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

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 \( k\varepsilon_0/r = 150 \).

p. 3-31 — Eq. 3.7.2 — The quantity \( r/\Sigma r \) 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, \( A_f \) or \( A'_{fy} \), should be included in the calculation of \( T \) or \( C \).

p. 4-30 — Left column, change bottom equation to
\[
T_e = \frac{0.8\sqrt{5000 \text{ (3680)}}}{1000} \frac{1}{\sqrt{1 + \left( \frac{0.4(53.1)}{0.1125(376)} \right) \lambda}} = 186 \text{ in.-kips}
\]
calculations in right column change accordingly as indicated on the corrected p. 4-30.

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

p. 4-40 — Sect. 4.5.3 — coefficients \( K_{es} \) and \( K_{cr} \) are for pretensioned members as are \( K_{air} \) and \( K_{eh} \).

pp. 4-44 and 4-49 — Change coefficient in Eq. 4.6.2 and in equation for \( l_c \), 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.
\[
\Delta_f = (1.80 - 1.24) + 2.36 = 2.92 \text{ in} < 3.50 \text{ in.} \text{ OK}
\]

p. 4-61 — In equations 1 (Design), and 3 (Analysis), change \( d_0 \) to \( d' \).

p. 4-62 — The tabular values for concrete strengths of 5000 psi and greater are incorrect. Also, in sketch, change \( d \) to \( d_0 \); second note, change \( \omega_{mu} > 0.366 \beta_1 \) to \( \omega_0 > 0.36 \beta_1 \); 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 \( d_0 \).

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

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 \( d_0 \), or 0.3 \( d_0 \), 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 \( P_{ct} \), change \( \ell \) to \( \ell_c \).

p. 6-8 — Left column, in Eq. 6.5.5 and in line immediately following Eq. 6.5.5, change \( f_v \) 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 $\bar{x}$, 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 $\lambda$ should read: $= f_u/\sqrt{f_c^2}$

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 $\varepsilon_a$. For (c), $\varepsilon_a$ 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 $\phi$ as indicated. Change $\phi P_c$ 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 — The tables do not contain the limitation through 6-67

of Eq. 6.11.7, $\leq 800$ bd. This will frequently control, especially on small corbels.

Table 6.20.20 — Add to criteria:

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

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

For 270-ksi, 7-wire strand —

<table>
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<tr>
<th>Value</th>
<th>0.217</th>
<th>0.74</th>
<th>41.0</th>
<th>44.0</th>
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</table>
| For 250-ksi, 7-wire strand —

<table>
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<th>Value</th>
<th>0.216</th>
<th>0.74</th>
<th>37.8</th>
<th>43.2</th>
<th>54.0</th>
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</table>


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:

$$M_u \leq \phi M_n = K'_u \frac{bd'^2}{12,000}$$

$$\text{Req'd } K'_u = \frac{M_u (12,000)}{\frac{bd'^2}{12,000}} = \frac{600(12,000)}{16(21)^2} = 1020$$

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

$= 0.24$, $K'_u = 1045$

therefore

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

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

Use 12 – 1/2” diameter strands; $A_{ps} = 1.84 \text{ sq in.}$

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

**Given:**
- 3'-4" x 8" hollow-core slab
- $h_t = 1 1/4''$
- $d = 7''$

**Concrete:**
- $f'_c = 5000$ psi normal weight concrete
- Prestressing steel:
  - 10 - 3/8" diameter 250K stress-relieved strand
- $A_{ps} = 10(0.080) = 0.800 \text{ sq in.}$

**Section properties:**
- $A = 218 \text{ in}^2$
- $Z_b = 381 \text{ in}^3$
- $Y_b = 3.98 \text{ in.}$

**Problem:**
Find design flexural strength, $\phi M_n$.

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

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

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, $f_{ps} = 150$ 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:

$$\phi M_n = \phi \left( A_{ps} f_{ps} (d_p - a/2) + A_t f_t (d - a/2) \right)$$

a = $(A_{ps} f_{ps} + A_t f_t)/(0.85 f'_c b)$

Since $A_t = 0$:

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

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

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

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

$$1.2 M_{cr} = 1.2 \left( \frac{P}{A} + \frac{P e}{Z_b} + 7.5 \sqrt{f'_c} Z_b \right)$$

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

$$+ 7.5 \sqrt{5000} \right) 381$$

$$= 923 \text{ in-kips} = 76.9 \text{ ft-kips} < 93.0 \text{ ft-kips OK}$$
Span of spandrel beam = 30 ft clear
f'c = 5000 psi, normal weight concrete
Reinforcement fy = 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
- wu = 5.31

Problem:
Determine torsion reinforcement requirements

Solution:
1. Compute torsion moment (Tu) at critical section, assumed to be 5'-0" from face of support:
   \[ V_u = w_u(15 - 5) = 53.1(10) = 531 \text{ kips} \]
   \[ \sigma_u \text{ for torsion} = 1.68 + 0.70 + 0.28 = 4.96 \text{kips/ft} \]
   Eccentricity = 2/3(8) + 3.29 = 8.62 in.
   \[ T_u = w_u(\epsilon)(e/2 - 5) = 4.36(8.62)(10) = 376 \text{ in.-kips} \]

2. Determine if torsion effects must be considered.
   If \( T_u \geq \phi(0.5 \sqrt{f'_c} \Sigma x^2 y) \) must consider torsion.
   \[ \Sigma x^2 y = 6^2(72) + 6^2(8)(2) + 8^2(8) = 3680 \text{ in.}^3 \]
   \[ \phi(0.5 \sqrt{f'_c} \Sigma x^2 y) = \frac{0.85(0.5)\sqrt{5000}}{1000} = 0.85(0.5)\sqrt{5000} = 3680 \text{ in.-kips} \]
   376 > 110.6, consider torsion

3. Determine the torsion moment strength provided by concrete

   \[ T_o = \frac{0.8 \sqrt{f'_c \Sigma x^2 y}}{1 + \left( \frac{0.4 V_o}{C \sigma_u} \right)} \]

   where:
   \[ C_t = \frac{d b_w}{\Sigma x^2 y} = \frac{6(69)}{3680} = 0.1125 \]
   \[ T_o = \frac{0.8 \sqrt{5000}}{1000} \]

4. Determine torsion reinforcement requirements:
   \[ T_o = \phi T_n = \phi(T_o + T_s) \]
   or
   \[ T_o = \frac{T_u}{\phi} - T_c = \frac{376}{0.85} - 186 = 256 \text{ in.-kips} \]

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
\[ T_o = \frac{A_t \alpha x_y y_f}{s} \]

Assume \( x_t = 4 \text{ in.}; y_t = 70 \text{ in.} \)
\[ \alpha_t = 0.66 + 0.33(y_t/x_t) = 1.5 \]
\[ = 0.66 + 0.33(70/4) = 6.44, \text{ use } \alpha_t = 1.5 \]
\[ A_t = \frac{256(12)}{1.5(4)(70)(60)} = 0.12 \text{ sq in./ft} \]
\[ = 0.01 \text{ sq in./in.} \]

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 \( A_t \). The minimum area of closed stirrups is:
\[ A_t + 2A_t = 50b_w/s_f \]

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

\[
\Delta_d = \frac{5w(c^4)}{384 E_d} = \frac{5 \cdot (0.080)}{384 \cdot (4287)(20,985)} = 0.48 \text{ in.} \\
\Delta_f = 2.36 \text{ in.} \downarrow \text{ (see Example 4.6.3)}
\]

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

The estimated critical cambers and deflections would then be:

- At erection of the member after \( w_{ed} \) is applied: \( 1.80 \text{ in.} \)
- "Final" long-time camber: \( 1.24 \text{ in.} \)

The deflection limitation of Table 9.5(b) for the above condition is \( \frac{\ell}{240} \).

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

Total deflection occurring after attachment of non-structural elements:

\[
\Delta_f = (1.80 - 1.24) + 2.36 = 2.92 \text{ in.} < 3.50 \text{ in.} \text{ OK}
\]

### 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\(^1\) (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 \( P_n \) for \( M_n = 0 \). (See Fig. 4.7.1(c))

**Step 2:** Determine \( M_n \) 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).
**FLEXURE**

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

**Procedure:**

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

2. Find $K'_u$ from table

3. Determine $\phi M_n = K'_u \frac{b d_p^2}{12000}$ (ft-kips)

**Basis:**

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

**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 $\phi M_n = \phi [f_{ps} b d_p^2 (0.36 \beta_1 - 0.08 \beta_3)]$

**Values of $K'_u$**

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<th>$f'_c$</th>
<th>$\omega_{pu}$</th>
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<th>.01</th>
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<th>.03</th>
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Problem:
Find the maximum shear strength.

Solution:
\[ A_b = 0.20 \text{ sq in.} \]

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

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

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

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

Maximum capacity of the group:
1. \( 10(1510) = 15,000 \text{ lb} \)
2. \( 4(1510)(3) = 18,120 \text{ lb} \)
   or \( 2(4624)(3) = 27,744 \text{ lb} \)
3. \( 4(9436) = 37,744 \text{ lb} \)

Thus condition 1 controls.

The design shear strength as governed by steel strength is:
\[ \phi V_s = 0.75 A_b f_s = 45,000 A_b \] (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:

**Concrete:**
\[ \phi \left[ \frac{P_u}{P_c} \right]^2 + \left( \frac{V_u}{V_c} \right)^2 \leq 1.0 \] (Eq. 6.5.9)

where \( \phi = 0.85 \)

**Steel:**
\[ \phi \left[ \left( \frac{P_u}{P_s} \right)^2 + \left( \frac{V_u}{V_s} \right)^2 \right] \leq 1.0 \] (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 \( \frac{3}{4} \) of the diameter of the stud.

Example 6.5.2 Capacity of welded headed studs

**Given:**
Bracket on column as shown.
\( f' = 5000 \text{ psi (normal weight)} \)
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:
\( d_e = 5 \text{ in.}, \ell_e = 6 \text{ in.}, \frac{s}{d} \text{ in. studs} \)
\[ P_1 = 6(27.4) = 164.4 \text{ kips} \]
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):
\[ y = 3 \text{ in.}, x = 16 \text{ in.}, \ell_e = 6 \text{ in.} \]
\[ P_{c1} = 67.5 \text{ kips} \]
\[ \phi P_{c1} = 0.85(67.5) = 57.4 \text{ kips} \]
This is the capacity of a truncated pyramid accounting for the stud spacing and controls the design.

A moment-resisting couple is formed:
\[ C = T = 0.85 f' \cdot ba = 57.4 \text{ kips} \]

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

\[ j_o d = 11 - 1.35/2 = 10.3 \text{ in.} \]

\[ P_u = T + N_u = M/\ell_o d + N_u = 75(6)/10.3 + 12 = 55.7 \text{ kips} \]

Check shear (all studs):
From Table 6.20.7:
\[ f_c' = 5000 \text{ psi}, \ d_e > 9 \text{ in.}, 5/8 \text{ in. studs} \]
\[ \phi V_c = 12(14.7) = 176.4 \text{ kips} \]
To satisfy Eq. 6.5.7:
\[ \phi V_c = \phi 2\pi d_e' \sqrt{f_c} \]
(not critical)
\[ V_c = 176.4/0.85 = 207.5 \text{ kips} \]
Combined capacity:

From Eq. 6.5.9:
\[ 1 \left[ \left( \frac{P_u}{P_c} \right)^2 + \left( \frac{V_u}{V_c} \right)^2 \right] \leq 1.0 \]
\[ \frac{1}{0.85} \left[ \left( \frac{55.7}{67.5} \right)^2 + \left( \frac{75}{207.5} \right)^2 \right] \]
\[ = 0.95 < 1.0 \text{ OK} \]
(b) Check steel strength:

Tension in top group of studs:
From Table 6.20.6 for 5/8 in. studs:
\[ P_u = 6(16.6) = 99.6 \text{ kips} \]
(Could also be determined from Eq. 6.5.5)
\[ C = T = 0.85 f_c' d_e = 99.6 \text{ kips} \]
comp. block, \( a = \frac{99.6}{0.85(5)(10)} = 2.34 \text{ in.} \)
\[ j, d = 11 - 2.34/2 = 9.83 \text{ in.} \]
\[ P_u = M_u/j, d + N_u = 75(6)/9.83 + 12 = 57.8 \text{ kips} \]

Shear in studs:
From Table 6.20.7 for 5/8 in. studs:
\[ V_s = 12(13.8) = 165.6 \text{ kips} \]
(Could also be determined from Eq. 6.5.8)

Combined capacity:
From Eq. 6.5.10:
\[ \left( \frac{57.8}{99.6} \right)^2 + \left( \frac{75}{165.6} \right)^2 \]
\[ = 0.34 + 0.21 = 0.55 < 1.0 \text{ OK} \]

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:

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

<table>
<thead>
<tr>
<th></th>
<th>4&quot; Projection</th>
<th>6&quot; Projection</th>
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<td>6 8 10 12 14 16 18</td>
<td>6 8 10 12 14 16 18</td>
<td>6 8 10 12 14 16 18</td>
</tr>
</tbody>
</table>

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

- \( f_c = 60,000 \text{ psi} \)
- \( N_o = 0.2 V_o \)
- \( b = \text{width of bearing} \)
- \( a = 0.75 \ell_p \)
- \( d = h - 1.25 \)
- \( V_o = \phi V'_n < 0.8bd \)

Values of \( \phi V'_n \), kips

<table>
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<td>6 8 10 12 14 16 18</td>
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</table>

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

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**Note:** The table continues with similar entries for different values of h and A, with projections for 10", 12", and 14".
### Table 6.20.20 (continued) Design strength of concrete brackets, corbels, or haunches

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

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<th>A₁</th>
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<td>14&quot; Projection</td>
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</table>

- **4" Projection**
  - **A₁**
  - **h**
  - **4** | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22
  - **6** | 38 | 50 | 61 | 70 | 0 | 0 | 0 | 0 | 0
  - **8** | 51 | 68 | 83 | 96 | 107 | 0 | 0 | 0 | 0
  - **10** | 67 | 89 | 108 | 125 | 140 | 153 | 166 | 177 | 0
  - **12** | 71 | 101 | 131 | 158 | 177 | 194 | 210 | 224 | 0
  - **14** | 76 | 75 | 91 | 105 | 118 | 129 | 140 | 0 | 0
  - **16** | 71 | 101 | 124 | 143 | 161 | 176 | 190 | 203 | 0
  - **18** | 82 | 101 | 131 | 148 | 163 | 177 | 190 | 202 | 0
  - **20** | 80 | 99 | 115 | 131 | 144 | 157 | 169 | 180 | 0
  - **22** | 101 | 131 | 161 | 191 | 221 | 251 | 281 | 310 | 0

- **6" Projection**
  - **A₁**
  - **h**
  - **6** | 33 | 51 | 66 | 79 | 0 | 0 | 0 | 0 | 0
  - **8** | 45 | 78 | 110 | 140 | 159 | 176 | 186 | 194 | 0
  - **10** | 78 | 110 | 136 | 158 | 177 | 194 | 210 | 224 | 0
  - **12** | 85 | 101 | 121 | 140 | 157 | 173 | 186 | 199 | 0
  - **14** | 76 | 75 | 91 | 106 | 118 | 129 | 0 | 0 | 0
  - **16** | 82 | 102 | 124 | 143 | 161 | 176 | 190 | 203 | 0
  - **18** | 107 | 131 | 154 | 174 | 193 | 210 | 225 | 240 | 0
  - **20** | 110 | 143 | 175 | 206 | 230 | 258 | 285 | 303 | 0
  - **22** | 90 | 111 | 130 | 147 | 163 | 177 | 190 | 202 | 0

- **8" Projection**
  - **A₁**
  - **h**
  - **6** | 33 | 51 | 66 | 79 | 0 | 0 | 0 | 0 | 0
  - **8** | 45 | 78 | 110 | 140 | 159 | 176 | 186 | 194 | 0
  - **10** | 85 | 110 | 136 | 158 | 177 | 194 | 210 | 224 | 0
  - **12** | 76 | 75 | 91 | 106 | 118 | 129 | 0 | 0 | 0
  - **14** | 82 | 102 | 124 | 143 | 161 | 176 | 190 | 203 | 0
  - **16** | 107 | 131 | 154 | 174 | 193 | 210 | 225 | 240 | 0
  - **18** | 110 | 143 | 175 | 206 | 230 | 258 | 285 | 303 | 0
  - **20** | 90 | 111 | 130 | 147 | 163 | 177 | 190 | 202 | 0

- **10" Projection**
  - **A₁**
  - **h**
  - **6** | 33 | 51 | 66 | 79 | 0 | 0 | 0 | 0 | 0
  - **8** | 45 | 78 | 110 | 140 | 159 | 176 | 186 | 194 | 0
  - **10** | 85 | 110 | 136 | 158 | 177 | 194 | 210 | 224 | 0
  - **12** | 76 | 75 | 91 | 106 | 118 | 129 | 0 | 0 | 0
  - **14** | 82 | 102 | 124 | 143 | 161 | 176 | 190 | 203 | 0
  - **16** | 107 | 131 | 154 | 174 | 193 | 210 | 225 | 240 | 0
  - **18** | 110 | 143 | 175 | 206 | 230 | 258 | 285 | 303 | 0
  - **20** | 90 | 111 | 130 | 147 | 163 | 177 | 190 | 202 | 0

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