AREA OF BARS (mm^2)

Size of bars (mm)	Number of bars							
	1	2	3	4	5	6	7	8
8	50	101	151	201	251	302	352	402
10	79	157	236	314	393	471	550	628
12	113	226	339	452	566	679	792	905
14	154	308	462	616	770	924	1078	1232
16	201	402	603	804	1005	1206	1407	1609
18	255	509	763	1018	1272	1527	1781	2036
20	314	628	943	1257	1571	1885	2199	2513
22	380	760	1140	1521	1901	2281	2661	3041
25	491	982	1473	1964	2454	2945	3436	3927
32	804	1609	2413	3217	4021	4826	5630	6434
50*	1964	3927	5891	7854	9818	11781	13745	15708

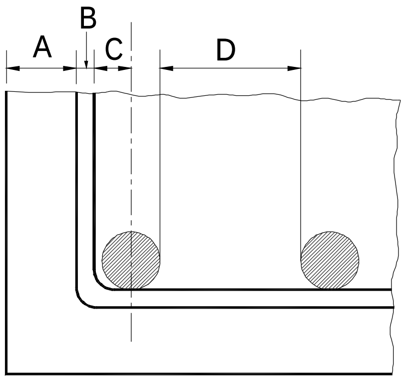
- Available through special request.

MINIMUM BEAM WIDTH (mm) ACCORDING TO THE ACI CODE

Size of Bars (mm)	Number of bars							Add for each added bar
	2	3	4	5	6	7	8	
10	175	211	246	282	317	352	388	35
12	177	215	252	290	327	364	402	37
14	179	219	258	298	337	376	416	39
16	181	223	264	306	347	388	430	41
18	183	227	270	314	357	400	444	43
20	185	231	276	322	367	412	458	45
22	187	235	282	330	377	424	472	47
25	190	241	291	342	392	442	493	50
32	204	268	332	396	460	524	588	64
50	240	340	440	540	640	740	840	100

- Table shows minimum beam widths when ϕ10 stirrups are used.
- For additional bars, add dimension in last column for each added bar.
- For bars of different sizes, determine from the table the beam width for smaller size bars and then add last column figure for each larger bar used.
- Assume maximum aggregate size does not exceed three-forth of the clear space between bars (ACI-3-3.3). Table computation procedure is in agreement with the ACI code interpretation of the ACI Committee 340.

A = 40 mm clear cover to stirrups
B = 10 mm stirrup bar diameter
C = use twice the diameter of ϕ10 stirrups.
D = clear distance between bars = d_b or 25.4 mm, whichever is greater (where d_b is the diameter of the larger adjacent longitudinal bar)



Development Length of Straight Bars and Standard Hooks

For deformed bars, ACI318-05 Section 12.2.2 defines the development length l_d given in the table below. Note that l_d shall not be less than 300 mm.

Case	$\le \phi 20$	$> \phi 20$
Case 1: Clear spacing of bars being developed not less than d_b, clear cover not less than d_b, and stirrups throughout l_d not less than code minimum **or** **Case 2:** Clear spacing of bars being developed not less than $2d_b$ and clear cover not less than d_b	$l_d = \frac{12 f_y \psi_t \psi_e \lambda}{25\sqrt{f_c^{'}}} d_b$	$l_d = \frac{12 f_y \psi_t \psi_e \lambda}{20\sqrt{f_c^{'}}} d_b$
Other cases	$l_d = \frac{18 f_y \psi_t \psi_e \lambda}{25\sqrt{f_c^{'}}} d_b$	$l_d = \frac{18 f_y \psi_t \psi_e \lambda}{20\sqrt{f_c^{'}}} d_b$

The terms in the foregoing equations are as follows:

ψ_t = **reinforcement location factor**

Horizontal reinforcement so placed that more than 300 mm of fresh concrete is cast in the member *below* the development length 1.3

Other reinforcement...1.0

ψ_e = **coating factor**

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

All other epoxy-coated bars 1.2

Uncoated reinforcement.. 1.0

λ = **lightweight aggregate concrete factor**

When all-lightweight aggregate concrete is used 1.3

When sand-lightweight aggregate concrete is used 1.2

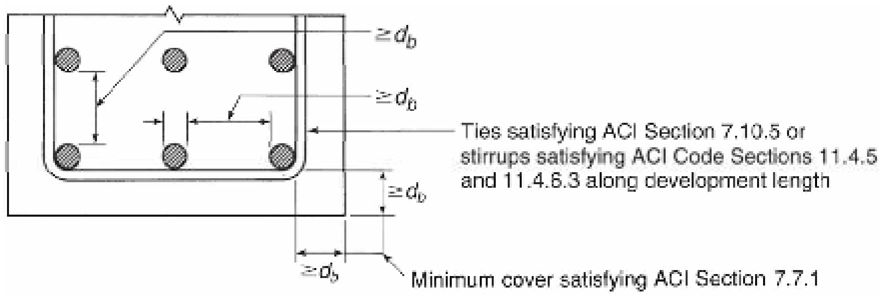
Normal weight concrete is used.. 1.0

Table 1: Basic tension development-length ratio, l_d/d_b *(mm/mm)*

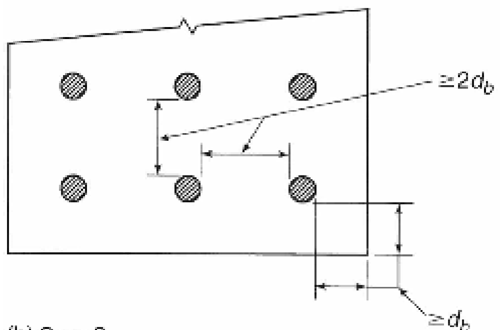
$$l_d = \frac{l_{db}}{d_b} \times y_e l \times d_b, \text{but not less than } 300 \; mm$$

Bar size (mm)	f_c' = 21 MPa		f_c' = 25 MPa		f_c' = 28 MPa		f_c' = 30 MPa		f_c' = 35 MPa	
	Bottom bar	Top bar	Bottom bar	Top bar	Bottom bar	Top bar	Bottom bar	Top bar	Bottom bar	Top bar
	Case 1: Clear spacing of bars being developed not less than db, clear cover not less than db, and stirrups throughout ld not less than code minimum, or **Case 2:** Clear spacing of bars being developed not less than 2db and clear cover not less than db; f_y = 420 MPa, uncoated bars, normal weight concrete									
£ f**20**	43.6	56.7	40.0	52.0	37.8	49.1	36.5	47.5	33.8	43.9
> f**20**	53.9	70.1	49.4	64.2	46.7	60.7	45.1	58.6	41.8	54.3
	f_y = 300 MPa, uncoated bars, normal weight concrete									
£ f**20**	31.2	40.5	28.6	37.1	27.0	35.1	26.1	33.9	24.1	31.4
	Other Cases:									
£ f**20**	64.5	83.9	59.1	76.9	55.9	72.7	54.0	70.2	50.0	65.0
> f**20**	82.1	106.8	75.3	97.9	71.1	92.5	68.7	89.3	63.6	82.7
	f_y = 300 MPa, uncoated bars, normal weight concrete									
£ f**20**	46.8	60.8	42.9	55.7	40.5	52.6	39.1	50.9	36.2	47.1

- For top bars, more than 300 mm of fresh concrete is cast in the member (i.e. $\alpha = 1.3$)
- β is the coating factor, and λ is the lightweight concrete factor



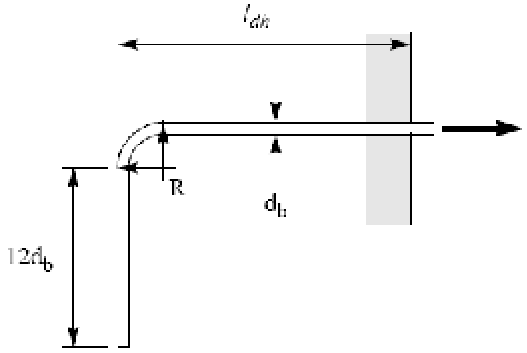
(a) Case 1.



(b) Case 2.

When there is insufficient length available to develop a straight bar, standard hooks are used. The standard 90 degree hook is shown below:

ϕ10 to ϕ25: R = $3d_b$
ϕ28 to ϕ32: R = $4d_b$
ϕ28 to ϕ50: R = $5d_b$

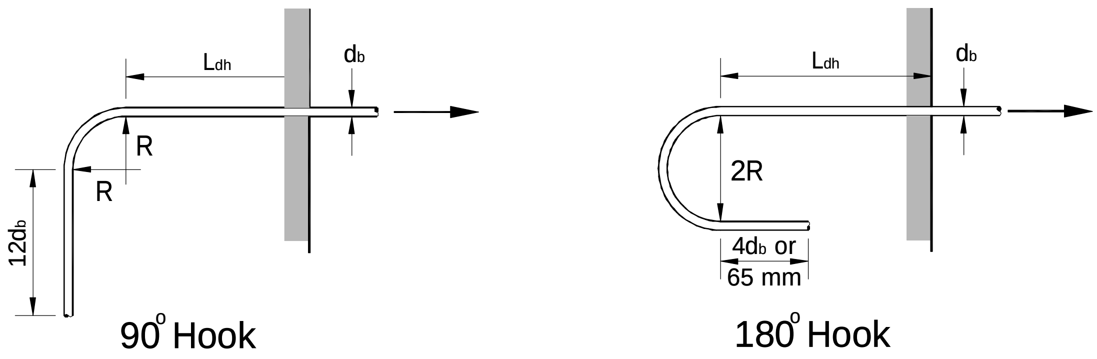


The development length of a hook, l_{dh}, is given by the following equation. Note that the development length shall not be less than 8db nor less than 150mm:

$$l_{dh} = \frac{0.24 f_y y_e l}{\sqrt{f_c'}} d_b \geq \text{larger of} \begin{vmatrix} 8d_b \\ 150mm \end{vmatrix}$$

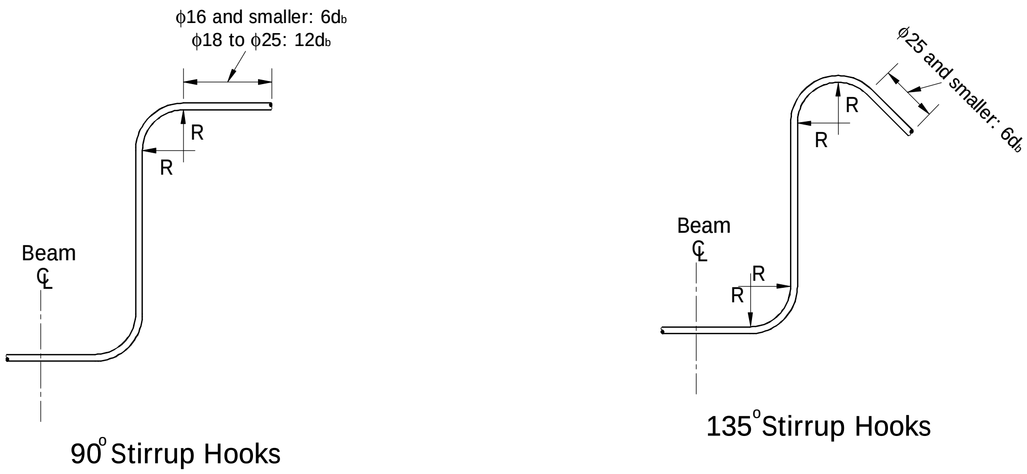
where y_e = the coating factor = 1.2 for epoxy coated bars and 1.0 for uncoated reinforcement, and λ is the lightweight aggregate factor = 1.3 for lightweight aggregate concrete. For other cases y_e and λ, shall be taken as 1.0

Standard Hooks – ACI sections 7.1 and 7.2.1



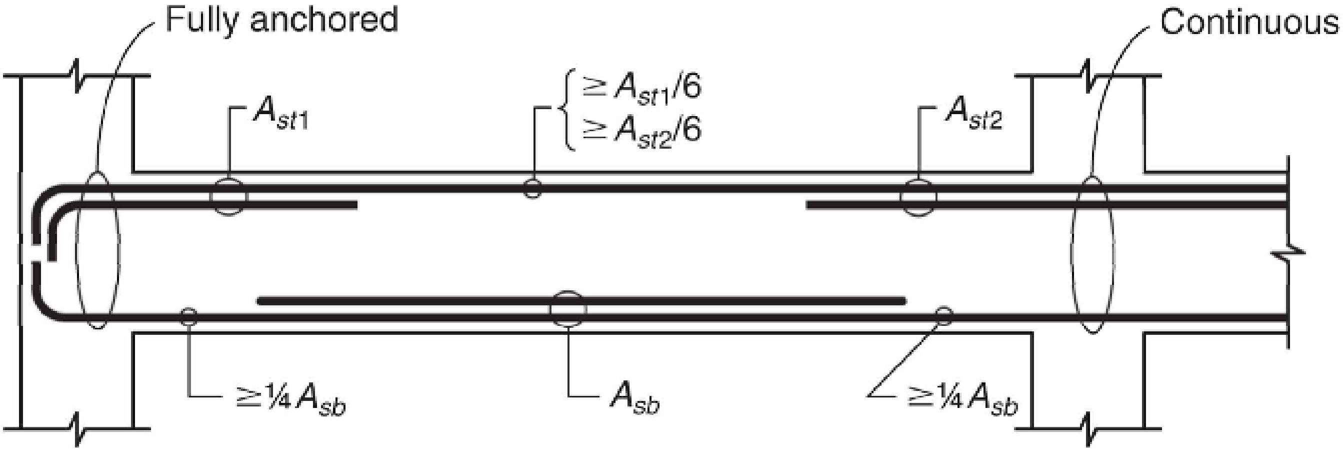
ϕ10 to ϕ25: R = $3d_b$
ϕ28 and ϕ32: R = $4d_b$
ϕ50: R = $5d_b$

Stirrups and tie hooks – ACI section 7.1.3



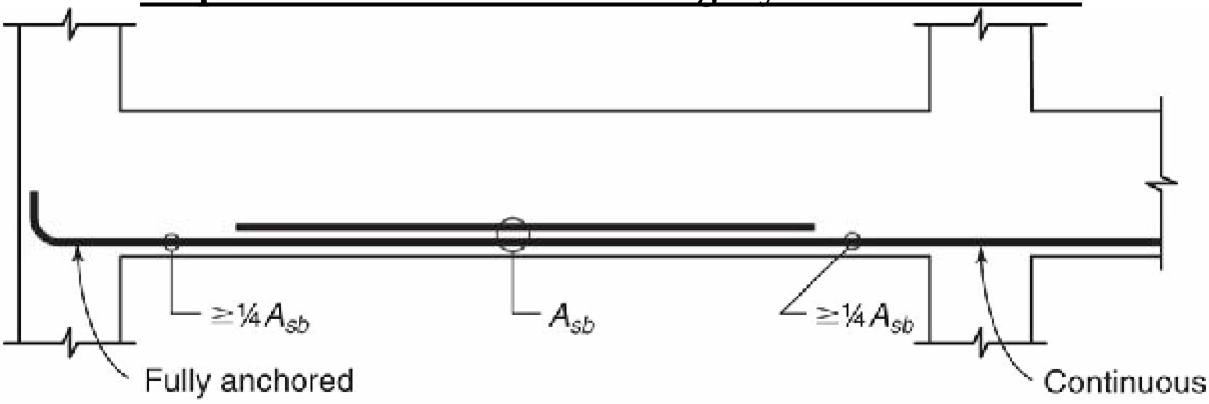
ϕ16 & smaller: R = $2d_b$
ϕ18 to ϕ25: R = $4d_b$

Requirements for structural integrity in spandrel beams



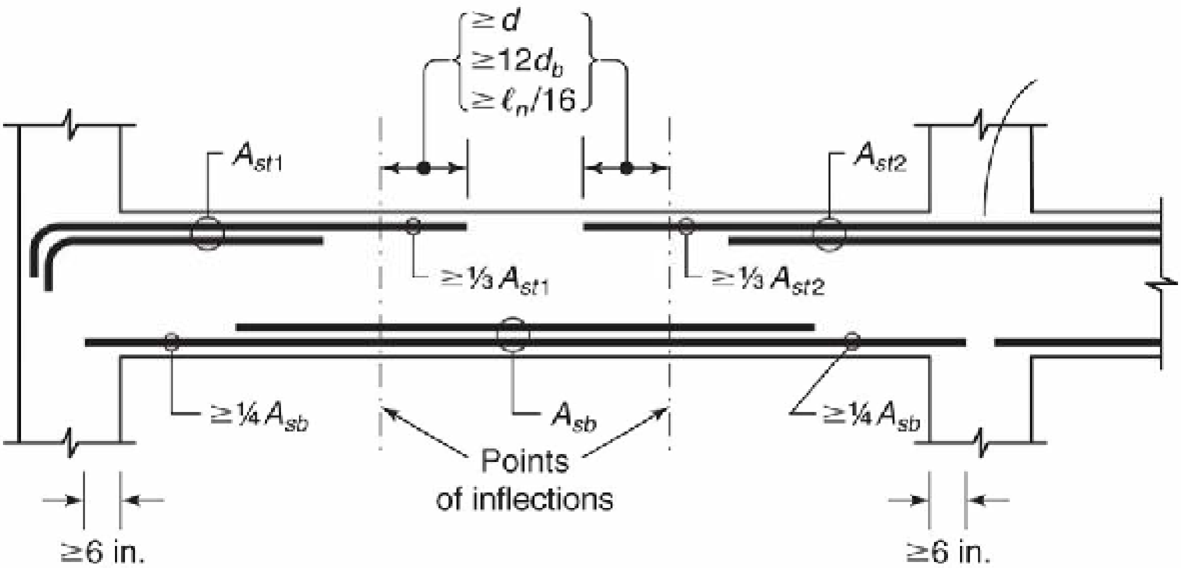
(Must use at least two longitudinal bars at all locations)

Requirements for structural integrity in interior beams



(Must use at least two longitudinal bars at all locations)

(a) Interior beam without closed transverse reinforcement.

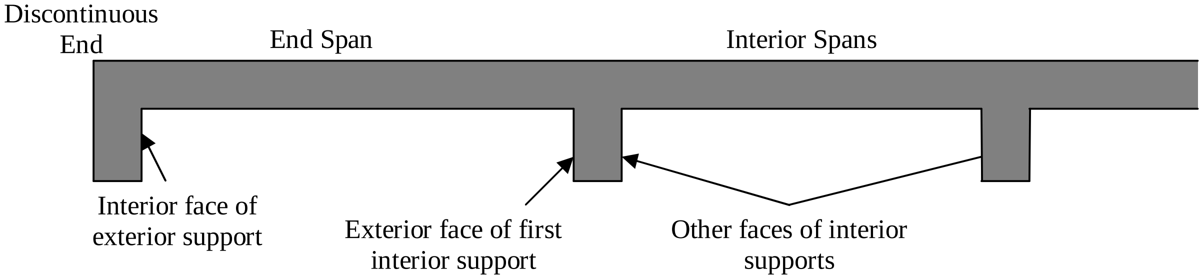


(Must use at least two longitudinal bars at all locations)

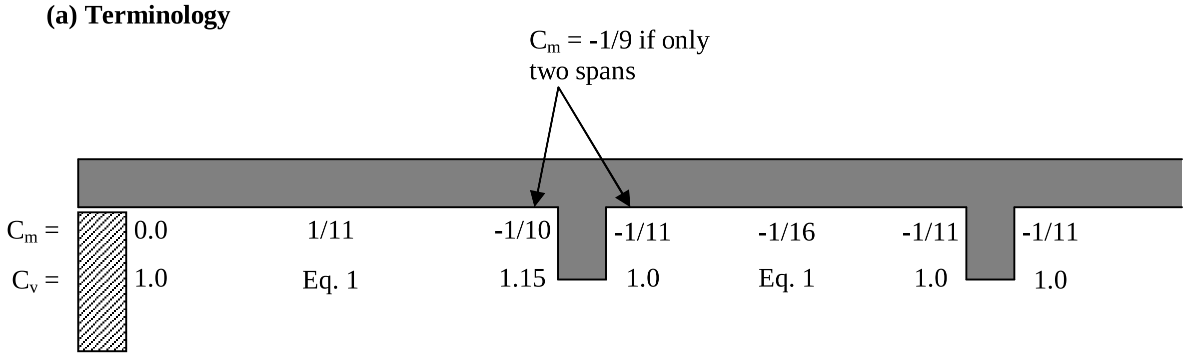
(b) Interior beam with closed transverse reinforcement over total clear span at spacing less than or equal to $d/2$ (transverse reinforcement is not shown).

ACI Moment and Shear Coefficients

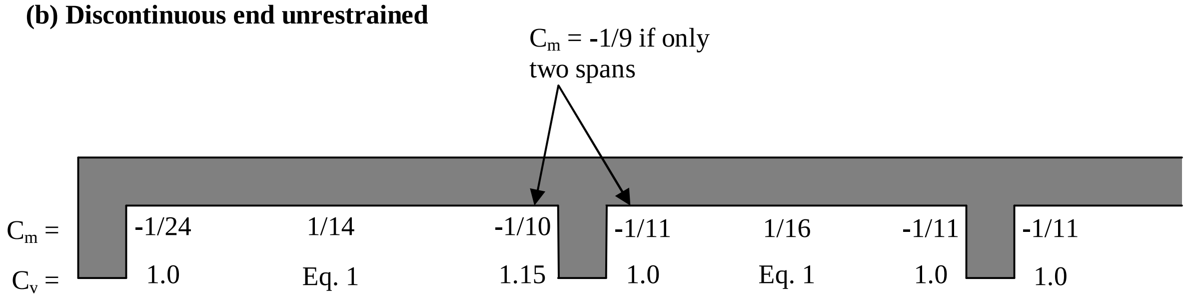
$M_u = C_m(w_u l_n^2)$; C_m : moment envelope coefficient
$V_u = C_v(w_u l_n/2)$; C_v : shear envelope coefficient
Where w_u is total factored load and l_n is clear span



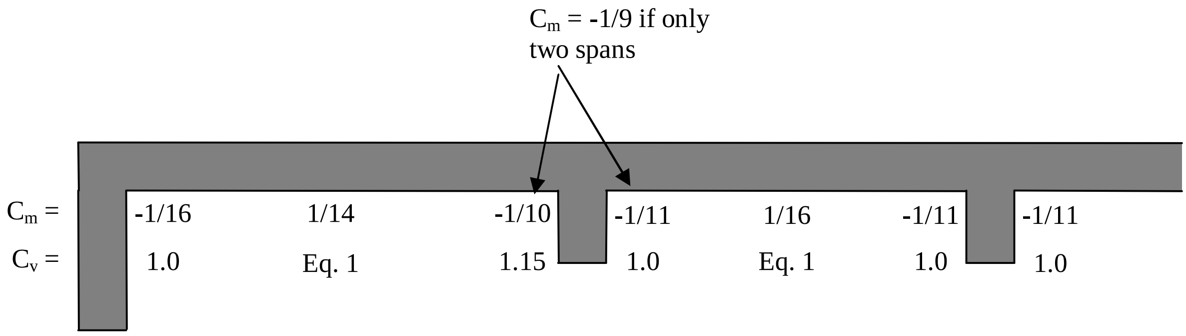
(a) Terminology



(b) Discontinuous end unrestrained



(c) Discontinuous end integral with support where support is spandrel beam



(d) Discontinuous end integral with support where support is a column

$$Eq.1:\ C_v = \text{larger of } (0.15) \text{ or } \left(\frac{0.25 w_{Lu}}{w_u}\right)$$, where w_{Lu} is factored live

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	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$

Notes:
Values given shall be used directly for members with normalweight concrete (density w_c = 2320 kg/m^3) and Grade 420 reinforcement. For other conditions, the values shall be modified as follows:
a) For structural lightweight concrete having unit density, w_c, in the range 1440-1920 kg/m^3, the values shall be multiplied by $(1.65 - 0.003w_c)$ but not less than 1.09.
b) For f_y other than 420 MPa, the values shall be multiplied by $(0.4 + f_y/700)$.

TABLE 9.5(b) — MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\ell/180$*
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\ell/360$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\ell/480$‡
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\ell/240$§

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.
† Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.3, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.
‡ Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.
§ Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

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

	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.

Instantaneous Deflection Calculations:

$$\Delta_i = K\frac{5}{48}\frac{M_a l^2}{E_c I_e}$$

M_a is the support moment for cantilevers and the midspan moment (when K is so defined) for simple and continuous beams.

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	$1.2-0.2\,M_o/M_a$
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, K values are given in Ref. 8.2.

M_o = Simple span moment at midspan $\left(\frac{w\ell^2}{8}\right)$

M_a = Net midspan moment.

Long-term Deflection:

$$\Delta_{long-term} = \lambda(\Delta_i)_{sustained\ load}$$

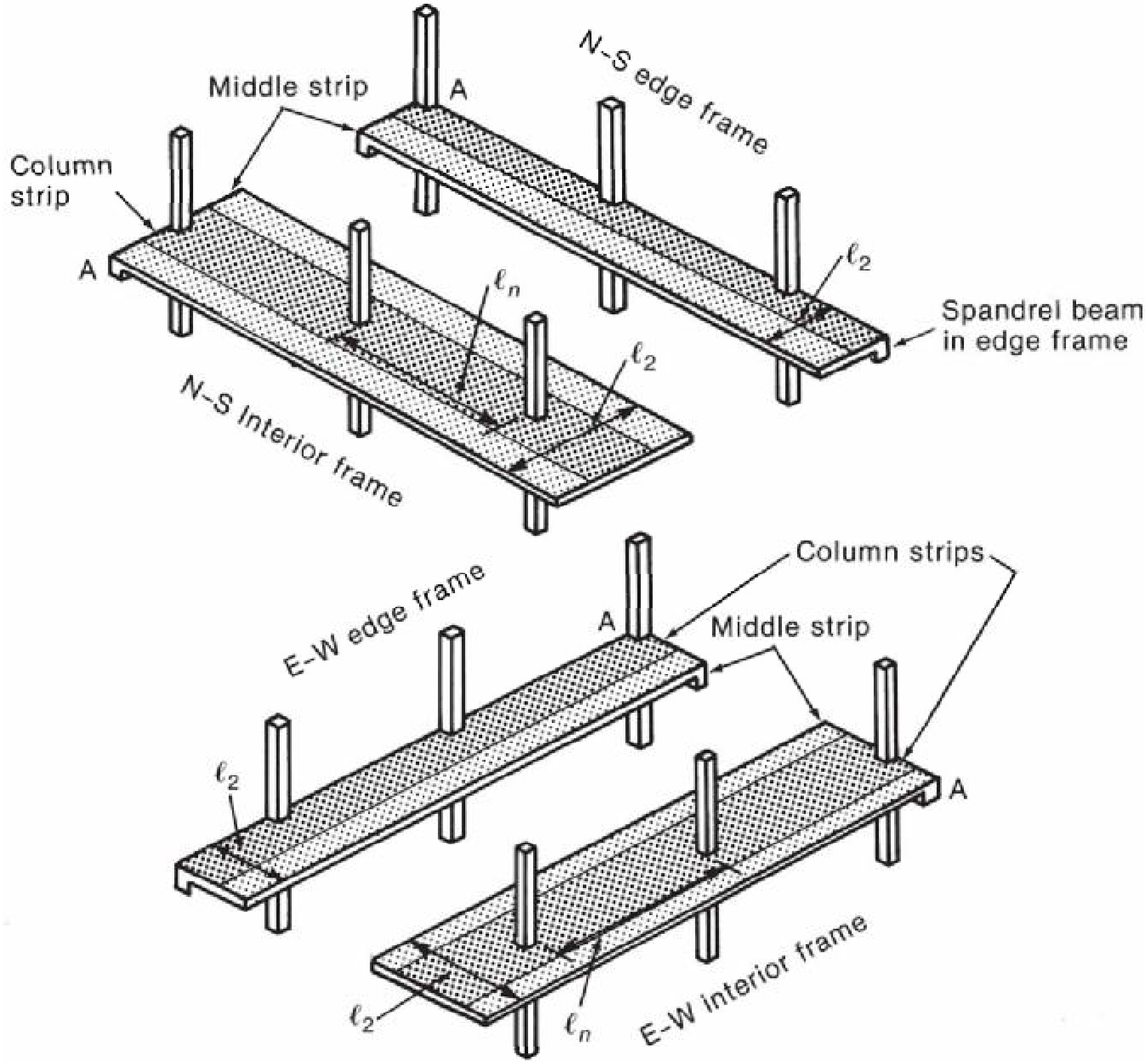
$$\lambda = \frac{\xi}{1 + 50\rho'}$$

$\xi = 1.0$ ……………….. t = 3 months
$\xi = 1.2$ ……………….. t = 6 months
$\xi = 1.4$ ……………….. t = one year
$\xi = 2.0$ ……………….. t > 5 years

Direct Design Method (DDM) – Two-way Slabs

Total Static Moment =

$$M_o = \frac{w_u l_2 l_n^2}{8}$$



Total static moment distribution

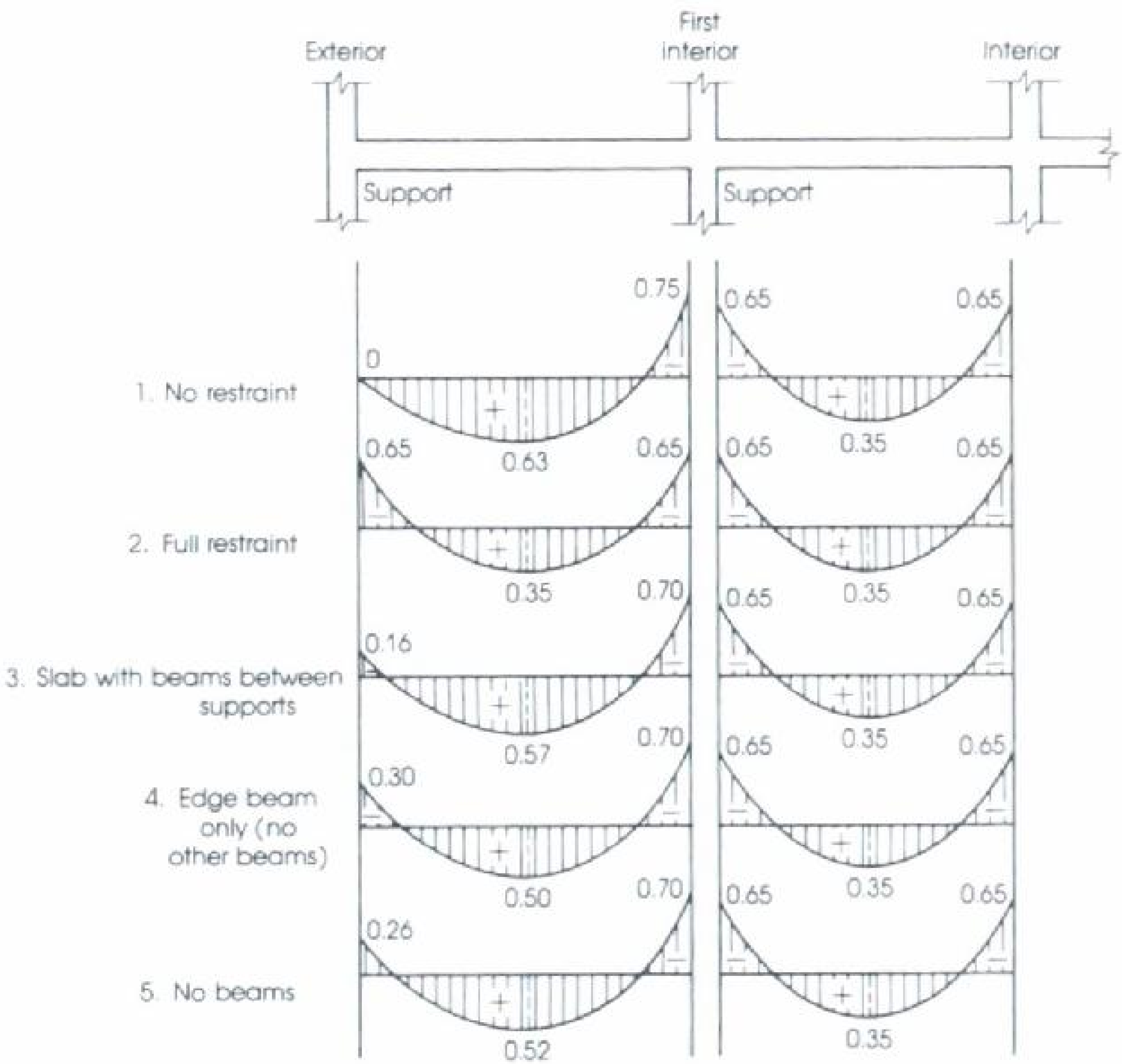


TABLE 13-3 Percentage Distribution of Interior Negative Factored Moment to Column Strip

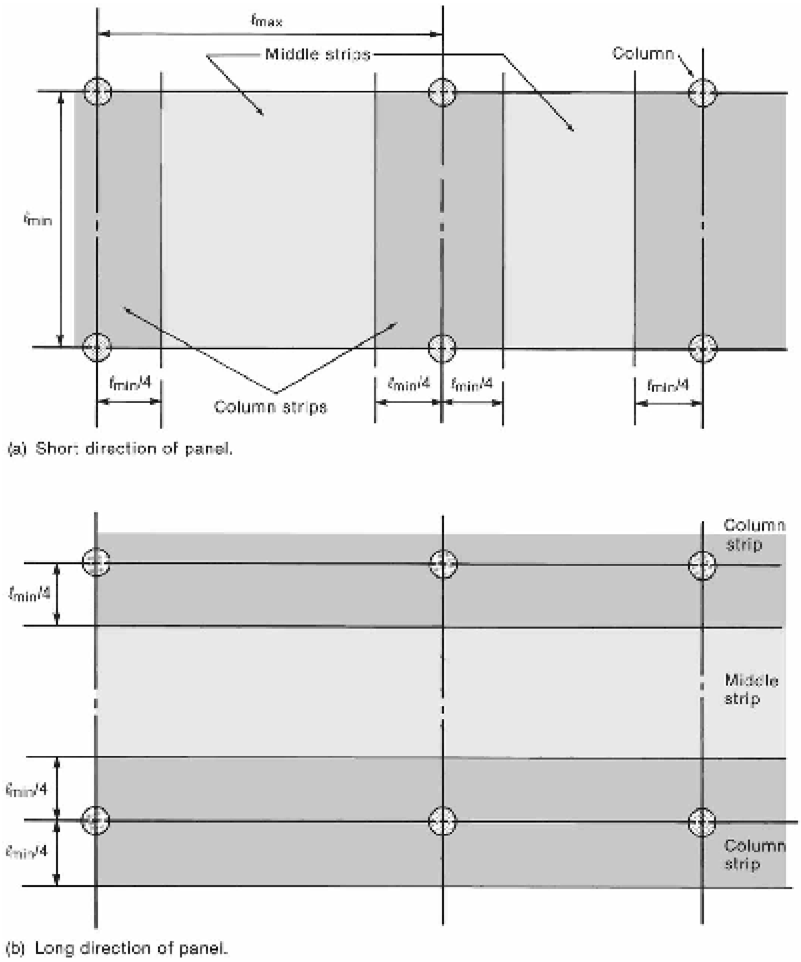
ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_{f1}\ell_2/\ell_1) = 0$	75	75	75
$(\alpha_{f1}\ell_2/\ell_1) \geq 1.0$	90	75	45

TABLE 13-4 Percentage Distribution of Midspan Positive Factored Moment to Column Strip

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_{f1}\ell_2/\ell_1) = 0$	60	60	60
$(\alpha_{f1}\ell_2/\ell_1) \geq 1.0$	90	75	45

TABLE 13-5 Percentage Distribution of Exterior Negative Factored Moment to Column Strip

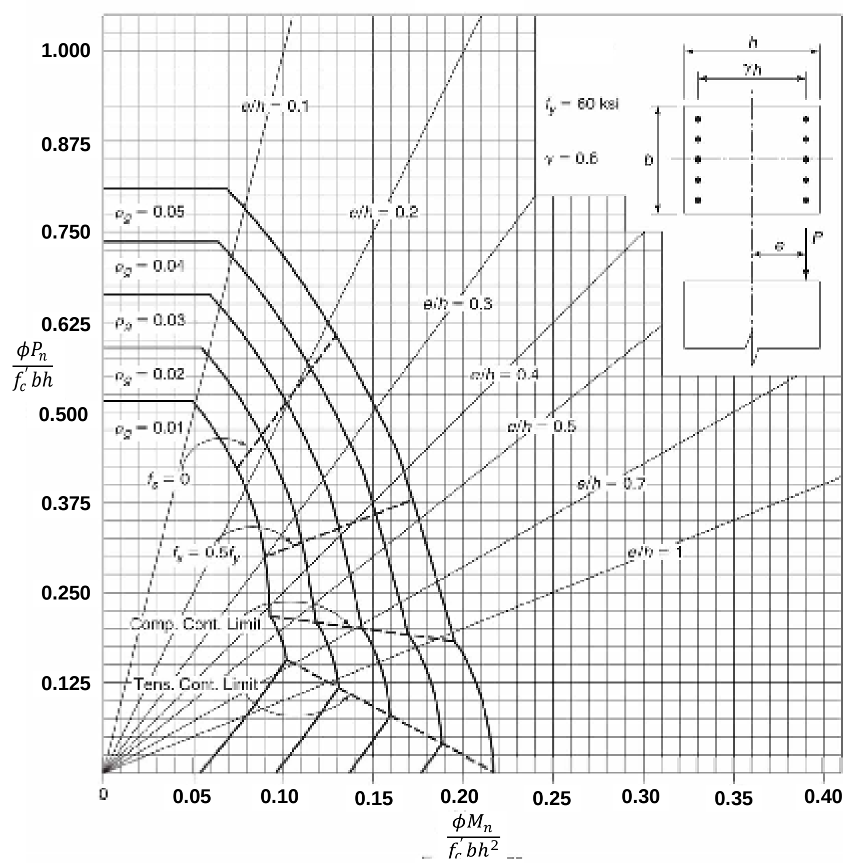
ℓ_2/ℓ_1		0.5	1.0	2.0
$(\alpha_{f1}\ell_2/\ell_1) = 0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	75	75	75
$(\alpha_{f1}\ell_2/\ell_1) \geq 1.0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	90	75	45

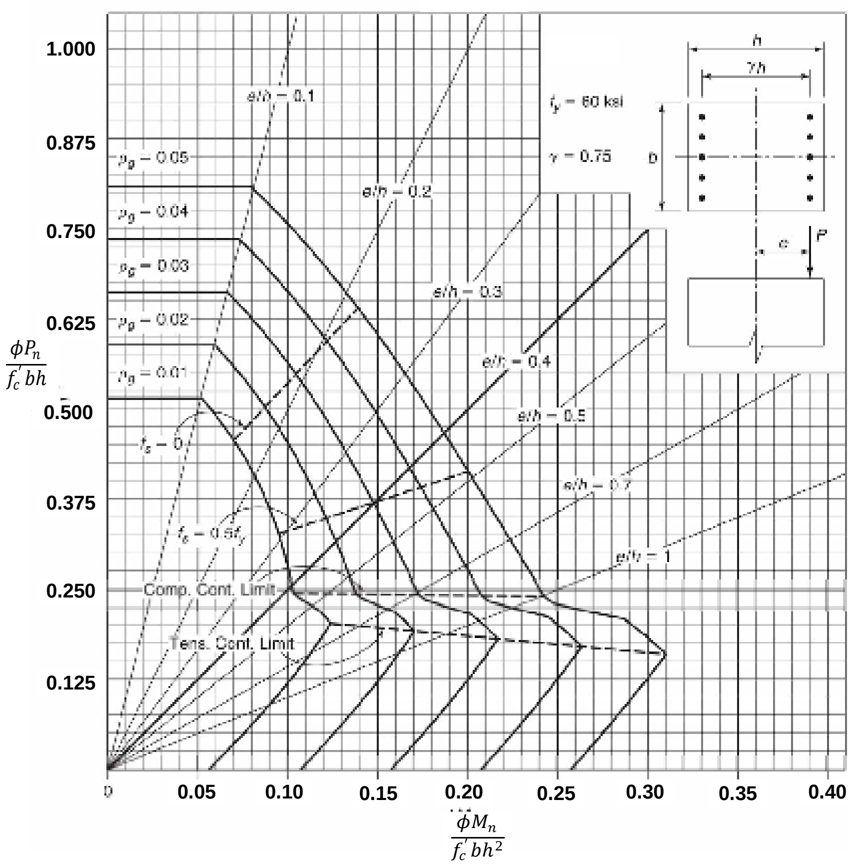


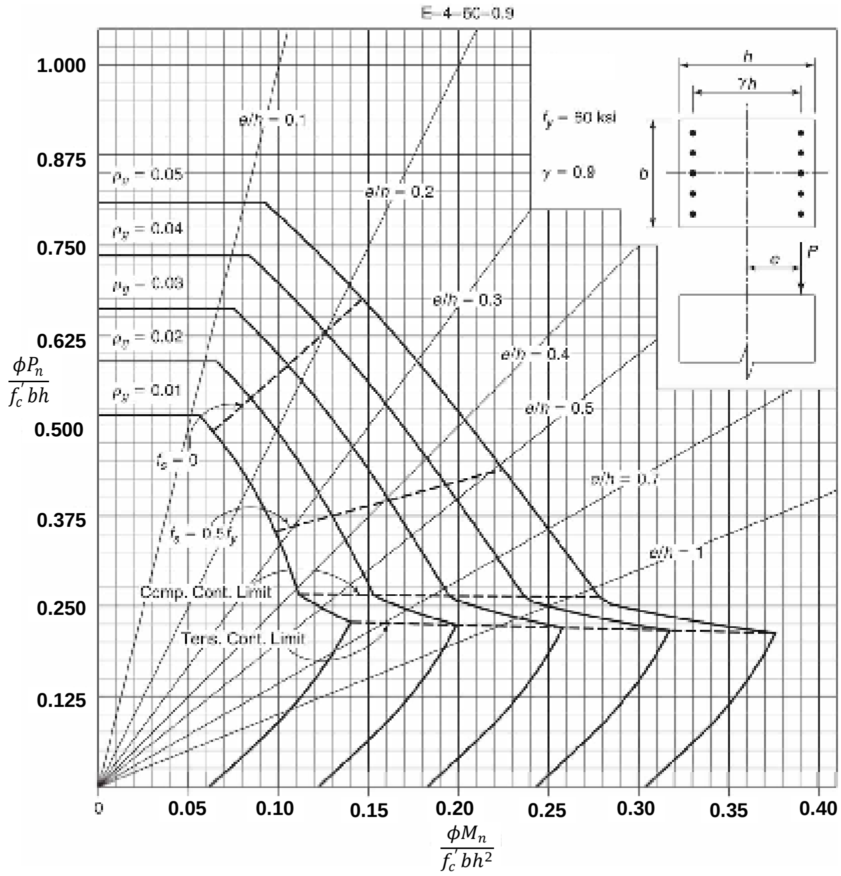
(a) Short direction of panel.

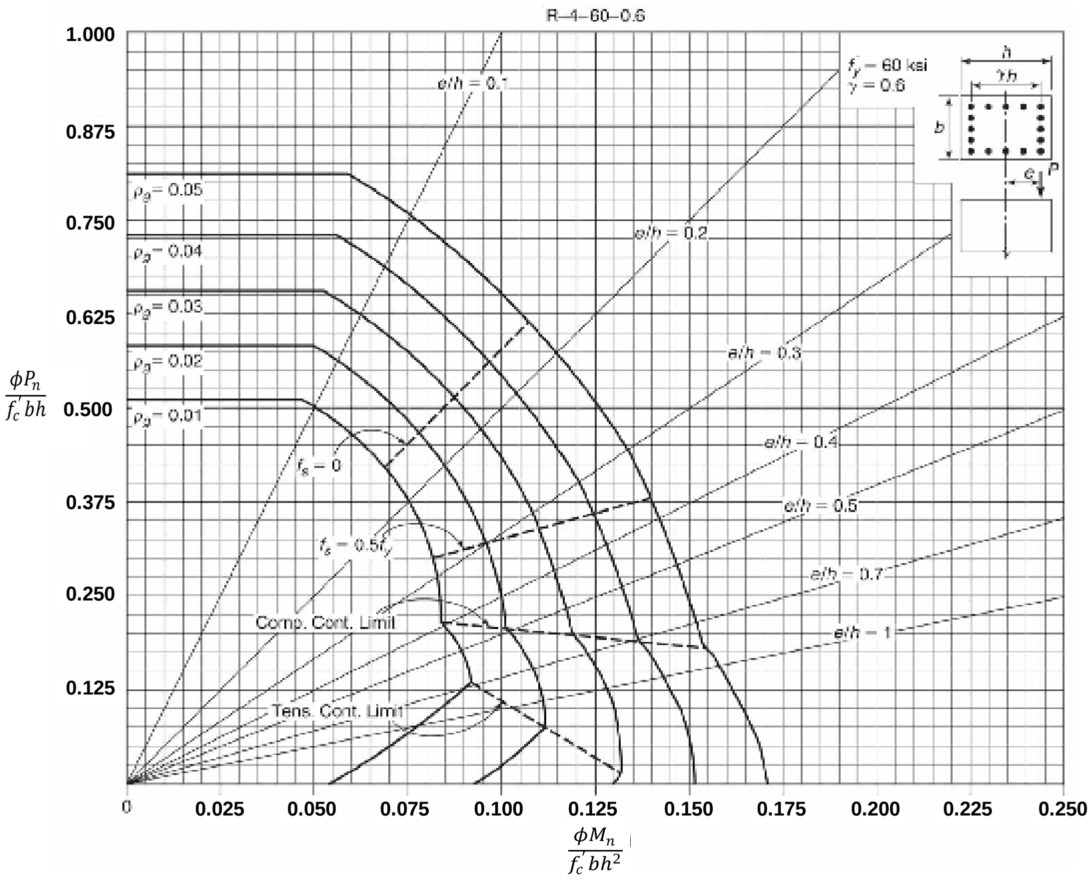
(b) Long direction of panel.

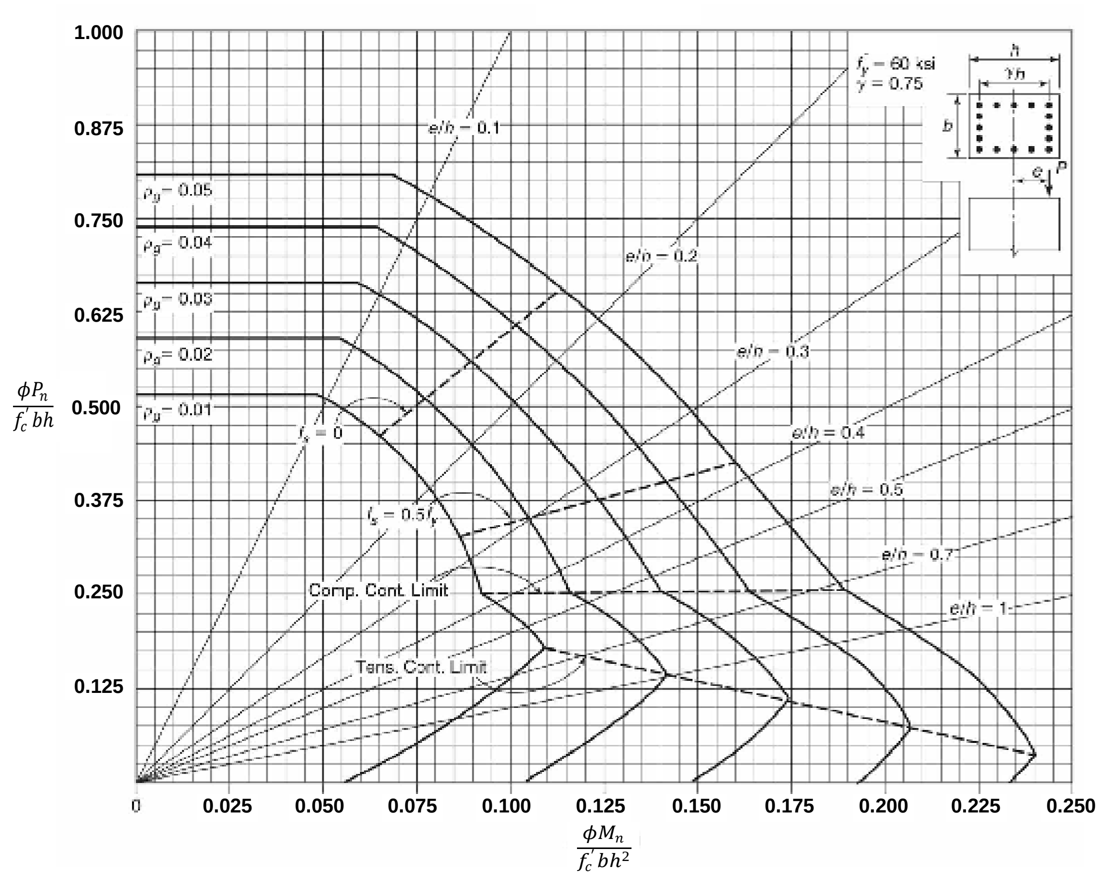
Rectangular Column Interaction Diagrams



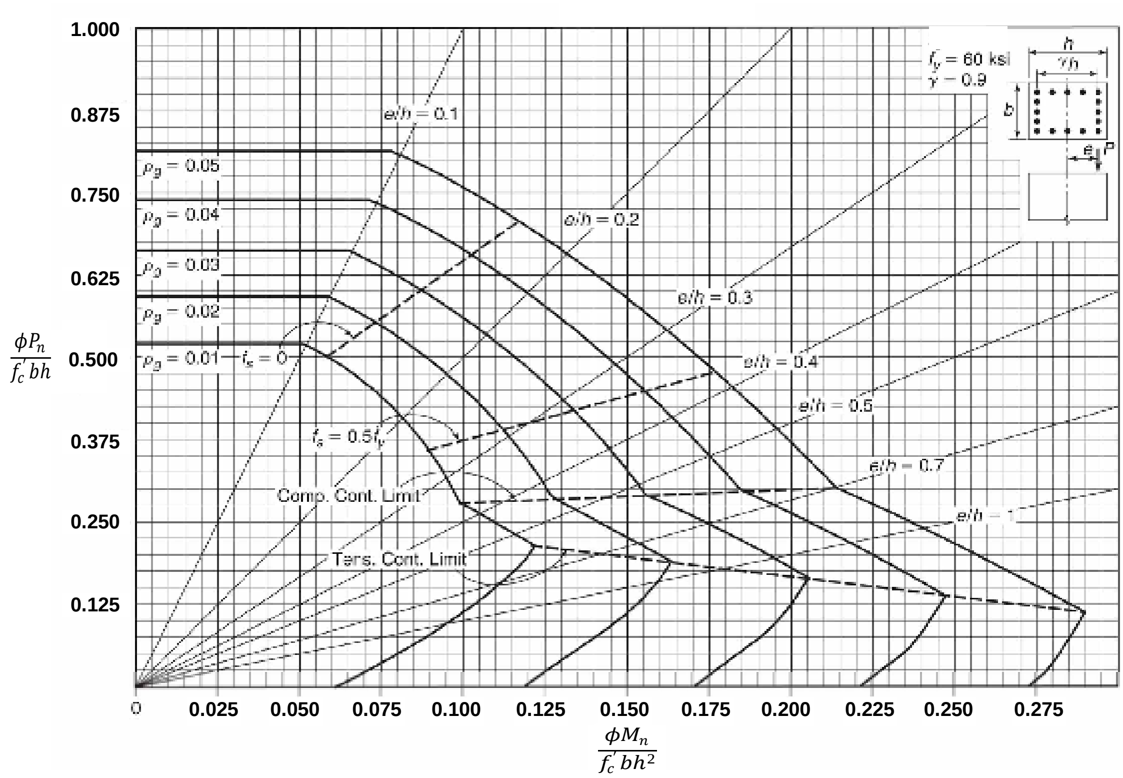



E-4-60-0.9
f_y = 60 ksi
γ = 0.9
h
γh
b
e
P
e/h = 0.1
e/h = 0.2
e/h = 0.3
e/h = 0.4
e/h = 0.5
e/h = 0.7
e/h = 1
ρ_g = 0.05
ρ_g = 0.04
ρ_g = 0.03
ρ_g = 0.02
ρ_g = 0.01
f_s = 0
f_s = 0.5f_y
Comp. Cont. Limit
Tens. Cont. Limit
1.000
0.875
0.750
0.625
0.500
0.375
0.250
0.125
$\frac{\phi P_n}{f_c' bh}$
0
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
$\frac{\phi M_n}{f_c' bh^2}$


R-4-60-0.6
f_y = 60 ksi
γ = 0.6
h
γh
b
e
P
e/h = 0.1
e/h = 0.2
e/h = 0.3
e/h = 0.4
e/h = 0.5
e/h = 0.7
e/h = 1
ρ_g = 0.05
ρ_g = 0.04
ρ_g = 0.03
ρ_g = 0.02
ρ_g = 0.01
f_s = 0
f_s = 0.5f_y
Comp. Cont. Limit
Tens. Cont. Limit
1.000
0.875
0.750
0.625
0.500
0.375
0.250
0.125
$\frac{\phi P_n}{f_c' bh}$
0
0.025
0.050
0.075
0.100
0.125
0.150
0.175
0.200
0.225
0.250
$\frac{\phi M_n}{f_c' bh^2}$

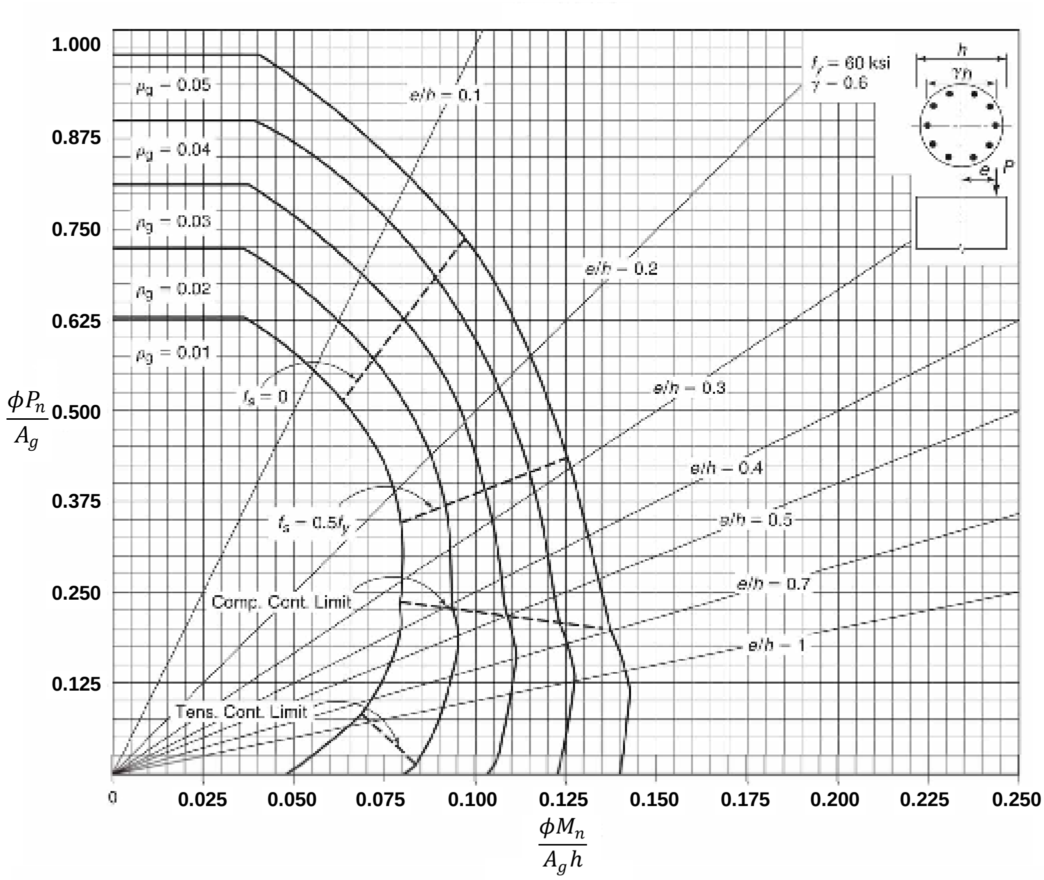


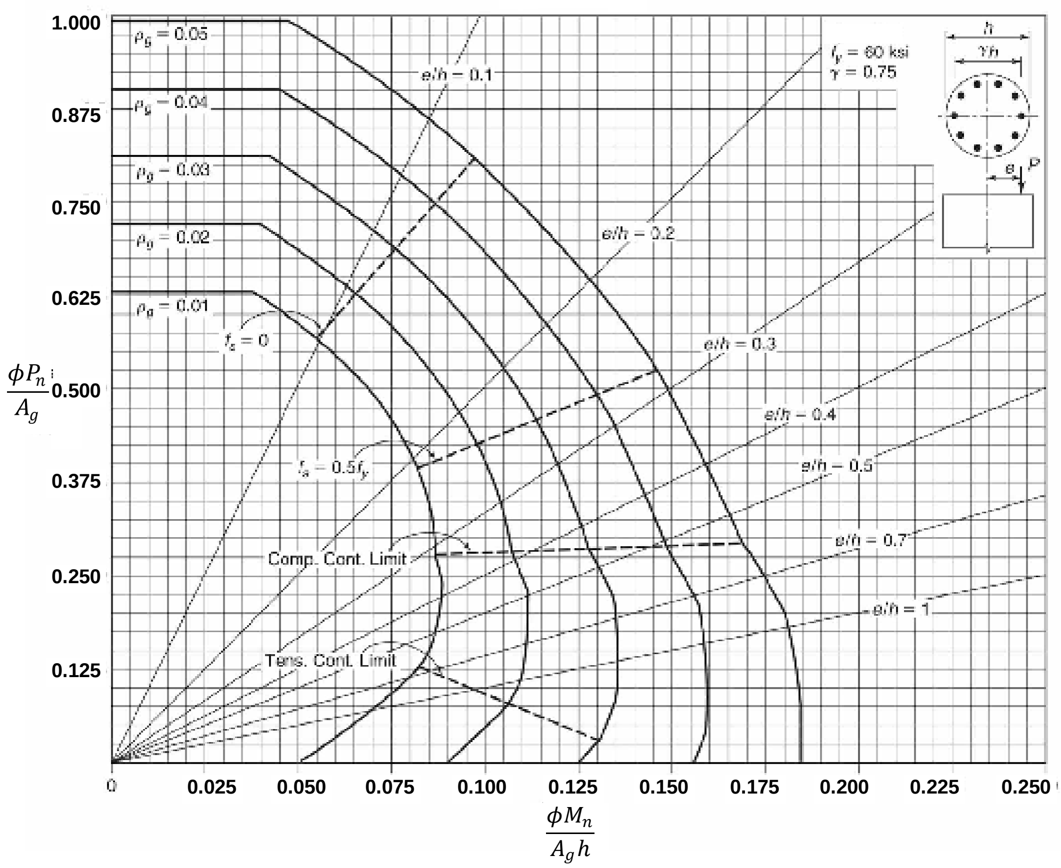
f_y = 60 ksi
γ = 0.75
e/h = 0.1
e/h = 0.2
e/h = 0.3
e/h = 0.4
e/h = 0.5
e/h = 0.7
e/h = 1
ρ_g = 0.05
ρ_g = 0.04
ρ_g = 0.03
ρ_g = 0.02
ρ_g = 0.01
f_s = 0
$f_s = 0.5f_y$
Comp. Cont. Limit
Tens. Cont. Limit
$\frac{\phi P_n}{f_c' bh}$
$\frac{\phi M_n}{f_c' bh^2}$

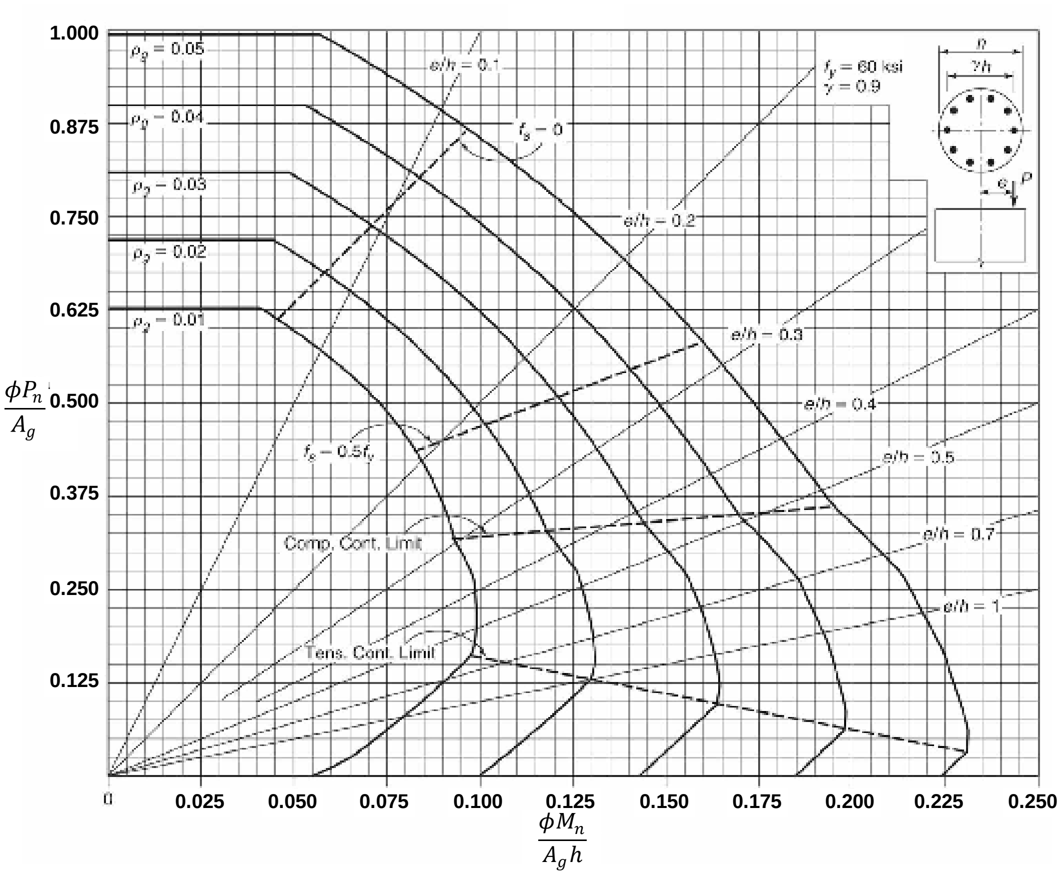


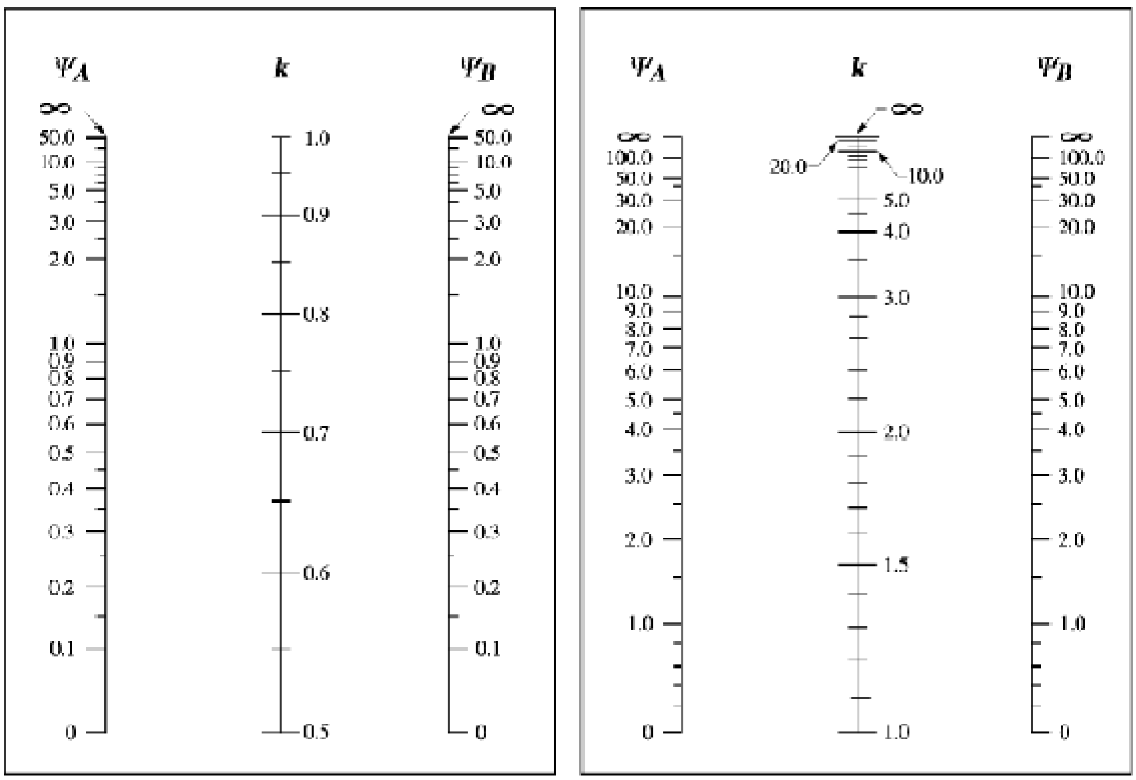
f_y = 60 ksi
γ = 0.9
e/h = 0.1
e/h = 0.2
e/h = 0.3
e/h = 0.4
e/h = 0.5
e/h = 0.7
e/h = 1
ρ_g = 0.05
ρ_g = 0.04
ρ_g = 0.03
ρ_g = 0.02
ρ_g = 0.01
f_s = 0
$f_s = 0.5f_y$
Comp. Cont. Limit
Tens. Cont. Limit
$\frac{\phi P_n}{f_c' bh}$
$\frac{\phi M_n}{f_c' bh^2}$

Circular Column Interaction Diagrams









(a)
Nonsway Frames

(b)
Sway Frames

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

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

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