pci design handbook

Precast and Prestressed Concrete

ERRATA

PCI Design Handbook
Precast and Prestressed Concrete
(Third Edition, MNL-120-85)

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pp. 2-38 through 2-43 — Change dimension on each sketch from "Width varies" to 4′–0″.

p. 3-24 — Fig. 3.5.1 — Upper chart, bottom curve should be $k\ell_u/r = 150$.

p. 3-31 — Eq. 3.7.2 — The quantity $r_i/\Sigma\, r$ should be multiplied by the lateral load, W.

p. 4-10 — Fig. 4.10.2, p. 4-62, is corrected Example 4.2.2 is affected as indicated on the corrected p. 4-10.

pp. 4-27 and 4-29 — In Example 4.3.9, if non-prestressed reinforcement is present (as in Example 4.2.6), the force in the bars, A_sf_y or $A_s'f_y$, should be included in the calculation of T or C.

p. 4-30 — Left column, change bottom equation to

$$T_c = \frac{0.8\sqrt{5000}\,(3680)}{1000\sqrt{1 + \left(\dfrac{0.4(53.1)}{0.1125(376)}\right)^2}} = 186 \text{ in.-kips}$$

calculations in right column change accordingly as indicated on the corrected p. 4-30.

p. 4-35 — Right column, line 11, change to
$>\phi 0.25\, T_{ocr} = 0.85\,(0.25)\,(81.8) =$ 17.38 ft-kips

p. 4-40 — Sect. 4.5.3 — coefficients K_{es} and K_{cr} are for *pretensioned* members as are K_{cir} and K_{sh}.

pp. 4-44 and 4-74 — Change coefficient in Eq. 4.6.2 and in equation for I_{cr} in heading for table, Fig. 4.10.14 from 1.67 to 1.6. Tabular values for C in Fig. 4.10.14 are based on coefficient of 1.6 and are correct as given.

p. 4-47 — Tabulated cambers and deflections (top of page) are incorrect. A corrected p. 4-47 is attached. Change numbers in Example 4.6.4 as follows:
"Final" long-time camber = 1.24 in.
$\Delta_\ell = (1.80 - 1.24) + 2.36$
$= 2.92 \text{ in} < 3.50 \text{ in.}$ OK

p. 4-61 — In equations 1 (Design), and 3 (Analysis), change d_s' to d'.

p. 4-62 — The tabular values for concrete strengths of 5000 psi and greater are incorrect. Also, in sketch, change d to d_p; second note, change $\omega_{pu} > 0.366\,\beta_1$ to $\omega_p > 0.36\,\beta_1$; in table heading, Col. 1, change f_u' to f_c'. A corrected table (p. 4-62) is attached.

p. 4-63 — In equation, change d in denominator of 1st term to d_p.

p. 5-4 — Top of left column, below Fig. 5.2.2 should read
$\lambda\sqrt{f_{ci}'} = f_{ct}/6.7 \leq \sqrt{f_{ci}'}$

p. 5-15 — In Fig. 5.2.12, change heading, Col. 2, to:
Reactions and vertical shear
In Col. 3, change third line to:
$$M = -\frac{Wc^2}{2\ell} \text{ at B and C}$$

p. 5-16 — Right column — Units for "w" should be lb/in² (or lb/in/in).

p. 5-19 — Left column, line 6, change "less" to "more".

pp. 6-5 and 6-49 — The effective throat dimensions of 0.2 d_b or 0.3 d_b are valid only when the space is filled flush to the solid section of the bar.

p. 6-7 — In Fig. 6.5.3, Case 1, in the expression for P_{c1}, change ℓ to ℓ_e.

p. 6-8 — Left column, in Eq. 6.5.5 and in line immediately following Eq. 6.5.5, change f_y to f_s.

p. 6-11 — Right column, line 6 — change to ASTM A-36.

p. 6-13 — In Fig. 6.5.9, Section No. 4, expression for $\overline{x}$, change minus sign to plus sign in denominator.

pp. 6-19, 6-20 and 6-30 — Equations 6.7.2, 6.9.2 and 6.13.5 — complete equations by adding: $\leq$ (values in Table 6.7.1)

p. 6-19 — In Table 6.7.1, right column, change heading to read: Maximum V_n ($= V_u/\phi$), lb

p. 6-19 — Left column, the expression for λ should read: $= f_{ct}/6.7/\sqrt{f'_c}$

p. 6-21 — Right column, line 6 — should read: Use 2 - #3 stirrups = 0.44 sq in.

p. 6-23 — Left column, top: The value shown for f_{bu} is an upper limit.

p. 6-25 — Right column, line 10 — Change Ref. 7 to Ref. 13.

p. 6-26 — Fig. 6.12.2 — For (a), missing dimension is ℓ_e. For (c), ℓ_e is to end of embedment, not full width of column; change strain and stress diagrams accordingly.

pp. 6-54 through 6-58 — Values in the tables do *not* include ϕ as indicated. Change ϕP_c in headings to P_{c1}. Example 6.5.2, p. 6-9, is affected by the above, as indicated on the corrected pp. 6-9 and 6-10.

p. 6-62 — In Table 6.20.19, middle table for Tension on External Anchor Bolts, missing values for b in Col. 1 vary from 12 to 28 in., same as in lower table.

pp. 6-63 through 6-67 — The tables do *not* contain the limitation of Eq. 6.11.7, $\leq$ 800 bd. This will frequently control, especially on small corbels.

Table 6.20.20 — Add to criteria:

$a = 0.75\ \ell_p$

$d = h - 1.25$

$V_u = \phi V_n \leq 0.8\phi bd$

Change tabular values accordingly. A corrected set of tables is attached.

p. 11-11 — In Beam No. 24, the expression for M_x should read:

$$= M_{max}\left(1 - \frac{x}{\ell}\right)$$

p. 11-16 — In Table 11.2.3, change values in tables for 0.600-in. strand:

For 270-ksi, 7-wire strand —
0.217
0.74
41.0
44.0
46.9
58.6

For 250-ksi, 7-wire strand —
0.216
0.74
37.8
43.2
54.0

p. 11-30 — Change 16.387 to 16,387. Change 416.231 to 416,231.

For explanation of the changes and background on new material in the PCI Design Handbook, see "Explanatory Discussion on PCI Design Handbook, Third Edition" by PCI Committee on Industry Handbook, PCI JOURNAL, May-June 1988, pp. 64-89. Reprints available from Prestressed Concrete Institute at $3.00 per copy (Code No.: JR344) member price.

Example 4.2.2 Use of Fig.4.10.2 for determination of prestressing steel requirements—bonded strand

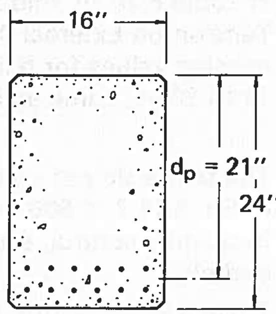
Given:

PCI standard rectangular beam 16RB24

Applied factored moment, M_u = 600 ft-kips

f'_c = 6000 psi normal weight concrete

f_{pu} = 270 ksi, low-relaxation strand



Problem:

Find the required amount of prestressing steel.

Solution:

Referring to Fig. 4.10.2:

$$M_u \le \phi M_n = K'_u \, bd_p^2/12{,}000$$

$$\text{Req'd } K'_u = \frac{M_u\,(12{,}000)}{bd_p^2} = \frac{600(12{,}000)}{16(21)^2}$$

$$= 1020$$

$$\text{for } \omega_{pu} = 0.23,\ K'_u = 1014$$
$$= 0.24,\ K'_u = 1045$$

therefore

$$\omega_{pu} = 0.23 + \frac{1020 - 1014}{1045 - 1014}(0.01) = 0.232$$

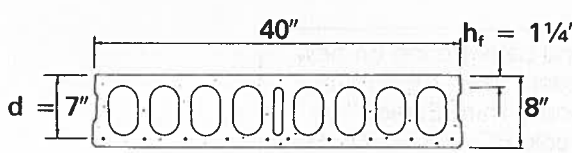
$$A_{ps} = \frac{\omega_{pu}\,bd_pf'_c}{f_{pu}} = \frac{0.232(16)(21)(6)}{270}$$
$$= 1.73 \text{ sq. in.}$$

Use 12 – 1/2″ diameter strands; A_{ps} = 1.84 sq in.

Example 4.2.3 Use of Fig. 4.10.3—values of f_{ps} by stress-strain relationship—bonded strand

Given:

3′-4″ × 8″ hollow-core slab



Concrete:

f'_c = 5000 psi normal weight concrete

Prestressing steel:

10 - 3/8″ diameter 250K stress-relieved strand

A_{ps} = 10(0.080) = 0.800 sq in.

Section properties:

$A = 218$ in²

$Z_b = 381$ in³

$y_b = 3.98$ in.

Problem:

Find design flexural strength, ϕM_n

Determine $C\omega_{pu}$ for the section:

$$C\omega_{pu} = C\,\frac{A_{ps}f_{pu}}{bd_pf'_c} + \frac{d}{d_p}(\omega - \omega')$$

since $\omega = \omega' = 0$

$$C\omega_{pu} = \frac{1.06(0.8)(250)}{(40)(7)(5)} = 0.151$$

Entering Fig.4.10.3 with this parameter and an assumed effective stress, f_{se} = 150 ksi gives a value of:

$$f_{ps}/f_{pu} = 0.965 \text{ or } f_{ps} = 0.965(250)$$
$$= 241 \text{ ksi}$$

Determine the flexural strength:

$$\phi M_n = \phi[A_{ps}f_{ps}(d_p - a/2) + A_sf_s(d - a/2)]$$

$$a = (A_{ps}f_{ps} + A_sf_y)/(0.85f'_cb)$$

Since A_s = 0:

$$a = \frac{0.800(241)}{0.85(5)(40)} = 1.134 \text{ in.}$$

$$\phi M_n = 0.9[0.8\,(241)\,(7 - 1.134/2) + 0]$$
$$= 1116 \text{ in-kips} = 93.0 \text{ ft-kips}$$

Check the ductility requirement, $\phi M_n > 1.2M_{cr}$

$$P = f_{se}\,A_{ps} = 150\,(0.80)$$
$$= 120 \text{ kips}$$

$$1.2M_{cr} = 1.2(P/A + Pe/Z_b + 7.5\sqrt{f'_c})Z_b$$

$$= 1.2\left(\frac{120}{218} + \frac{120(2.98)}{381} + \frac{7.5\sqrt{5000}}{1000}\right)381$$

$$= 923 \text{ in.-kips} = 76.9 \text{ ft-kips}$$
$$< 93.0 \text{ ft-kips} \qquad \text{OK}$$

Span of spandrel beam = 30 ft clear

f'_c = 5000 psi, normal weight concrete

Reinforcement f_y = 60,000 psi

d = 69 in.

Loads (kips/ft):

D.L.:				
Precast floor	60 psf	(20ft)	= 1.2 (1.4)	= 1.68
Topping	25	(20)	= 0.5 (1.4)	= 0.70
Superimposed	10	(20)	= 0.2 (1.4)	= 0.28
Window			= 0.50 (1.4)	= 0.07
Spandrel			= 0.63 (1.4)	= 0.88
L.L.:	50 psf	(20)	= 1.00 (1.7)	= 1.70
			w_u	= 5.31

Problem:

Determine torsion reinforcement requirements

Solution:

1. Compute torsion moment (T_u) at critical section, assumed to be 5′-0″ from face of support:

$$V_u = w_u(15-5) = 5.31(10) = 53.1 \text{ kips}$$

$$w_u \text{ for torsion} = 1.68 + 0.70 + 0.28 + 1.70 = 4.36 \text{ kips/ft}$$

$$\text{Eccentricity} = 2/3\ (8) + 3.29 = 8.62 \text{ in.}$$

$$T_u = w_u\ (e)\ (\ell/2-5) = 4.36\ (8.62)\ (10) = 376 \text{ in.-kips}$$

2. Determine if torsion effects must be considered.

If $T_u \geq \phi\ (0.5\sqrt{f'_c}\ \Sigma x^2y)$ must consider torsion.

$$\Sigma x^2y = 6^2(72) + 6^2(8)(2) + 8^2(8) = 3680 \text{ in.}^3$$

$$\phi(0.5\sqrt{f'_c}\ \Sigma x^2y) = \frac{0.85(0.5)\sqrt{5000}\ (3680)}{1000} = 110.6 \text{ in.-kips}$$

376 > 110.6, consider torsion

3. Determine the torsion moment strength provided by concrete

$$T_c = \frac{0.8\sqrt{f'_c}\ \Sigma x^2y}{\sqrt{1+\left(\dfrac{0.4\ V_u}{C_tT_u}\right)^2}}$$

where:

$$C_t = \frac{b_wd}{\Sigma x^2y} = \frac{6(69)}{3680} = 0.1125$$

$$T_c = \frac{0.8\ \sqrt{5000}\ (3680)}{1000\sqrt{1+\left(\dfrac{0.4\ (53.1)}{0.1125\ (376)}\right)^2}} = 186 \text{ in.-kips}$$

4. Determine torsion reinforcement requirements:

$$T_u = \phi T_n = \phi(T_c + T_s)$$

or

$$T_s = \frac{T_u}{\phi} - T_c = \frac{376}{0.85} - 186 = 256 \text{ in.-kips}$$

By Sect. 11.6.9.4 (ACI 318-83)

$$T_s \leq 4T_c = 4(186) = 744 \text{ OK}$$

By Eq. 11-23 of ACI 318-83

$$T_s = \frac{A_t\alpha_tx_1y_1f_y}{s}$$

or

$$A_t = \frac{T_s(s)}{\alpha_tx_1y_1f_y}$$

Assume x_1 = 4 in.; y_1 = 70 in.

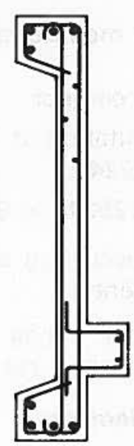
$$\alpha_t = 0.66 + 0.33(y_1/x_1) \leq 1.5$$
$$= 0.66 + 0.33(70/4) = 6.44, \text{ use } \alpha_t = 1.5$$

$$A_t = \frac{256(12)}{1.5(4)(70)(60)} = 0.12 \text{ sq in./ft} = 0.01 \text{ sq in./in.}$$

This is the required area of steel in each leg of the closed stirrup for torsion only. The shear steel requirement must be added to A. The minimum area of closed stirrups is:

$$A_v + 2A_t = 50b_ws/f_y$$

Placement of closed ties in a 6-in. web is difficult. Consider re-design with greater web thickness, or arrange reinforcement as follows:



	(1) Release	Multiplier	(2) Erection	Multiplier	(3) Final
Prestress	4.35 ↑	1.80 × (1)	7.83 ↑	2.45 × (1)	10.66 ↑
w_d	3.00 ↓	1.85 × (1)	5.55 ↓	2.7 × (1)	8.10 ↓
	1.35 ↑		2.28 ↑		2.56 ↑
w_{sd}			0.48 ↓	3.0 × (2)	1.44 ↓
			1.80 ↑		1.12 ↑
w_ℓ					2.36 ↓
					1.24 ↓

Example 4.6.4 Use of multipliers for determining long-time cambers and deflections

Given:

8DT24 of Examples 4.2.9, 4.6.1, 4.6.2 and 4.6.3. Non-structural elements are attached, but not likely to be damaged by deflections (light fixtures, etc.).

Problem:

Estimate the camber and deflection and determine if it meets the requirements of Table 9.5(b) of the Code (see Table 4.6.1).

Solution:

Calculate the instantaneous deflections caused by the superimposed dead and live loads.

$$\Delta_d = \frac{5w\ell^4}{384\,E_cI} = \frac{5\left(\frac{0.080}{12}\right)(70\times12)^4}{384\,(4287)(20{,}985)}$$

$$= 0.48 \text{ in.} \downarrow$$

$\Delta_\ell = 2.36$ in. ↓ (see Example 4.6.3)

For convenience, a tabular format is used (above).

The estimated critical cambers and deflections would then be:

At erection of the member after w_{sd} is applied = 1.80 in.
"Final" long-time camber = 1.24 in.

The deflection limitation of Table 9.5(b) for the above condition is $\ell/240$.

$$(70 \times 12)/240 = 3.50 \text{ in.}$$

Total deflection occurring after attachment of non-structural elements:

$$\Delta_\ell = (1.80 - 1.24) + 2.36$$
$$= 2.92 \text{ in.} < 3.50 \text{ in. OK}$$

4.7 Compression Members

Precast and prestressed concrete columns and load bearing wall panels are usually proportioned on the basis of strength design. Stresses under service conditions, particularly during handling and erection (especially wall panels) must also be considered. The procedures in this section are based on Chapter 10 of the Code and on the recommendations of the PCI Committee on Prestressed Concrete Columns[11] (referred to in this section as "the Recommended Practice").

4.7.1 Strength Design of Precast Concrete Compression Members

The capacity of a reinforced concrete compression member with eccentric loads is most easily determined by constructing a capacity interaction curve. Points on this curve are calculated using the compatibility of strains and solving the equations of equilibrium as prescribed in Chapter 10 of the Code. Solution of these equations is illustrated in Fig. 4.7.1.

ACI 318-83 waives the minimum vertical reinforcement requirements for compression members if the concrete is prestressed to at least an average of 225 psi after all losses. In addition, the Recommended Practice permits the elimination of column ties, if the nominal capacity is multiplied by 0.85. Interaction curves for typical prestressed square columns and wall panels are provided in Part 2.

Construction of an interaction curve usually follows these steps:

Step 1: Determine P_o for $M_n = 0$. (See Fig. 4.7.1(c))

Step 2: Determine M_o for $P_n = 0$. This is normally done by neglecting the reinforcement above the neutral axis and determining the moment capacity by one of the methods described in Sect. 4.2.1.

Step 3: For non-prestressed columns, P_{nb} and M_{nb} at the balance point may be determined (see Fig. 4.7.1(d)). For prestressed columns, the yield point of the prestressed reinforcement is not well defined and the stress-strain relationship is non-linear over a broad range (see Fig. 11.2.5).

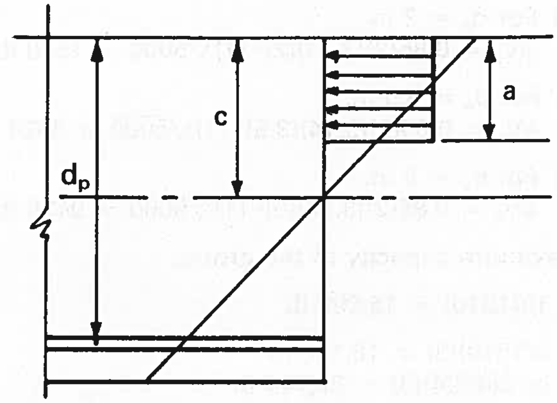
Fig. 4.10.2 Coefficients, K'_u, for determining flexural design strength — bonded prestressing steel

Procedure:

1. Determine $\omega_{pu} = \dfrac{A_{ps}\, f_{pu}}{b d_p\, f'_c}$

2. Find K'_u from table

3. Determine $\phi\, M_n = K'_u \dfrac{b d_p^2}{12{,}000}$ (ft-kips)



Basis:

$$K'_u = \frac{\phi\, f_{ps} f'_c}{f_{pu}}(\omega_{pu})\left[1 - (0.59\omega_{pu})\left(\frac{f_{ps}}{f_{pu}}\right)\right]$$

Note: K'_u from this table is approximately equivalent to $\phi\, K_u\, f'_c$ from Table 4.10.1.

Table values are based on a strain compatibility analysis, using a stress-strain curve for prestressing strand similar to that shown in Fig. 11.2.5. Asterisk(*) indicates $\omega_p > 0.36\beta_1$, and $\phi M_n = \phi[f'_c b d_p^2\, (0.36\ \beta_1 - 0.08\ \beta_1^2)]$

Values of K'_u

f'_c	ω_{pu}	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
3000 psi	0.0	0	26	52	78	103	128	153	178	202	225
	0.1	248	271	293	315	337	358	379	399	419	439
	0.2	458	477	495	513	529	546	562	577	592	607
	0.3	621	634	646	658	670	670*	670*	670*	670*	670*
4000 psi	0.0	0	35	70	104	138	171	204	237	269	300
	0.1	331	361	391	420	449	477	505	532	559	585
	0.2	610	636	660	683	706	728	750	770	790	809
	0.3	827	845	861	878	893	894*	894*	894*	894*	894*
5000 psi	0.0	0	44	87	130	172	214	255	296	336	375
	0.1	413	451	488	524	560	595	630	664	697	729
	0.2	761	792	821	850	878	904	930	955	979	1001
	0.3	1023	1044	1064	1066*	1066*	1066*	1066*	1066*	1066*	1066*
6000 psi	0.0	0	53	105	156	207	257	307	355	403	450
	0.1	495	540	585	628	671	713	754	794	834	873
	0.2	910	945	980	1014	1045	1076	1106	1135	1161	1187
	0.3	1212	1215*	1215*	1215*	1215*	1215*	1215*	1215*	1215*	1215*
7000 psi	0.0	0	61	122	182	241	300	358	414	470	524
	0.1	577	629	681	731	781	830	877	924	970	1013
	0.2	1055	1096	1135	1172	1208	1243	1275	1306	1337	1341*
	0.3	1341*	1341*	1341*	1341*	1341*	1341*	1341*	1341*	1341*	1341*
8000 psi	0.0	0	70	139	208	276	343	409	473	536	598
	0.1	658	718	777	834	890	945	999	1052	1102	1150
	0.2	1197	1241	1284	1325	1363	1400	1436	1441*	1441*	1441*
	0.3	1441*	1441*	1441*	1441*	1441*	1441*	1441*	1441*	1441*	1441*

Problem:

Find the maximum shear strength.

Solution:

$A_b = 0.20$ sq in.

(a) $\phi V_c = 0.85(800)(0.20)(1)\sqrt{5000} = 9617$ lb/stud

(b) For $d_e = 2$ in.
$\phi V_c = 0.85(2)(3.14)(2)^2(1)\sqrt{5000} = 1510$ lb

(c) For $d_e = 3.5$ in.
$\phi V_c = 0.85(2)(3.14)(3.5)^2(1)\sqrt{5000} = 4624$ lb

(d) For $d_e = 5$ in.
$\phi V_c = 0.85(2)(3.14)(5)^2(1)\sqrt{5000} = 9436$ lb

Maximum capacity of the group:

1. 10(1510) = 15,000 lb
2. 4(1510)(3) = 18,120 lb
   or 2(4624)(3) = 27,744 lb
3. 4(9436) = 37,744 lb

Thus condition 1 controls.

The design shear strength as governed by steel strength is:

$$\phi V_s = 0.75\, A_b\, f_s = 45{,}000\, A_b \qquad \text{(Eq. 6.5.8)}$$

where $\phi = 1.0$

Table 6.20.7 tabulates the maximum capacities from the above equations.

Combined shear and tension

The design strength of studs under combined tension and shear should satisfy the following interaction equations:

$$\text{Concrete: } \frac{1}{\phi}\left[\left(\frac{P_u}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right] \le 1.0 \qquad \text{(Eq. 6.5.9)}$$

where $\phi = 0.85$

$$\text{Steel: } \frac{1}{\phi}\left[\left(\frac{P_u}{P_s}\right)^2 + \left(\frac{V_u}{V_s}\right)^2\right] \le 1.0 \qquad \text{(Eq. 6.5.10)}$$

where $\phi = 1.0$

P_u and V_u are the factored tension and shear loads.

Plate thickness

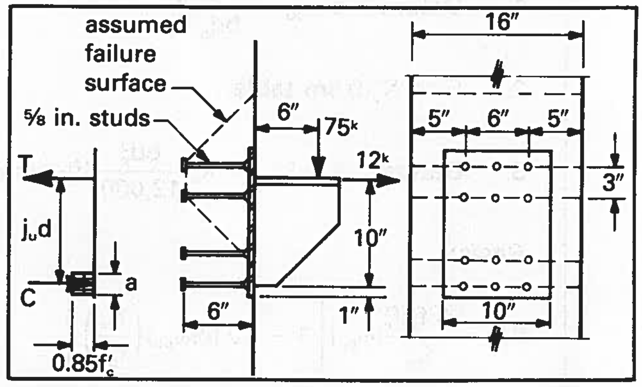
Thickness of plates to which studs are attached should be at least ⅔ of the diameter of the stud.

Example 6.5.2 Capacity of welded headed studs

Given:

Bracket on column as shown.

$f'_c = 5000$ psi (normal weight)
Factored load on bracket = 75 kips



Problem:

Determine if studs are adequate to resist the loads shown.

Solution:

(a) Check concrete strength:

Tension (top group of studs) from Table 6.20.6:

d_e = 5 in., ℓ_e = 6 in., ⅝ in. studs

$\phi P_c = 6(27.4) = 164.4$ kips

This is the cumulative capacity of six individual cones, reduced for edge distance. It can also be determined from Eqs. 6.5.2a and 6.5.3.

Or P_{c1} from Table 6.20.10 (Case 3):

y = 3 in., x = 16 in. ℓ_e = 6 in.
P_{c1} = 67.5 kips
$\phi P_{c1} = 0.85\,(67.5) = 57.4$ kips

This is the capacity of a truncated pyramid accounting for the stud spacing and controls the design.

A moment-resisting couple is formed:

$$C = T = 0.85\, f'_c\, ba = 57.4 \text{ kips}$$

$$\text{comp. block, } a \approx \frac{57.4}{0.85(5)(10)} = 1.35 \text{ in.}$$

$j_u d = 11 - 1.35/2 = 10.3$ in.

$P_u = T + N_u = M_u/j_u d + N_u$
$= 75(6)/10.3 + 12$
$= 55.7$ kips

Check shear (all studs):

From Table 6.20.7:

f'_c = 5000 psi, $d_e > 9$ in., 5/8 in. studs

$\phi V_c = 12(14.7) = 176.4$ kips

To satisfy Eq. 6.5.7:

$\phi V_c = \phi\ 2\pi d_e^2\ \lambda\ \sqrt{f'_c}$

(not critical)

$V_c = 176.4/0.85 = 207.5$ kips

Combined capacity:

From Eq. 6.5.9:

$$\frac{1}{\phi}\left[\left(\frac{P_u}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right] \leq 1.0$$

$$\frac{1}{0.85}\left[\left(\frac{55.7}{67.5}\right)^2 + \left(\frac{75}{207.5}\right)^2\right]$$

$= 0.95 < 1.0$ OK

(b) Check steel strength:

Tension in top group of studs:

From Table 6.20.6 for ⅝ in. studs:

$P_s = 6\ (16.6) = 99.6$ kips

(Could also be determined from Eq. 6.5.5)

$C = T = 0.85\ f'_c\ ba = 99.6$ kips

$$\text{comp. block, } a = \frac{99.6}{0.85(5)(10)} = 2.34 \text{ in.}$$

$j_u d = 11 - 2.34/2 = 9.83$ in.

$P_u = M_u/j_u d + N_u = 75(6)/9.83 + 12$
$= 57.8$ kips

Shear in studs:

From Table 6.20.7 for ⅝ in. studs:

$V_s = 12\ (13.8) = 165.6$ kips

(Could also be determined from Eq. 6.5.8)

Combined capacity:

From Eq. 6.5.10:

$$\left(\frac{57.8}{99.6}\right)^2 + \left(\frac{75}{165.6}\right)^2$$

$= 0.34 + 0.21 = 0.55 < 1.0$ OK

6.5.3 Deformed Bar Anchors

Deformed bar anchors are automatically welded to steel plates, similar to headed studs. They are anchored to the concrete by bond, and the development length can be taken the same as Grade 60 reinforcing bars (see Table 11.2.7).

6.5.4 Bolts and Threaded Connectors

In most connections, bolts are shipped loose and threaded into inserts. Occasionally a precast concrete member will be cast with a threaded connector projecting from the face. This is usually undesirable because of possible damage during handling. When embedded in such a manner, design for concrete strength is similar to that for studs.

High strength bolts are used infrequently in precast concrete connections because it is questionable as to whether the tension can be held when tightened against concrete. When used, AISC recommendations should be followed.

Table 6.20.13 gives allowable working and design strengths for most commonly used threaded fasteners.

High strength threaded rods

Rods with threads and specially designed nuts and couplers are available with properties similar to Grade 60 reinforcing bars and post-tensioning bars. Design information is give in Part 11.

6.5.5 Inserts Cast in Concrete

Loop inserts of the type shown in Fig. 6.5.5 can be investigated in a manner similar to that for welded studs, using Eqs. 6.5.2, 6.5.6 and 6.5.7 for the concrete tensile and shear strengths. The strength as controlled by steel can be taken from manufacturers' catalogs, or calculated based on wire strengths shown in Table 6.20.15 or the strength of the bolt or threaded rod shown in Tables 6.20.13 and 6.20.14.

An evaluation of published test results leads to certain characteristics common to most of the available inserts:[7]

1. Controlling strength conditions of various types of inserts are similar.
2. Pullout strength decreases with decreasing unit weight of concrete.
3. For inserts located in zones of potential flexural cracking, the pullout strength should be reduced by about 10%.

Table 6.20.20 Design strength of concrete brackets, corbels, or haunches

Design strength by Eqs. 6.11.3 or 6.11.4 for following criteria:

$f_y = 60{,}000$ psi

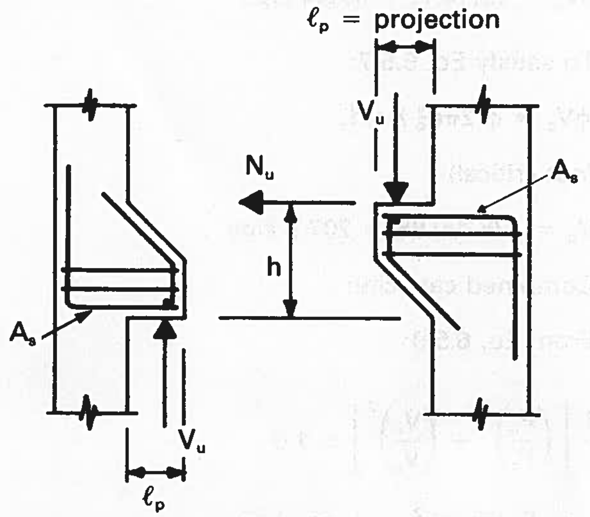
$N_u = 0.2\ V_u$

b = width of bearing

$a = 0.75\ \ell_p$

$d = h - 1.25$

$V_u = \phi V_n \leq 0.8\phi bd$



Values of $\phi\ V_n$, kips

b	A_s \ h	4″ Projection								6″ Projection								8″ Projection							
b = 6″	A_s \ h	4	6	8	10	12	14	16	18	6	8	10	12	14	16	18	20	8	10	12	14	16	18	10	22
	2–#4	11	19	28	35	40	44	47	48	17	22	27	31	35	38	41	0	18	22	26	29	32	35	0	0
	2–#5	11	19	28	36	44	52	60	65	19	28	36	44	52	60	65	67	28	34	40	45	50	55	59	62
	2–#6	11	19	28	36	44	52	60	68	19	28	36	44	52	60	68	77	28	36	44	52	60	68	77	85
	2–#7	11	19	28	36	44	52	60	68	19	28	36	44	52	60	68	77	28	36	44	52	60	68	77	85
	2–#8	11	19	28	36	44	52	60	68	19	28	36	44	52	60	68	77	28	36	44	52	60	68	77	85
	2–#9	11	19	28	36	44	52	60	68	19	28	36	44	52	60	68	77	28	36	44	52	60	68	77	85
b = 8″	A_s \ h	4	6	8	10	12	14	16	18	6	8	10	12	14	16	18	20	8	10	12	14	16	18	20	22
	2–#4	14	23	29	35	40	44	0	0	17	22	27	31	35	0	0	0	18	22	26	29	0	0	0	0
	2–#5	15	26	37	48	58	65	69	72	26	35	42	49	55	60	65	69	28	34	40	45	50	55	59	62
	2–#6	15	26	37	48	58	69	80.	90	26	37	48	58	69	80	90	94	37	48	58	65	72	79	84	90
	2–#7	15	26	37	48	58	69	80	91	26	37	48	58	69	80	91	102	37	48	58	69	80	91	102	113
	2–#8	15	26	37	48	58	69	80	91	26	37	48	58	69	80	91	102	37	48	58	69	80	91	102	113
	2–#9	15	26	37	48	58	69	80	91	26	37	48	58	69	80	91	102	37	48	58	69	80	91	102	113
b = 10″	A_s \ h	4	6	8	10	12	14	16	18	6	8	10	12	14	16	18	20	8	10	12	14	16	18	20	22
	2–#4	14	23	29	35	0	0	0	0	17	22	27	0	0	0	0	0	18	22	0	0	0	0	0	0
	2–#5	19	32	46	55	62	69	74	77	26	35	42	49	55	60	65	0	28	34	40	45	50	55	0	0
	2–#6	19	32	46	60	73	87	94	98	32	46	60	70	79	86	93	99	40	49	58	65	72	79	84	90
	2–#7	19	32	46	60	73	87	100	114	32	46	60	73	87	100	114	124	46	60	73	87	98	107	115	122
	2–#8	19	32	46	60	73	87	100	114	32	46	60	73	87	100	114	128	46	60	73	87	100	114	128	141
	2–#9	19	32	46	60	73	87	100	114	32	46	60	73	87	100	114	128	46	60	73	87	100	114	128	141
b = 12″	A_s \ h	4	6	8	10	12	14	16	18	6	8	10	12	14	16	18	20	8	10	12	14	16	18	20	22
	2–#4	14	23	29	0	0	0	0	0	17	22	0	0	0	0	0	0	18	0	0	0	0	0	0	0
	2–#5	22	35	46	55	62	69	0	0	26	35	42	49	55	0	0	0	28	34	40	45	0	0	0	0
	2–#6	22	39	55	71	88	96	100	105	38	50	61	70	79	86	93	99	40	49	58	65	72	79	84	90
	2–#7	22	39	55	71	88	104	120	128	39	55	71	88	104	117	127	133	54	67	78	89	98	107	115	122
	2–#8	22	39	55	71	88	104	120	137	39	55	71	88	104	120	137	153	55	71	88	104	120	137	150	160
	2–#9	22	39	55	71	88	104	120	137	39	55	71	88	104	120	137	153	55	71	88	104	120	137	153	169
	3–#4	22	34	44	53	60	66	0	0	25	33	40	47	52	0	0	0	27	33	38	44	0	0	0	0
	3–#5	22	39	55	71	88	98	103	107	39	52	63	73	82	90	97	104	42	51	60	68	75	82	88	94
	3–#6	22	39	55	71	88	104	120	136	39	55	71	88	104	120	136	141	55	71	87	98	108	118	127	135
	3–#7	22	39	55	71	88	104	120	137	39	55	71	88	104	120	137	153	55	71	88	104	120	137	153	169
	3–#8	22	39	55	71	88	104	120	137	39	55	71	88	104	120	137	153	55	71	88	104	120	137	153	169
	3–#9	22	39	55	71	88	104	120	137	39	55	71	88	104	120	137	153	55	71	88	104	120	137	153	169

Table 6.20.20 (continued) Design strength of concrete brackets, corbels, or haunches

		10" Projection								12" Projection								14" Projection							
	A_s \ h	10	12	14	16	18	20	22	24	12	14	16	18	20	22	24	26	14	16	18	20	22	24	26	28
b = 6"	2-#4	18	22	25	28	30	0	0	0	19	22	24	27	0	0	0	0	19	22	24	0	0	0	0	0
	2-#5	29	34	39	43	47	51	55	58	30	34	38	42	45	48	52	55	30	34	37	40	44	47	49	52
	2-#6	36	44	52	60	68	73	79	83	42	49	54	60	65	70	74	79	43	49	54	58	63	67	71	75
	2-#7	36	44	52	60	68	77	85	93	44	52	60	68	77	85	93	101	52	60	68	77	85	91	97	102
	2-#8	36	44	52	60	68	77	85	93	44	52	60	68	77	85	93	101	52	60	68	77	85	93	101	109
	2-#9	36	44	52	60	68	77	85	93	44	52	60	68	77	85	93	101	52	60	68	77	85	93	101	109
	A_s \ h	10	12	14	16	18	20	22	24	12	14	16	18	20	22	24	26	14	16	18	20	22	24	26	28
b = 8"	2-#4	18	22	25	0	0	0	0	0	19	22	0	0	0	0	0	0	19	0	0	0	0	0	0	0
	2-#5	29	34	39	43	47	51	55	0	30	34	38	42	45	48	0	0	30	34	37	40	44	0	0	0
	2-#6	42	49	56	62	68	73	79	83	42	49	54	60	65	70	74	79	43	49	54	58	63	67	71	75
	2-#7	48	58	69	80	91	100	107	113	58	66	74	82	88	95	101	107	59	66	73	79	85	91	97	102
	2-#8	48	58	69	80	91	102	113	124	58	69	80	91	102	113	124	135	69	80	91	102	112	119	126	133
	2-#9	48	58	69	80	91	102	113	124	58	69	80	91	102	113	124	135	69	80	91	102	113	124	135	146
	A_s \ h	10	12	14	16	18	20	22	24	12	14	16	18	20	22	24	26	14	16	18	20	22	24	26	28
b = 10"	2-#4	18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2-#5	29	34	39	43	47	0	0	0	30	34	38	42	0	0	0	0	30	34	37	0	0	0	0	0
	2-#6	42	49	56	62	68	73	79	83	42	49	54	60	65	70	74	79	43	49	54	58	63	67	71	0
	2-#7	56	67	76	85	93	100	107	113	58	66	74	82	88	95	101	107	59	66	73	79	85	91	97	102
	2-#8	60	73	87	100	114	128	140	148	73	87	97	106	116	124	132	140	77	86	95	104	112	119	126	133
	2-#9	60	73	87	100	114	128	141	155	73	87	100	114	128	141	155	168	87	100	114	128	141	151	160	168
	A_s \ h	10	12	14	16	18	20	22	24	12	14	16	18	20	22	24	26	14	16	18	20	22	24	26	28
b = 12"	2-#4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2-#5	29	34	39	0	0	0	0	0	30	34	0	0	0	0	0	0	30	0	0	0	0	0	0	0
	2-#6	42	49	56	62	68	73	79	0	42	49	54	60	65	70	0	0	43	49	54	58	63	0	0	0
	2-#7	56	67	76	85	93	100	107	113	58	66	74	82	88	95	101	107	59	66	73	79	85	91	97	102
	2-#8	71	87	99	110	121	131	140	148	76	87	97	106	116	124	132	140	77	86	95	104	112	119	126	133
	2-#9	71	88	104	120	137	153	169	186	88	104	120	135	146	157	167	177	97	109	120	131	141	151	160	168
	3-#4	28	33	37	0	0	0	0	0	28	32	0	0	0	0	0	0	29	0	0	0	0	0	0	0
	3-#5	43	51	58	65	71	77	82	0	44	51	57	62	68	73	0	0	45	51	56	61	65	0	0	0
	3-#6	62	73	84	93	102	110	118	125	64	73	82	90	97	105	111	118	65	73	80	87	94	101	107	112
	3-#7	71	88	104	120	137	150	160	170	87	99	111	122	133	142	152	160	88	99	109	119	128	137	145	153
	3-#8	71	88	104	120	137	153	169	186	88	104	120	137	153	169	186	202	104	120	137	153	167	179	189	200
	3-#9	71	88	104	120	137	153	169	186	88	104	120	137	153	169	186	202	104	120	137	153	169	186	202	218

Table 6.20.20 (continued) Design strength of concrete brackets, corbels, or haunches

		4" Projection								6" Projection								8" Projection							
b	A_s \ h	4	6	8	10	12	14	16	18	6	8	10	12	14	16	18	20	8	10	12	14	16	18	20	22
b = 14"	2–#4	14	23	29	0	0	0	0	0	17	22	0	0	0	0	0	0	18	0	0	0	0	0	0	0
	2–#5	23	35	46	55	62	0	0	0	26	35	42	49	0	0	0	0	28	34	40	0	0	0	0	0
	2–#6	26	45	64	79	90	99	106	110	38	50	61	70	79	86	93	0	40	49	58	65	72	79	0	0
	2–#7	26	45	64	83	102	121	129	135	45	64	83	96	107	117	127	135	54	67	78	89	98	107	115	122
	2–#8	26	45	64	83	102	121	140	159	45	64	83	102	121	140	159	166	64	83	102	116	128	140	150	160
	2–#9	26	45	64	83	102	121	140	159	45	64	83	102	121	140	159	179	64	83	102	121	140	159	179	198
	3–#4	22	34	44	53	60	0	0	0	25	33	40	47	0	0	0	0	27	33	38	0	0	0	0	0
	3–#5	26	45	64	82	93	103	109	113	39	52	63	73	82	90	97	0	42	51	60	68	75	82	0	0
	3–#6	26	45	64	83	102	121	137	143	45	64	83	102	118	129	140	149	60	74	87	98	108	118	127	135
	3–#7	26	45	64	83	102	121	140	159	45	64	83	102	121	140	159	179	64	83	102	121	140	159	173	184
	3–#8	26	45	64	83	102	121	140	159	45	64	83	102	121	140	159	179	64	83	102	121	140	159	179	198
	3–#9	26	45	64	83	102	121	140	159	45	64	83	102	121	140	159	179	64	83	102	121	140	159	179	198
b	A_s \ h	4	6	8	10	12	14	16	18	6	8	10	12	14	16	18	20	8	10	12	14	16	18	20	22
b = 16"	2–#6	30	51	66	79	90	99	107	0	38	50	61	70	79	86	0	0	40	49	58	65	72	0	0	0
	2–#7	30	52	73	95	117	129	136	141	51	68	83	96	107	117	127	135	54	67	78	89	98	107	115	122
	2–#8	30	52	73	95	117	139	160	168	52	73	95	117	139	153	166	174	71	88	103	116	128	140	150	160
	2–#9	30	52	73	95	117	139	160	182	52	73	95	117	139	160	182	202	73	95	117	139	160	177	190	202
	3–#6	30	52	73	95	117	137	144	151	52	73	91	105	118	129	140	149	60	74	87	98	108	118	127	135
	3–#7	30	52	73	95	117	139	160	182	52	73	95	117	139	160	182	190	73	95	117	133	148	161	173	184
	3–#8	30	52	73	95	117	139	160	182	52	73	95	117	139	160	182	204	73	95	117	139	160	182	204	226
	3–#9	30	52	73	95	117	139	160	182	52	73	95	117	139	160	182	204	73	95	117	139	160	182	204	226
	4–#6	30	52	73	95	117	139	160	181	52	73	95	117	139	160	181	188	73	95	115	131	144	157	169	180
	4–#7	30	52	73	95	117	139	160	182	52	73	95	117	139	160	182	204	73	95	117	139	160	182	204	226
	4–#8	30	52	73	95	117	139	160	182	52	73	95	117	139	160	182	204	73	95	117	139	160	182	204	226
	4–#9	30	52	73	95	117	139	160	182	52	73	95	117	139	160	182	204	73	95	117	139	160	182	204	226
b	A_s \ h	4	6	8	10	12	14	16	18	6	8	10	12	14	16	18	20	8	10	12	14	16	18	20	22
b = 18"	2–#6	33	51	66	79	90	99	0	0	38	50	61	70	79	0	0	0	40	49	58	65	0	0	0	0
	2–#7	34	58	83	107	122	135	141	147	51	68	83	96	107	117	127	135	54	67	78	89	98	107	115	0
	2–#8	34	58	83	107	132	156	168	175	58	83	107	125	140	153	166	177	71	88	103	116	128	140	150	160
	2–#9	34	58	83	107	132	156	181	203	58	83	107	132	156	181	203	211	83	107	130	147	163	177	190	202
	3–#6	34	58	83	107	132	143	151	157	56	75	91	105	118	129	140	149	60	74	87	98	108	118	127	135
	3–#7	34	58	83	107	132	156	181	191	58	83	107	132	156	176	190	199	82	101	118	133	148	161	173	184
	3–#8	34	58	83	107	132	156	181	205	58	83	107	132	156	181	205	230	83	107	132	156	181	205	225	240
	3–#9	34	58	83	107	132	156	181	205	58	83	107	132	156	181	205	230	83	107	132	156	181	205	230	254
	4–#6	34	58	83	107	132	156	181	189	58	83	107	132	156	173	186	196	80	99	115	131	144	157	169	180
	4–#7	34	58	83	107	132	156	181	205	58	83	107	132	156	181	205	230	83	107	132	156	181	205	230	245
	4–#8	34	58	83	107	132	156	181	205	58	83	107	132	156	181	205	230	83	107	132	156	181	205	230	254
	4–#9	34	58	83	107	132	156	181	205	58	83	107	132	156	181	205	230	83	107	132	156	181	205	230	254
b	A_s \ h	4	6	8	10	12	14	16	18	6	8	10	12	14	16	18	20	8	10	12	14	16	18	20	22
b = 20"	2–#6	33	51	66	79	90	0	0	0	38	50	61	70	0	0	0	0	40	49	58	0	0	0	0	0
	2–#7	37	65	90	107	122	135	146	153	51	68	83	96	107	117	127	0	54	67	78	89	98	107	0	0
	2–#8	37	65	92	119	146	166	174	182	65	89	108	125	140	153	166	177	71	88	103	116	128	140	150	160
	2–#9	37	65	92	119	146	173	201	211	65	92	119	146	173	194	210	220	90	111	130	147	163	177	190	202
	3–#6	37	65	92	118	135	149	156	163	56	75	91	105	118	129	140	0	60	74	87	98	108	118	0	0
	3–#7	37	65	92	119	146	173	190	199	65	92	119	143	161	176	190	203	82	101	118	133	148	161	173	184
	3–#8	37	65	92	119	146	173	201	228	65	92	119	146	173	201	228	245	92	119	146	173	193	210	225	240
	3–#9	37	65	92	119	146	173	201	228	65	92	119	146	173	201	228	255	92	119	146	173	201	228	255	282
	4–#6	37	65	92	119	146	173	188	196	65	92	119	140	157	173	186	199	80	99	115	131	144	157	169	180
	4–#7	37	65	92	119	146	173	201	228	65	92	119	146	173	201	228	248	92	119	146	173	197	214	230	245
	4–#8	37	65	92	119	146	173	201	228	65	92	119	146	173	201	228	255	92	119	146	173	201	228	255	282
	4–#9	37	65	92	119	146	173	201	228	65	92	119	146	173	201	228	255	92	119	146	173	201	228	255	282

Table 6.20.20 (continued) Design strength of concrete brackets, corbels, or haunches

b	A_s \ h	10″ Projection								12″ Projection								14″ Projection							
		10	12	14	16	18	20	22	24	12	14	16	18	20	22	24	26	14	16	18	20	22	24	26	28
b = 14″	2–#4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2–#5	29	34	0	0	0	0	0	0	30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2–#6	42	49	56	62	68	0	0	0	42	49	54	60	0	0	0	0	43	49	54	0	0	0	0	0
	2–#7	56	67	76	85	93	100	107	113	58	66	74	82	88	95	101	0	59	66	73	79	85	91	0	0
	2–#8	74	87	99	110	121	131	140	148	76	87	97	106	116	124	132	140	77	86	95	104	112	119	126	133
	2–#9	83	102	121	140	153	165	177	188	96	110	123	135	146	157	167	177	97	109	120	131	141	151	160	168
	3–#4	28	33	0	0	0	0	0	0	28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	3–#5	43	51	58	65	71	0	0	0	44	51	57	62	0	0	0	0	45	51	56	0	0	0	0	0
	3–#6	62	73	84	93	102	110	118	125	64	73	82	90	97	105	111	118	65	73	80	87	94	101	107	112
	3–#7	83	100	114	127	139	150	160	170	87	99	111	122	133	142	152	160	88	99	109	119	128	137	145	153
	3–#8	83	102	121	140	159	179	198	217	102	121	140	159	173	186	198	209	115	129	143	155	167	179	189	200
	3–#9	83	102	121	140	159	179	198	217	102	121	140	159	179	198	217	236	121	140	159	179	198	217	236	253
	A_s \ h	10	12	14	16	18	20	22	24	12	14	16	18	20	22	24	26	14	16	18	20	22	24	26	28
b = 16″	2–#6	42	49	56	62	0	0	0	0	42	49	54	0	0	0	0	0	43	49	0	0	0	0	0	0
	2–#7	56	67	76	85	93	100	107	0	58	66	74	82	88	95	0	0	59	66	73	79	85	0	0	0
	2–#8	74	87	99	110	121	131	140	148	76	87	97	106	116	124	132	140	77	86	95	104	112	119	126	133
	2–#9	93	110	126	140	153	165	177	188	96	110	123	135	146	157	167	177	97	109	120	131	141	151	160	168
	3–#6	62	73	84	93	102	110	118	125	64	73	82	90	97	105	111	0	65	73	80	87	94	101	0	0
	3–#7	85	100	114	127	139	150	160	170	87	99	111	122	133	142	152	160	88	99	109	119	128	137	145	153
	3–#8	95	117	139	160	181	196	210	222	113	130	145	160	173	186	198	209	115	129	143	155	167	179	189	200
	3–#9	95	117	139	160	182	204	226	248	117	139	160	182	204	226	248	265	139	160	181	197	212	226	240	253
	4–#6	83	98	112	124	136	147	157	167	85	97	109	120	130	140	149	157	86	97	107	117	126	134	142	150
	4–#7	95	117	139	160	182	200	214	227	116	133	148	163	177	190	202	214	118	132	146	159	171	182	193	204
	4–#8	95	117	139	160	182	204	226	248	117	139	160	182	204	226	248	269	139	160	182	204	223	238	253	266
	4–#9	95	117	139	160	182	204	226	248	117	139	160	182	204	226	248	269	139	160	182	204	226	248	269	291
	A_s \ h	10	12	14	16	18	20	22	24	12	14	16	18	20	22	24	26	14	16	18	20	22	24	26	28
b = 18″	2–#6	42	49	56	0	0	0	0	0	42	49	0	0	0	0	0	0	43	0	0	0	0	0	0	0
	2–#7	56	67	76	85	93	100	0	0	58	66	74	82	88	0	0	0	59	66	73	79	0	0	0	0
	2–#8	74	87	99	110	121	131	140	148	76	87	97	106	116	124	132	140	77	86	95	104	112	119	126	0
	2–#9	93	110	126	140	153	165	177	188	96	110	123	135	146	157	167	177	97	109	120	131	141	151	160	168
	3–#6	62	73	84	93	102	110	118	0	64	73	82	90	97	105	0	0	65	73	80	87	94	0	0	0
	3–#7	85	100	114	127	139	150	160	170	87	99	111	122	133	142	152	160	88	99	109	119	128	137	145	153
	3–#8	107	130	149	166	181	196	210	222	113	130	145	160	173	186	198	209	115	129	143	155	167	179	189	200
	3–#9	107	132	156	181	205	230	254	278	132	156	181	202	219	236	251	265	146	164	181	197	212	226	240	253
	4–#6	83	98	112	124	136	147	157	167	85	97	109	120	130	140	149	157	86	97	107	117	126	134	142	150
	4–#7	107	132	152	169	185	200	214	227	116	133	148	163	177	190	202	214	118	132	146	159	171	182	193	204
	4–#8	107	132	156	181	205	230	254	278	132	156	181	205	230	248	264	279	154	173	190	207	223	238	253	266
	4–#9	107	132	156	181	205	230	254	278	132	156	181	205	230	254	278	303	156	181	205	230	254	278	303	327
	A_s \ h	10	12	14	16	18	20	22	24	12	14	16	18	20	22	24	26	14	16	18	20	22	24	26	28
b = 20″	2–#6	42	49	0	0	0	0	0	0	42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2–#7	56	67	76	85	93	0	0	0	58	66	74	82	0	0	0	0	59	66	73	0	0	0	0	0
	2–#8	74	87	99	110	121	131	140	0	76	87	97	106	116	124	0	0	77	86	95	104	112	0	0	0
	2–#9	93	110	126	140	153	165	177	188	96	110	123	135	146	157	167	177	97	109	120	131	141	151	160	168
	3–#6	62	73	84	93	102	0	0	0	64	73	82	90	0	0	0	0	65	73	80	0	0	0	0	0
	3–#7	85	100	114	127	139	150	160	170	87	99	111	122	133	142	152	160	88	99	109	119	128	137	145	0
	3–#8	111	130	149	166	181	196	210	222	113	130	145	160	173	186	198	209	115	129	143	155	167	179	189	200
	3–#9	119	146	173	201	228	248	265	281	143	164	184	202	219	236	251	265	146	164	181	197	212	226	240	253
	4–#6	83	98	112	124	136	147	157	167	85	97	109	120	130	140	149	157	86	97	107	117	126	134	142	0
	4–#7	113	133	152	169	185	200	214	227	116	133	148	163	177	190	202	214	118	132	146	159	171	182	193	204
	4–#8	119	146	173	201	228	255	279	296	146	173	194	213	231	248	264	279	154	173	190	207	223	238	253	266
	4–#9	119	146	173	201	228	255	282	309	146	173	201	228	255	282	309	337	173	201	228	255	282	302	320	337

Table 6.20.20 (continued) Design strength of concrete brackets, corbels, or haunches

b	A_s \ h	4″ Projection								6″ Projection								8″ Projection							
		4	6	8	10	12	14	16	18	6	8	10	12	14	16	18	20	8	10	12	14	16	18	20	22
b = 22″	2–#6	33	51	66	79	90	0	0	0	38	50	61	70	0	0	0	0	40	49	58	0	0	0	0	0
	2–#7	41	69	90	107	122	135	146	0	51	68	83	96	107	117	0	0	54	67	78	89	98	0	0	0
	2–#8	41	71	101	131	159	172	180	188	67	89	108	125	140	153	166	177	71	88	103	116	128	140	150	0
	2–#9	41	71	101	131	161	191	210	219	71	101	131	158	177	194	210	224	90	111	130	147	163	177	190	202
	3–#6	41	71	99	118	135	149	161	168	56	75	91	105	118	129	140	0	60	74	87	98	108	118	0	0
	3–#7	41	71	101	131	161	188	197	206	71	101	124	143	161	176	190	203	82	101	118	133	148	161	173	184
	3–#8	41	71	101	131	161	191	221	244	71	101	131	161	191	221	244	253	101	131	154	174	193	210	225	240
	3–#9	41	71	101	131	161	191	221	251	71	101	131	161	191	221	251	281	101	131	161	191	221	251	281	303
	4–#6	41	71	101	131	161	185	195	203	71	100	121	140	157	173	186	199	80	99	115	131	144	157	169	180
	4–#7	41	71	101	131	161	191	221	247	71	101	131	161	191	221	247	257	101	131	157	178	197	214	230	245
	4–#8	41	71	101	131	161	191	221	251	71	101	131	161	191	221	251	281	101	131	161	191	221	251	281	310
	4–#9	41	71	101	131	161	191	221	251	71	101	131	161	191	221	251	281	101	131	161	191	221	251	281	310
	A_s \ h	4	6	8	10	12	14	16	18	6	8	10	12	14	16	18	20	8	10	12	14	16	18	20	22
b = 24″	2–#6	33	51	66	79	0	0	0	0	38	50	61	0	0	0	0	0	40	49	0	0	0	0	0	0
	2–#7	44	69	90	107	122	135	0	0	51	68	83	96	107	0	0	0	54	67	78	89	0	0	0	0
	2–#8	45	78	110	140	159	176	186	194	67	89	108	125	140	153	166	0	71	88	103	116	128	140	0	0
	2–#9	45	78	110	143	175	206	216	226	78	110	136	158	177	194	210	224	90	111	130	147	163	177	190	202
	3–#6	45	76	99	118	135	149	161	0	56	75	91	105	118	129	0	0	60	74	87	98	108	0	0	0
	3–#7	45	78	110	143	175	194	203	212	77	102	124	143	161	176	190	203	82	101	118	133	148	161	173	184
	3–#8	45	78	110	143	175	208	241	252	78	110	143	175	208	230	248	262	107	131	154	174	193	210	225	240
	3–#9	45	78	110	143	175	208	241	273	78	110	143	175	208	241	273	304	110	143	175	208	241	265	285	303
	4–#6	45	78	110	143	175	191	201	209	75	100	121	140	157	173	186	199	80	99	115	131	144	157	169	180
	4–#7	45	78	110	143	175	208	241	255	78	110	143	175	208	235	254	265	109	134	157	178	197	214	230	245
	4–#8	45	78	110	143	175	208	241	273	78	110	143	175	208	241	273	306	110	143	175	208	241	273	300	320
	4–#9	45	78	110	143	175	208	241	273	78	110	143	175	208	241	273	306	110	143	175	208	241	273	306	339

b	A_s \ h	10″ Projection								12″ Projection								14″ Projection							
		10	12	14	16	18	20	22	24	12	14	16	18	20	22	24	26	14	16	18	20	22	24	26	28
b = 22″	2–#6	42	49	0	0	0	0	0	0	42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2–#7	56	67	76	85	0	0	0	0	58	66	74	0	0	0	0	0	59	66	0	0	0	0	0	0
	2–#8	74	87	99	110	121	131	0	0	76	87	97	106	116	0	0	0	77	86	95	104	0	0	0	0
	2–#9	93	110	126	140	153	165	177	188	96	110	123	135	146	157	167	177	97	109	120	131	141	151	160	0
	3–#6	62	73	84	93	102	0	0	0	64	73	82	90	0	0	0	0	65	73	80	0	0	0	0	0
	3–#7	85	100	114	127	139	150	160	170	87	99	111	122	133	142	152	0	88	99	109	119	128	137	0	0
	3–#8	111	130	149	166	181	196	210	222	113	130	145	160	173	186	198	209	115	129	143	155	167	179	189	200
	3–#9	131	161	188	210	229	248	265	281	143	164	184	202	219	236	251	265	146	164	181	197	212	226	240	253
	4–#6	83	98	112	124	136	147	157	167	85	97	109	120	130	140	149	0	86	97	107	117	126	134	0	0
	4–#7	113	133	152	169	185	200	214	227	116	133	148	163	177	190	202	214	118	132	146	159	171	182	193	204
	4–#8	131	161	191	221	242	261	279	296	151	173	194	213	231	248	264	279	154	173	190	207	223	238	253	266
	4–#9	131	161	191	221	251	281	310	340	161	191	221	251	281	310	334	353	191	218	241	262	282	302	320	337
	A_s \ h	10	12	14	16	18	20	22	24	12	14	16	18	20	22	24	26	14	16	18	20	22	24	26	28
b = 24″	2–#6	42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2–#7	56	67	76	0	0	0	0	0	58	66	0	0	0	0	0	0	59	0	0	0	0	0	0	0
	2–#8	74	87	99	110	121	0	0	0	76	87	97	106	0	0	0	0	77	86	95	0	0	0	0	0
	2–#9	93	110	126	140	153	165	177	188	96	110	123	135	146	157	167	0	97	109	120	131	141	151	0	0
	3–#6	62	73	84	93	0	0	0	0	64	73	82	0	0	0	0	0	65	73	0	0	0	0	0	0
	3–#7	85	100	114	127	139	150	160	0	87	99	111	122	133	142	0	0	88	99	109	119	128	0	0	0
	3–#8	111	130	149	166	181	196	210	222	113	130	145	160	173	186	198	209	115	129	143	155	167	179	189	200
	3–#9	140	165	188	210	229	248	265	281	143	164	184	202	219	236	251	265	146	164	181	197	212	226	240	253
	4–#6	83	98	112	124	136	147	157	0	85	97	109	120	130	140	0	0	86	97	107	117	126	0	0	0
	4–#7	113	133	152	169	185	200	214	227	116	133	148	163	177	190	202	214	118	132	146	159	171	182	193	204
	4–#8	143	174	198	221	242	261	279	296	151	173	194	213	231	248	264	279	154	173	190	207	223	238	253	266
	4–#9	143	175	208	241	273	306	339	371	175	208	241	270	292	314	334	353	194	218	241	262	282	302	320	337