.013
0.80	$C_{a,dl}$	0.056	0.026	0.034	0.039	0.032	0.045	0.045	0.032	0.029
	$C_{b,dl}$	0.023	0.011	0.020	0.016	0.009	0.015	0.022	0.015	0.010
0.75	$C_{a,dl}$	0.061	0.028	0.040	0.043	0.033	0.048	0.051	0.036	0.031
	$C_{b,dl}$	0.019	0.009	0.018	0.013	0.007	0.012	0.020	0.013	0.007
0.70	$C_{a,dl}$	0.068	0.030	0.046	0.046	0.035	0.051	0.058	0.040	0.033
	$C_{b,dl}$	0.016	0.007	0.016	0.011	0.005	0.009	0.017	0.011	0.006
0.65	$C_{a,dl}$	0.074	0.032	0.054	0.050	0.036	0.054	0.065	0.044	0.034
	$C_{b,dl}$	0.013	0.006	0.014	0.009	0.004	0.007	0.014	0.009	0.005
0.60	$C_{a,dl}$	0.081	0.034	0.062	0.053	0.037	0.056	0.073	0.048	0.036
	$C_{b,dl}$	0.010	0.004	0.011	0.007	0.003	0.006	0.012	0.007	0.004
0.55	$C_{a,dl}$	0.088	0.035	0.071	0.056	0.038	0.058	0.081	0.052	0.037
	$C_{b,dl}$	0.008	0.003	0.009	0.005	0.002	0.004	0.009	0.005	0.003
0.50	$C_{a,dl}$	0.095	0.037	0.080	0.059	0.039	0.061	0.089	0.056	0.038
	$C_{b,dl}$	0.006	0.002	0.007	0.004	0.001	0.003	0.007	0.004	0.002

[a] A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

TABLE 3
Coefficients for live load positive moments in slabs[a]

$$M_{a,pos,ll} = C_{a,ll} w l_a^2$$
$$M_{b,pos,ll} = C_{b,ll} w l_b^2$$
where w = total uniform live load

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{a,ll}$	0.036	0.027	0.027	0.032	0.032	0.035	0.032	0.028	0.030
	$C_{b,ll}$	0.036	0.027	0.032	0.032	0.027	0.032	0.035	0.030	0.028
0.95	$C_{a,ll}$	0.040	0.030	0.031	0.035	0.034	0.038	0.036	0.031	0.032
	$C_{b,ll}$	0.033	0.025	0.029	0.029	0.024	0.029	0.032	0.027	0.025
0.90	$C_{a,ll}$	0.045	0.034	0.035	0.039	0.037	0.042	0.040	0.035	0.036
	$C_{b,ll}$	0.029	0.022	0.027	0.026	0.021	0.025	0.029	0.024	0.022
0.85	$C_{a,ll}$	0.050	0.037	0.040	0.043	0.041	0.046	0.045	0.040	0.039
	$C_{b,ll}$	0.026	0.019	0.024	0.023	0.019	0.022	0.026	0.022	0.020
0.80	$C_{a,ll}$	0.056	0.041	0.045	0.048	0.044	0.051	0.051	0.044	0.042
	$C_{b,ll}$	0.023	0.017	0.022	0.020	0.016	0.019	0.023	0.019	0.017
0.75	$C_{a,ll}$	0.061	0.045	0.051	0.052	0.047	0.055	0.056	0.049	0.046
	$C_{b,ll}$	0.019	0.014	0.019	0.016	0.013	0.016	0.020	0.016	0.013
0.70	$C_{a,ll}$	0.068	0.049	0.057	0.057	0.051	0.060	0.063	0.054	0.050
	$C_{b,ll}$	0.016	0.012	0.016	0.014	0.011	0.013	0.017	0.014	0.011
0.65	$C_{a,ll}$	0.074	0.053	0.064	0.062	0.055	0.064	0.070	0.059	0.054
	$C_{b,ll}$	0.013	0.010	0.014	0.011	0.009	0.010	0.014	0.011	0.009
0.60	$C_{a,ll}$	0.081	0.058	0.071	0.067	0.059	0.068	0.077	0.065	0.059
	$C_{b,ll}$	0.010	0.007	0.011	0.009	0.007	0.008	0.011	0.009	0.007
0.55	$C_{a,ll}$	0.088	0.062	0.080	0.072	0.063	0.073	0.085	0.070	0.063
	$C_{b,ll}$	0.008	0.006	0.009	0.007	0.005	0.006	0.009	0.007	0.006
0.50	$C_{a,ll}$	0.095	0.066	0.088	0.077	0.067	0.078	0.092	0.076	0.067
	$C_{b,ll}$	0.006	0.004	0.007	0.005	0.004	0.005	0.007	0.005	0.004

[a] A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

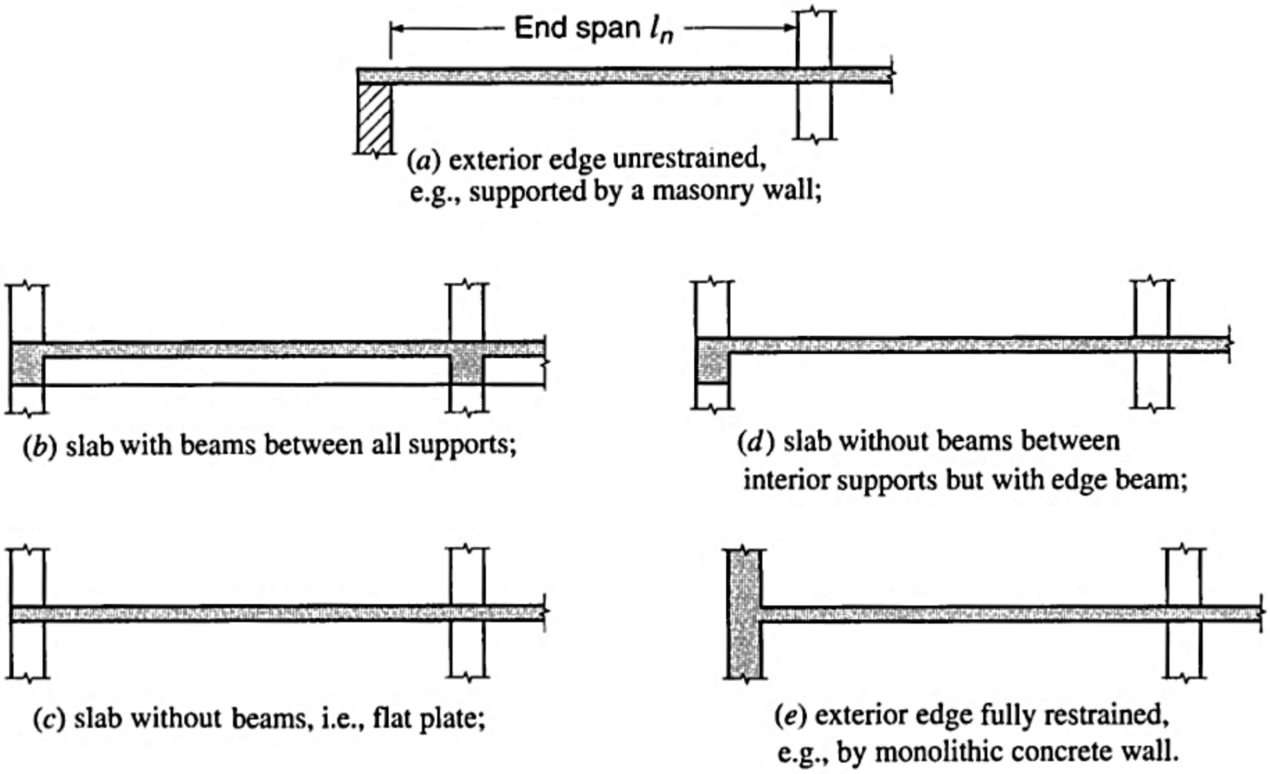
TABLE 4

Ratio of load W in l_a and l_b directions for shear in slab and load on supports[a]

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	W_a	0.50	0.50	0.17	0.50	0.83	0.71	0.29	0.33	0.67
	W_b	0.50	0.50	0.83	0.50	0.17	0.29	0.71	0.67	0.33
0.95	W_a	0.55	0.55	0.20	0.55	0.86	0.75	0.33	0.38	0.71
	W_b	0.45	0.45	0.80	0.45	0.14	0.25	0.67	0.62	0.29
0.90	W_a	0.60	0.60	0.23	0.60	0.88	0.79	0.38	0.43	0.75
	W_b	0.40	0.40	0.77	0.40	0.12	0.21	0.62	0.57	0.25
0.85	W_a	0.66	0.66	0.28	0.66	0.90	0.83	0.43	0.49	0.79
	W_b	0.34	0.34	0.72	0.34	0.10	0.17	0.57	0.51	0.21
0.80	W_a	0.71	0.71	0.33	0.71	0.92	0.86	0.49	0.55	0.83
	W_b	0.29	0.29	0.67	0.29	0.08	0.14	0.51	0.45	0.17
0.75	W_a	0.76	0.76	0.39	0.76	0.94	0.88	0.56	0.61	0.86
	W_b	0.24	0.24	0.61	0.24	0.06	0.12	0.44	0.39	0.14
0.70	W_a	0.81	0.81	0.45	0.81	0.95	0.91	0.62	0.68	0.89
	W_b	0.19	0.19	0.55	0.19	0.05	0.09	0.38	0.32	0.11
0.65	W_a	0.85	0.85	0.53	0.85	0.96	0.93	0.69	0.74	0.92
	W_b	0.15	0.15	0.47	0.15	0.04	0.07	0.31	0.26	0.08
0.60	W_a	0.89	0.89	0.61	0.89	0.97	0.95	0.76	0.80	0.94
	W_b	0.11	0.11	0.39	0.11	0.03	0.05	0.24	0.20	0.06
0.55	W_a	0.92	0.92	0.69	0.92	0.98	0.96	0.81	0.85	0.95
	W_b	0.08	0.08	0.31	0.08	0.02	0.04	0.19	0.15	0.05
0.50	W_a	0.94	0.94	0.76	0.94	0.99	0.97	0.86	0.89	0.97
	W_b	0.06	0.06	0.24	0.06	0.01	0.03	0.14	0.11	0.03

[a] A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

$$\alpha_f = \frac{E_{cb} I_b}{E_{cs} I_s}, \qquad \beta_t = \frac{E_{cb} C}{2E_{cs} I_s} \qquad M_o = \frac{w_u l_2 l_n^2}{8}, \qquad C = \sum \left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3}$$



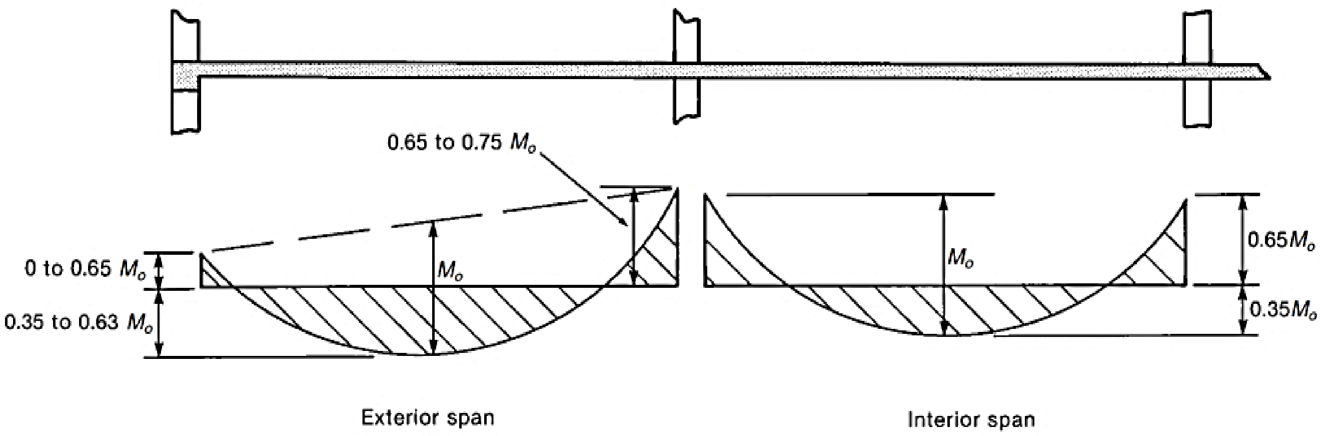
Conditions of edge restraint considered in distributing total static moment M_o to critical sections in an end span.

Distribution factors applied to static moment M_o for positive and negative moments in end span

	(a)	(b)	(c)	(d)	(e)
	Exterior Edge Unrestrained	Slab with Beams between All Supports	Slab without Beams between Interior Supports		Exterior Edge Fully Restrained
			Without Edge Beam	With Edge Beam	
Interior negative moment	0.75	0.70	0.70	0.70	0.65
Positive moment	0.63	0.57	0.52	0.50	0.35
Exterior negative moment	0	0.16	0.26	0.30	0.65

Column-strip moment, percent of total moment at critical section

		l_2/l_1 0.5	1.0	2.0
Interior negative moment				
$\alpha_{f1}l_2/l_1 = 0$		75	75	75
$\alpha_{f1}l_2/l_1 \geq 1.0$		90	75	45
Exterior negative moment				
$\alpha_{f1}l_2/l_1 = 0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	75	75	75
$\alpha_{f1}l_2/l_1 \geq 1.0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	90	75	45
Positive moment				
$\alpha_{f1}l_2/l_1 = 0$		60	60	60
$\alpha_{f1}l_2/l_1 \geq 1.0$		90	75	45



Assignment of positive- and negative-moment regions.

Footings and Foundations

$$\gamma_s = \frac{2}{\beta + 1} \qquad \beta = \frac{l}{b}$$

$$\phi(\,0.85 f_c' A_1) \qquad \phi(0.85 f_c' A_1)\sqrt{\frac{A_2}{A_1}} \qquad A_{dowels} = \frac{P_u - \phi P_{nb}}{\phi f_y} \geq 0.005 A_g$$

1. When $e = M/P < l/6$, the soil pressure is trapezoidal.

$$q_{max} = \frac{P}{A} + \frac{M}{I}c = \frac{P}{Bl} + \frac{6M}{Bl^2}, \qquad q_{min} = \frac{P}{A} - \frac{M}{I}c = \frac{P}{Bl} - \frac{6M}{Bl^2}$$

2. When $e = M/P = l/6$, the soil pressure is triangular.

$$q_{max} = \frac{P}{Bl} + \frac{6M}{Bl^2} = \frac{2P}{Bl}, \qquad q_{min} = 0 = \frac{P}{Bl} - \frac{6M}{Bl^2} \quad or \quad \frac{P}{Bl} = \frac{6M}{Bl^2}$$

3. When $e = M/P > l/6$, the soil pressure is triangular.

$$x = \frac{l}{2} - e \qquad P = q_{max}\left(\frac{3x}{2}\right)B, \qquad q_{max} = \frac{2P}{3xB} = \frac{2P}{3B\left(\frac{l}{2} - e\right)}$$

$$q = \frac{P}{A} + \frac{M_x}{I_x}y + \frac{M_y}{I_y}x \le q_{allow,net}$$

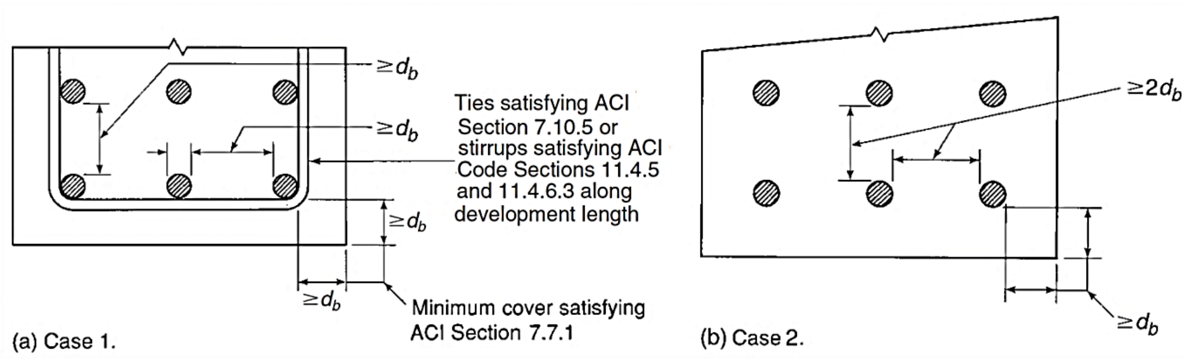
Development, Anchorage, and Splicing of Reinforcement

Tension-Development Lengths

$$l_d = \frac{9}{10} \times \frac{f_y}{\lambda\sqrt{f_c'}} \times \frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)} d_b \qquad (*)$$

	∅ 20 and Smaller Bars and Deformed Wires	∅ 22 and Larger Bars
Case 1: Clear spacing of bars being developed or spliced not less than d_b, **and** stirrups or ties throughout l_d not less than the code minimum **or** **Case 2:** Clear spacing of bars being developed or spliced not less than $2d_b$ and clear cover not less than d_b	$l_d = \frac{12}{25} \times \frac{f_y\psi_t\psi_e}{\lambda\sqrt{f_c'}} d_b$ (**)	$l_d = \frac{12}{20} \times \frac{f_y\psi_t\psi_e}{\lambda\sqrt{f_c'}} d_b$ (***)
Other cases	$l_d = \frac{18}{25} \times \frac{f_y\psi_t\psi_e}{\lambda\sqrt{f_c'}} d_b$ (****)	$l_d = \frac{18}{20} \times \frac{f_y\psi_t\psi_e}{\lambda\sqrt{f_c'}} d_b$ (*****)

- The length l_d computed using these equations shall not be taken less than 300 mm.



Factors in Equations ($*$) through ($*****$)

ψ_t – **bar location factor**

Horizontal reinforcement so placed that more than $300\,mm$ of fresh concrete is cast in the member below the development length or splice... 1.3

Other reinforcement... 1.0

ψ_e – **coating factor**

Epoxy-coated bars or wires with cover less than $3d_b$ or clear spacing less than $6d_b$.. 1.5

All other epoxy-coated bars or wires.. 1.2

Uncoated and galvanized reinforcement 1.0

The product of $(\psi_t \cdot \psi_e)$ need not be taken greater than 1.7.

ψ_s – **bar-size factor**

$\varnothing\,20$ and smaller bars and deformed wires 0.8

$\varnothing\,22$ and larger bars.. 1.0

λ – **lightweight-aggregate-concrete factor**

When any lightweight-aggregate concrete is used 0.75

However, when the splitting tensile strength f_{ct} is specified, λ shall be permitted to be taken as $f_{ct}/0.56\sqrt{f_c{}'}$ but not more than......................... 1.0

When normal-weight concrete is used 1.0

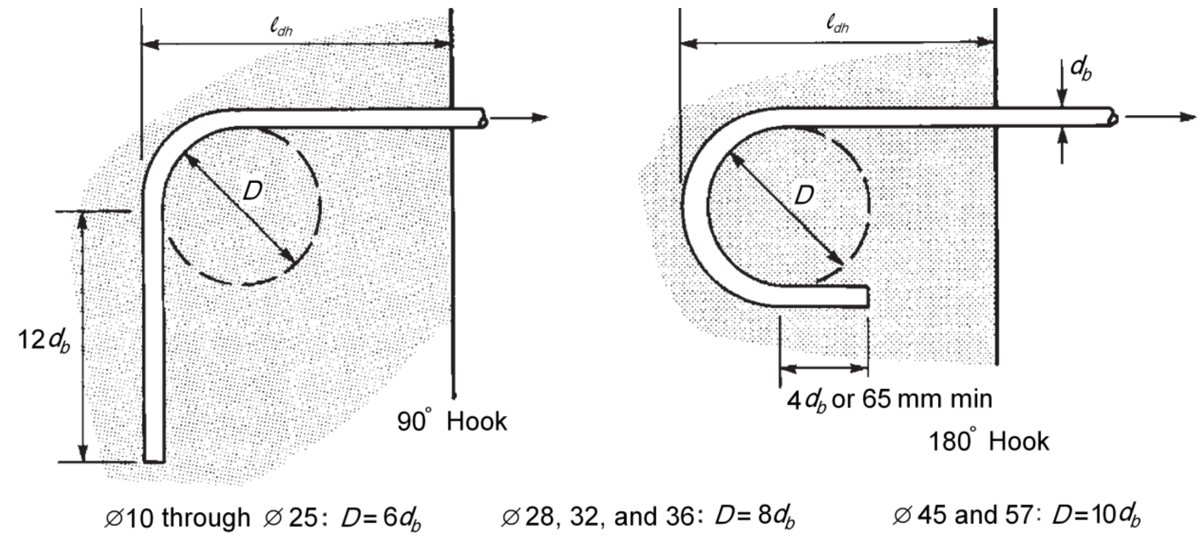
K_{tr} – **transverse reinforcement index**

$$K_{tr} = \frac{40A_{tr}}{s\,n}$$

Compression-Development Lengths

$$l_{dc} = \frac{0.24f_y}{\lambda\sqrt{f_c{}'}}d_b \qquad \textit{but not less than} \quad \geq 0.043f_y d_b$$

Hooked Anchorage



$$l_{dh} = 0.24\frac{\psi_e f_y}{\lambda\sqrt{f_c'}}d_b \times \left(\begin{matrix} applicable\ factor\ from\ ACI\ Code\ Section\ 12.5.3, \\ as\ summarized\ in\ the\ table\ below \end{matrix}\right)$$

where $\psi_e = 1.2$ — for epoxy-coated bars or wires and 1.0 for galvonized and uncoated reinforcement

	Location	Type	$d_b =$ Hooked Bar size ^{a}	Side Cover, mm	Top or Bottom Cover, mm	Tail Cover	Stirrups or ties	Factor ^{c}
1.	Anywhere, 12.5.3(a)	180°	$\leq \varnothing 36$	$\geq 65mm$	Any	Any	Not required	$\times 0.7$
2.	Anywhere, 12.5.3(a)	90°	$\leq \varnothing 36$	$\geq 65mm$	Any	$\geq 50mm$	Not required	$\times 0.7$
3.	Anywhere, 12.5.3(b)	90°	$\leq \varnothing 36$	Any	Any	Any	Enclosed in stirrups or ties perpendicular to hooked bar, spaced $\leq 3d_b$ along $l_{dh}{}^b$	$\times 0.8$ Except as in line 6
4.	Anywhere, 12.5.3(b)	90°	$\leq \varnothing 36$	Any	Any	Any	Enclosed in stirrups or ties parallel to hooked bar spaced $\leq 3d_b$ along $l_{dh}{}^b$	$\times 0.8$ Except as in line 6
5.	Anywhere, 12.5.3(c)	180°	$\leq \varnothing 36$	Any	Any	Any	Enclosed in stirrups or ties perpendicular to hooked bar, spaced $\leq 3d_b$ along $l_{dh}{}^b$	$\times 0.8$ Except as in line 6

6.	At the ends of members, 12.5.4 [d]	90 or 180°	$\leq \varnothing 36$	$\leq 65mm$	$\leq 65mm$	Any	Enclosed in stirrups or ties perpendicular to hooked bar, spaced $\leq 3{d_b}^b$	$\times 1.0$
[a] d_b is the diameter of the bar being developed by the hook								
[b] The first stirrup or tie should enclose the hook within $2d_b$ of the outside of the bend.								
[c] If two or more factors apply, l_{dh} is multiplied by the product of the factors								
[d] Line 6 (ACI Code Section 12.5.4) applies at the discontinuous ends of members								

Bar Cutoffs and Development of Bars in Flexural Members

No flexural bar shall be terminated in a tension zone unless one of the following conditions is satisfied:

1. The shear is not over two-thirds of the design strength ϕV_n.

$$V_u \leq \frac{2}{3}\phi(V_c + V_s)$$

2. Stirrups in excess of those normally required are provided over a distance along each terminated bar from the point of cutoff equal to $\frac{3}{4}d$. These "binder" stirrups shall provide an area $A_v \geq 0.41 b_w s/f_{yt}$. In addition, the stirrup spacing shall not exceed $s \leq d/8\beta_b$, where β_b is the ratio of the area of bars cut off to the total area of bars at the section.

3. The continuing bars, if $\varnothing$ 36 or smaller, provide twice the area required for flexure at that point, and the shear does not exceed three-quarters of the design strength ϕV_n

$$V_u \leq \frac{3}{4}\phi(V_c + V_s)$$

$$available\ embedment\ length\ = l_a + 1.3\frac{M_n}{V_u} \geq l_d$$

$$available\ embedment\ length\ = \begin{bmatrix} actual\ l_a, but \\ not\ exceeding\ the \\ larger\ of\ 12\ d_b\ or\ d \end{bmatrix} + \frac{M_n}{V_u} \geq l_d$$

Tension Lap Splices of Reinforcement

Class A splice: $1.0\ l_d$
Class B splice: $1.3\ l_d$

Type of Tension Lap Splices Required

$\frac{A_s \text{ Provided}}{A_s \text{ Required}}$	Maximum Percentage of A_s Spliced within Required Lap Length: 50%	100%
Two or more	Class A	Class B
Less than 2	Class B	Class B

Source: ACI Commentary Section R12.15.2.

Compression Lap Splices of Reinforcement

For bars with $f_y \leq 420\ MPa$ $\quad 0.071 f_y d_b$

For bars with $f_y > 420\ MPa$ $\quad (0.13 f_y - 24) d_b$

Column Splices

Lap Splices in Columns

12.17.2.1—Bar Stress in compression (Zone 1)	Use compression lap splice (12.16) modified by factor of 0.83 for ties or o.75 for spirals.
12.17.2.2—Bar Stress ≤ $0.5f_y$ in tension (Zone 2)	Use Class B tension lap splice (12.15) if more than 1/2 of total column bars spliced at same location. or Use Class A tension lap splice (12.15) if not more than 1/2 of total column bars splices at same location. Stagger alternate splices by ℓ_d.
12.17.2.3—Bar Stress > $0.5f_y$ in tension (Zone 3)	Use Class B tension lap splice (12.15).