24

Footings

GENERAL CONSIDERATIONS

Provisions of Chapter 15 apply primarily for design of footings supporting a single column (isolated footings) and do not provide specific design provisions for footings supporting more than one column (combined footings). The code states that combined footings shall be proportioned to resist the factored loads and induced reactions in accordance with the appropriate design requirements of the code. Detailed discussion of combined footing design is beyond the scope of Part 24. However, as a general design approach, combined footings may be designed as beams in the longitudinal direction and as an isolated footing in the transverse direction over a defined width on each side of the supported columns. Code references 15.1 and 15.2 are suggested for detailed design recommendations for combined footings.

15.2 LOADS AND REACTIONS

Footings must be designed to safely resist the effects of the applied factored axial loads, shears and moments. The size (base area) of a footing or the arrangement and number of piles is determined based on the permissible soil pressure or permissible pile capacity, respectively. The permissible soil or pile capacity is determined by principles of soil mechanics in accordance with general building codes. The following procedure is specified for footing design:

1. The footing size (plan dimensions) or the number and arrangement of piles is to be determined on the basis of unfactored (service) loads (dead, live, wind, earthquake, etc.) and the permissible soil or pile capacity (15.2.2).

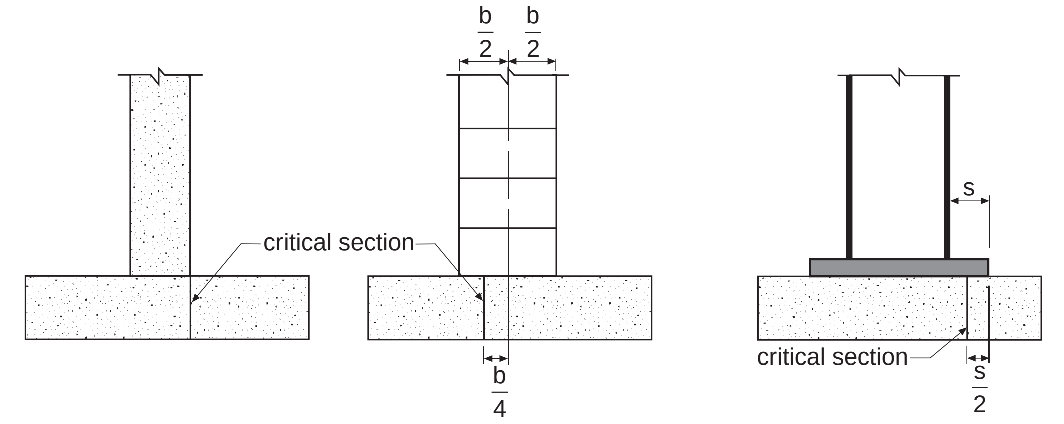
2. After having established the plan dimensions, the depth of the footing and the required amount of reinforcement are determined based on the strength design method (15.2.1). The service pressures and the resulting shear and moments are multiplied by the appropriate load factors specified in 9.2 and are used to proportion the footing.

For purposes of analysis, an isolated footing may be assumed to be rigid, resulting in a uniform soil pressure for concentric loading, and a triangular or trapezoidal soil pressure distribution for eccentric loading (combined axial and bending effect). Only the computed bending moment that exists at the base of the column or pedestal is to be transferred to the footing. The minimum moment requirement for slenderness considerations in 10.12.3.2 need not be transferred to the footing (R15.2).

15.4 MOMENT IN FOOTINGS

At any section of a footing, the external moment due to the base pressure shall be determined by passing a vertical plane through the footing and computing the moment of the forces acting over the entire area of the footing on one side of the vertical plane. The maximum factored moment in an isolated footing is determined by passing a vertical plane through the footing at the critical locations shown in Fig. 24-1. This moment is subsequently used to determine the required area of flexural reinforcement in that direction.

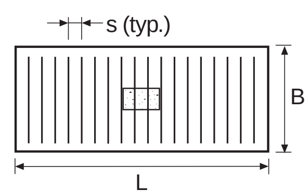
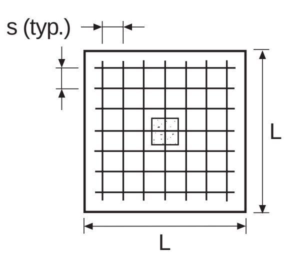
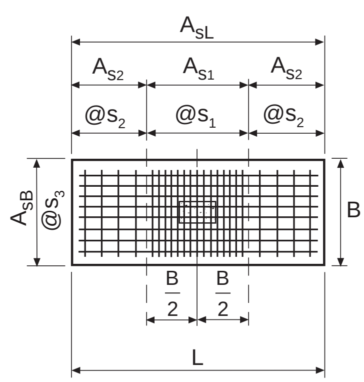
In one-way footings and two-way square footings, flexural reinforcement shall be distributed uniformly across the entire width of the footing (15.4.3). For two-way rectangular footings, the reinforcement must be distributed as shown in Table 24-1.



(a) Concrete Column, Pedestal or Wall (b) Masonry Wall (c) Column with Steel Base Plate

Figure 24-1 Critical Location for Maximum Factored Moment in an Isolated Footing

Table 24-1 Distribution of Flexural Reinforcement

Footing Type	Square Footing	Rectangular Footing
One-way	———	
Two-way		 $A_{s1} = \left(\frac{2}{\beta + 1}\right) A_{sL}$ $A_{s2} = \frac{A_{sL} - A_{s1}}{2}$ $\beta = \frac{L}{B}$

15.5 SHEAR IN FOOTINGS

Shear strength of a footing in the vicinity of the supported member (column or wall) must be determined for the more severe of the two conditions stated in 11.12. Both wide-beam action (11.12.1.1) and two-way action (11.12.1.2) must be checked to determine the required footing depth. Beam action assumes that the footing acts as a wide beam with a critical section across its entire width. If this condition is the more severe, design for shear proceeds in accordance with 11.1 through 11.5. Even though wide-beam action rarely controls the shear strength of footings, the designer must ensure that shear strength for beam action is not exceeded. Two-way action for the footing checks "punching" shear strength. The critical section for punching shear is a perimeter b_o around the supported member with the shear strength computed in accordance with 11.12.2.1. Tributary areas and corresponding critical sections for wide-beam action and two-way action for an isolated footing are illustrated in Fig. 24-2. Note that it is permissible to use a critical section with four straight sides for square or rectangular columns (11.12.1.3).

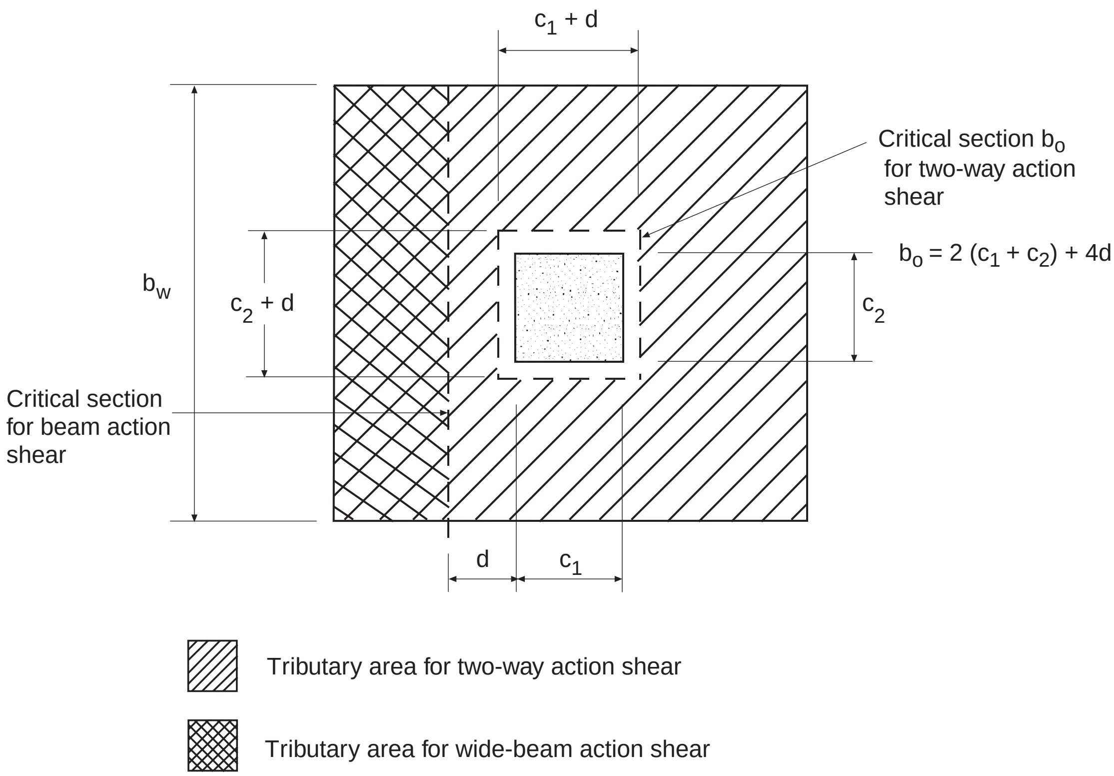


Figure 24-2 Tributary Areas and Critical Sections for Shear

The shear strength V_c of a footing for two-way action depends on the support size β_c, which is the ratio of the long-to-short side of the column or support area. For two-way action, V_c has an upper limit of $4\sqrt{f'_c}\, b_o d$ (Eq. (11-37)) for $\beta_c \le 2$ and reduces to $2\sqrt{f'_c}\, b_o d$ (Eq. (11-35)) as $1/\beta_c$ approaches zero for wide-beam action (see Fig. 24-3).

The shear strength also decreases as the ratio of the critical perimeter b_o to the effective depth d increases. Equation (11-36) has been introduced to account for this decrease. This equation rarely governs for an isolated footing supporting a single column, since the ratio b_o/d is considerably less than the limiting value to reduce the shear strength below the upper limit of $4\sqrt{f'_c}\, b_o d$.

If the factored shear force V_u at the critical section exceeds the shear strength ϕV_c given by the lesser of Eqs. (11-35), (11-36), or (11-37), shear reinforcement must be provided. For shear reinforcement consisting of bars or wires, the shear strength may be increased to a maximum value of $6\sqrt{f'_c}\, b_o d$ (11.12.3.2). However, shear reinforcement must be designed to carry the shear in excess of $2\sqrt{f'_c}\, b_o d$ (11.12.3.1). This limit is one-half that permitted by Eq. (11-36) with a b_o/d ratio of 2 or less.

For footing design (without shear reinforcement), the shear strength equations may be summarized as follows:

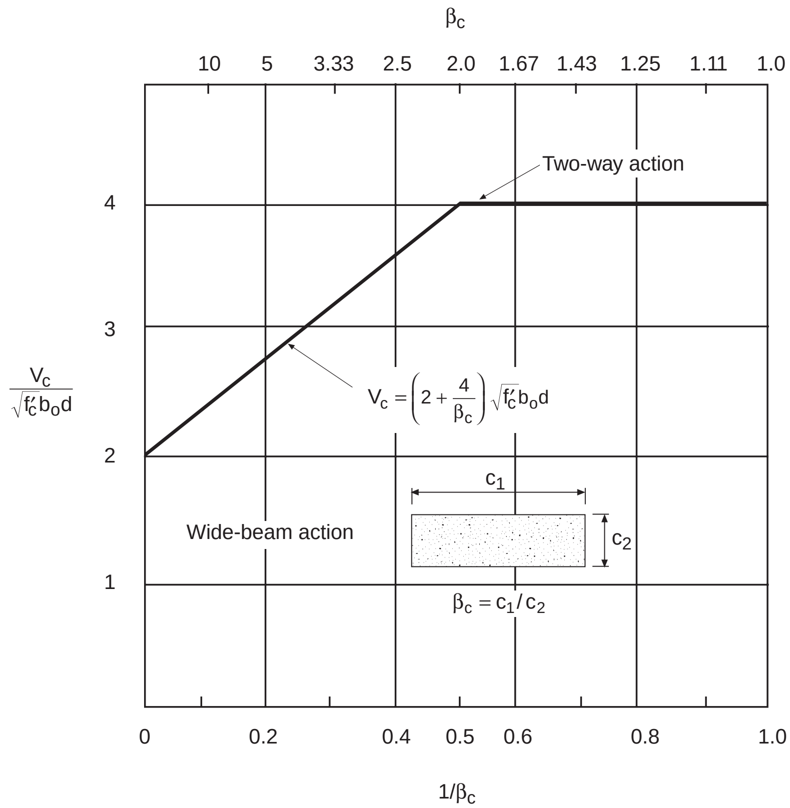


Figure 24-3 Shear Strength of Concrete in Footings

- Wide beam action

$$V_u \leq \phi V_n \qquad \textit{Eq. (11-1)}$$

$$\leq \phi\left(2\sqrt{f'_c}\, b_w d\right) \qquad \textit{Eq. (11-3)}$$

where b_w and V_u are computed for the critical section defined in 11.12.1.1 (see Fig. 24-2).

- Two-way action

$$V_u \leq \text{minimum of} \begin{cases} \left(2 + \dfrac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d & \textit{Eq. (11-35)} \\ \left(\dfrac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d & \textit{Eq. (11-36)} \\ 4\sqrt{f'_c}\, b_o d & \textit{Eq. (11-37)} \end{cases}$$

where

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

α_s = 40 for interior columns
= 30 for edge columns
= 20 for corner columns

b_o = perimeter of critical section shown in Fig. 24-2

15.8 TRANSFER OF FORCE AT BASE OF COLUMN, WALL, OR REINFORCED PEDESTAL

With the publication of ACI 318-83, 15.8 addressing transfer of force between a footing and supported member (column, wall, or pedestal) was revised to address both cast-in-place and precast construction. Section 15.8.1 gives general requirements applicable to both cast-in-place and precast construction. Sections 15.8.2 and 15.8.3 give additional rules for cast-in-place and precast construction, respectively. For force transfer between a footing and a precast column or wall, anchor bolts or mechanical connectors are specifically permitted by 15.8.3. (Prior to the '83 code, connection between a precast member and footing required either longitudinal bars or dowels crossing the interface, contrary to common practice.) Also note that walls are specifically addressed in 15.8 for force transfer to footings.

Section 15.8.3 for the connection between precast columns and walls to supporting members was modified in the 1995 edition of the code. This section refers to 16.5.1.3 for minimum connection strength. Additionally, for precast columns with larger cross-sectional areas than required for loading, it is permitted to use a reduced effective area based on the cross-section required, but not less than one-half the total area when determining the nominal strength in tension.

The minimum tensile strength of a connection between a precast wall panel and its supporting member, previously specified as $50A_g$, was also changed in the 1995 edition. The connection is required to have a minimum of two ties per panel with a minimum nominal tensile capacity of 10 kips per tie.

All forces applied at the base of a column or wall (supported member) must be transferred to the footing (supporting member) by bearing on concrete and/or by reinforcement. Tensile forces must be resisted entirely by reinforcement. Bearing on concrete for both supported and supporting member must not exceed the concrete bearing strength permitted by 10.15 (see discussion on 10.15 in Part 6).

For a supported column, the bearing capacity ϕP_{nb} is

$$\phi P_{nb} = \phi(0.85 f'_c A_1) \qquad \textit{10.17.1}$$

where

f'_c = compressive strength of the column concrete

A_1 = loaded area (column area)

ϕ = 0.70 *9.3.2.4*

For a supporting footing,

$$\phi P_{nb} = \phi(0.85 f'_c A_1)\sqrt{\frac{A_2}{A_1}} \le 2\phi(0.85 f'_c A_1)$$

where

f'_c = compressive strength of the footing concrete

A_2 = area of the lower base of the largest frustrum of a pyramid, cone, or tapered wedge contained wholly within the footing and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal (see Fig. R10.17).

Example 24.4 illustrates the design for force transfer at the base of a column.

When bearing strength is exceeded, reinforcement must be provided to transfer the excess load. A minimum area of reinforcement must be provided across the interface of column or wall and footing, even where concrete bearing strength is not exceeded. With the force transfer provisions addressing both cast-in-place and precast construction, including force transfer between a wall and footing, the minimum reinforcement requirements are based on the type of supported member, as shown in Table 24-2.

Table 24-2 Minimum Reinforcement for Force Transfer Between Footing and Supported Member

	Cast-in-Place	Precast
Columns	$0.005A_g$	$\frac{200A_g}{f_y}$
Walls	see 14.3.2	see 16.5.1.3(b) and (c)

For cast-in-place construction, reinforcement may consist of extended reinforcing bars or dowels. For precast construction, reinforcement may consist of anchor bolts or mechanical connectors. Unfortunately, the code does not give any specific data for design of anchor bolts or mechanical connectors (see Example 24.8). Reference 24.1 devotes an entire chapter on connection design for precast construction.

The shear-friction design method of 11.7.4 should be used for horizontal force transfer between columns and footings (15.8.1.4; see Example 24.6). Consideration of some of the lateral force being transferred by shear through a formed shear key is questionable. Considerable slip is required to develop a shear key. Shear keys, if provided, should be considered as an added mechanical factor of safety only, with no design shear force assigned to the shear key.

PLAIN CONCRETE PEDESTALS AND FOOTINGS

Plain concrete pedestals and footings are designed in accordance with Chapter 22. See Part 32 for an in-depth discussion and examples.

REFERENCE

24.1 *PCI Design Handbook—Precast and Prestressed Concrete*, 4th Edition, Precast/Prestressed Concrete Institute, Chicago, IL, 1992.

Example 24.1—Design for Base Area of Footing

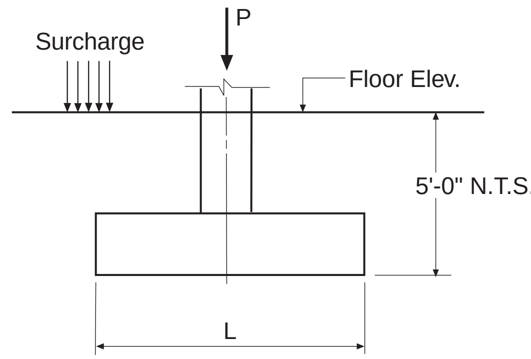
Determine the base area A_f required for a square spread footing with the following design conditions:

Service dead load = 350 kips
Service live load = 275 kips
Service surcharge = 100 psf

Assume average weight of soil and concrete above footing base = 130 pcf

Permissible soil pressure = 4.5 ksf

Column dimensions = 30 × 12 in.



Calculations and Discussion	Code Reference

1. Determination of base area:

   The base area of the footing is determined using service (unfactored) loads with the net permissible soil pressure.

   Total weight of surcharge = (0.130 × 5) + 0.100 = 0.750 ksf

   Net permissible soil pressure = 4.5 - 0.75 = 3.75 ksf

   Required base area of footing: *15.2.2*

   $$A_f = \frac{350 + 275}{3.75} = 167 \text{ ft}^2$$

   Use a 13 × 13 ft square footing (A_f = 169 ft^2)

2. Factored loads and soil reaction:

   To proportion the footing for strength (depth and required reinforcement) factored *15.2.1*

   P_u = 1.4 (350) + 1.7 (275) = 957.5 kips *Eq. (9-1)*

   $$q_s = \frac{P_u}{A_f} = \frac{957.5}{169} = 5.70 \text{ ksf}$$

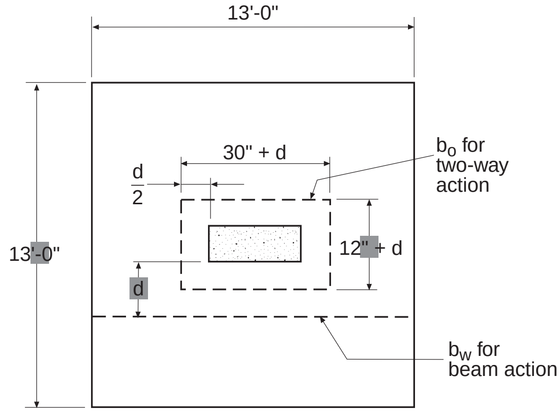
Example 24.2—Design for Depth of Footing

For the design conditions of Example 24.1, determine the overall thickness of footing required.

$f'_c = 3000$ psi

$P_u = 957.5$ kips

$q_s = 5.70$ ksf



Calculations and Discussion	Code Reference
Determine depth based on shear strength without shear reinforcement. Depth required for shear usually controls the footing thickness. Both wide-beam action and two-way action for strength computation need to be investigated to determine the controlling shear criteria for depth.	11.12

Assume overall footing thickness = 33 in. and average effective thickness d = 28 in. = 2.33 ft

1. Wide-beam action:

$V_u = q_s \times \text{tributary area}$

$b_w = 13 \text{ ft} = 156 \text{ in.}$

$\text{Tributary area} = 13\,(6.0 - 2.33) = 47.7 \text{ ft}^2$

$V_u = 5.7 \times 47.7 = 272 \text{ kips}$

$$\phi V_n = \phi\left(2\sqrt{f'_c}\,b_w d\right) \qquad \text{Eq. (11-3)}$$

$$= 0.85\left(2\sqrt{3000} \times 156 \times 28\right)/1000$$

$$= 407 \text{ kips} > V_u \quad \text{O.K.}$$

2. Two-way action:

$V_u = q_s \times \text{tributary area}$

$$\text{Tributary area} = \left[(13 \times 13) - \frac{(30 + 28)\,(12 + 28)}{144}\right] = 152.9 \text{ ft}^2$$

$V_u = 5.70 \times 152.9 = 872 \text{ kips}$

$$\frac{V_c}{\sqrt{f'_c}\,b_o d} = \text{minimum of} \begin{cases} 2 + \dfrac{4}{\beta_c} & \textit{Eq. (11-35)} \\ \dfrac{\alpha_s d}{b_o} + 2 & \textit{Eq. (11-36)} \\ 4 & \textit{Eq. (11-37)} \end{cases}$$

$b_o = 2\,(30 + 28) + 2\,(12 + 28) = 196$ in.

$$\beta_c = \frac{30}{12} = 2.5$$

$$\frac{b_o}{d} = \frac{196}{28} = 7$$

$\alpha_s = 40$ for interior columns

$$\frac{V_c}{\sqrt{f'_c}\,b_o d} = \begin{cases} 2 + \dfrac{4}{2.5} = 3.6 \quad \text{(governs)} \\ \dfrac{40}{7} + 2 = 7.7 \\ 4 \end{cases}$$

$$\phi V_c = 0.85 \times 3.6\sqrt{3000} \times 196 \times 28/1000$$

$$= 920 \text{ kips} > 872 \text{ kips} \quad \text{O.K.}$$

Example 24.3—Design for Footing Reinforcement

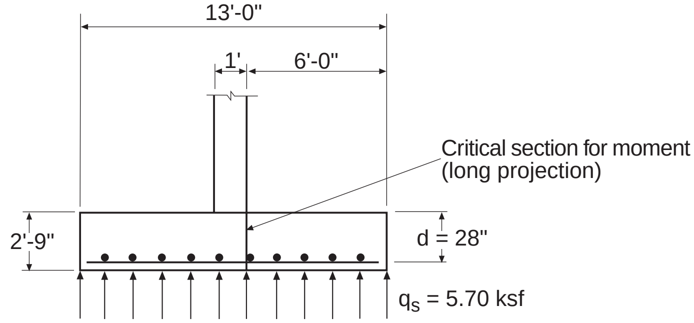
For the design conditions of Example 24.1, determine required footing reinforcement.

f'_c = 3000 psi

f_y = 60,000 psi

P_u = 957.5 kips

q_s = 5.70 ksf



Calculations and Discussion	Code Reference

1. Critical section for moment is at face of column — 15.4.2

$$M_u = 5.70 \times 13 \times 6^2/2 = 1334 \text{ ft-kips}$$

2. Compute A_s required (see Part 10)

$$\text{Required } R_n = \frac{M_u}{\phi bd^2} = \frac{1334 \times 12 \times 1000}{0.9 \times 156 \times 28^2} = 145 \text{ psi}$$

$$\rho = \frac{0.85f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}}\right)$$

$$= \frac{0.85 \times 3}{60}\left(1 - \sqrt{1 - \frac{2 \times 145}{0.85 \times 3000}}\right) = 0.0025$$

$$\rho \text{ (gross area)} = \frac{d}{h} \times 0.0025 = \frac{28}{33} \times 0.0025 = 0.0021$$

Check minimum A_s required for footings of uniform thickness; for Grade 60 reinforcement: — 10.5.4

ρ_{min} = 0.0018 < 0.0021 O.K. — 7.12.2

Required A_s = ρbd

A_s = 0.0025 × 156 × 28 = 10.92 in.^{2}

Use 14-No. 8 bars (A_s = 11.06 in.^{2}) each way

Note that a lesser amount of reinforcement is required in the perpendicular direction, but for ease of placement, the same uniformly distributed reinforcement will be used each way (see Table 24-1).

3. Check development of reinforcement. *15.6*

Critical section for development is the same as that for moment (at face of column). *15.6.3*

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)}$$ *Eq. (12-1)*

Clear cover (bottom and side) = 3.0 in.

$$\text{Center-to-center bar spacing} = \frac{156 - 2\ (3) - 2\ (0.5)}{13} = 11.5 \text{ in.}$$

$$c = \text{minimum of} \begin{cases} 3.0 + 0.5 = 3.5 \text{ in.} \quad \text{(governs)} \\ \dfrac{11.5}{2} = 5.75 \text{ in.} \end{cases}$$ *12.2.4*

K_{tr} = 0 (no transverse reinforcement)

$$\frac{c + K_{tr}}{d_b} = \frac{3.5 + 0}{1.0} = 3.5 > 2.5, \text{ use } 2.5$$ *12.2.3*

α = 1.0 (less than 12 in. of concrete below bars) *12.2.4*

β = 1.0 (uncoated reinforcement)

$\alpha\beta$ = 1.0 < 1.7

γ = 1.0 (larger than No. 7 bars)

λ = 1.0 (normal weight concrete)

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{60{,}000}{\sqrt{3000}}\frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.5} = 32.9$$

$\ell_d = 32.9 \times 1.0 = 32.9$ in. > 12.0 in. O.K. *12.2.1*

Since ℓ_d = 32.9 in. is less than the available embedment length in the short direction

$\left(\frac{156}{2} - \frac{30}{2} - 3 = 60 \text{ in.}\right)$, the No. 8 bars can be fully developed.

Example 24.4—Design for Transfer of Force at Base of Column

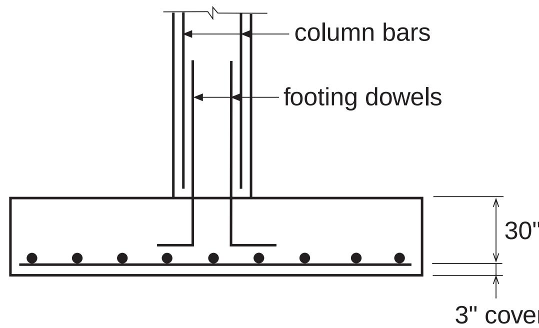
For the design conditions of Example 24.1, check force transfer at interface of column and footing.

f'_c (column) = 5000 psi

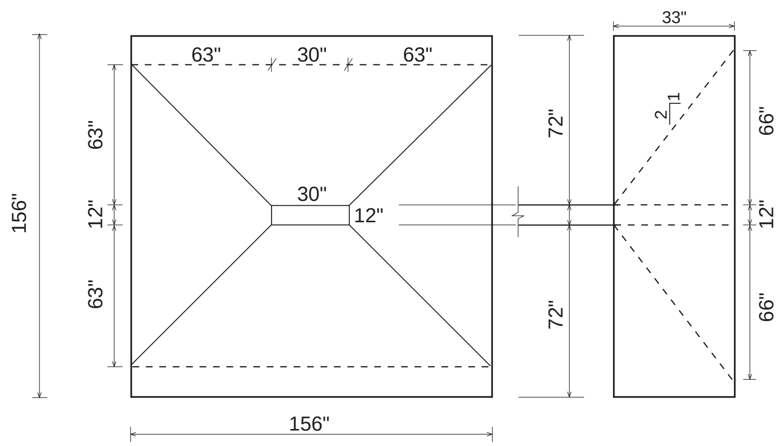
f'_c (footing) = 3000 psi

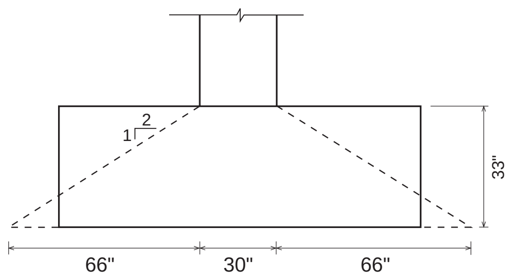
f_y = 60,000 psi

P_u = 957.5 kips



Calculations and Discussion	Code Reference
1. Bearing strength of column (f'_c = 5000 psi):	*15.8.1.1*
$\phi P_{nb} = \phi(0.85 f'_c A_1)$	*10.17.1*
$= 0.70\ (0.85 \times 5 \times 12 \times 30) = 1071$ kips > 957.5 kips O.K.	
2. Bearing strength of footing (f'_c = 3000 psi):	*15.8.1.1*
The bearing strength of the footing is increased by a factor $\sqrt{A_2/A_1} \leq 2$ due to the to the large footing area permitting a greater distribution of the column load.	*10.17.1*





A_1 is the column (loaded) area and A_2 is the plan area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal. For the 30 × 12 in. column supported on the 13 × 13 ft square footing, $A_2 = (66 + 12 + 66) \times (63 + 30 + 63)$.

$$\sqrt{\frac{A_2}{A1}} = \sqrt{\frac{144 \times 156}{30 \times 12}} = 7.9 > 2, \text{ use } 2$$

Note that bearing on the column concrete will always govern until the strength of the column concrete exceeds twice that of the footing concrete.

$$\phi P_{nb} = 2\left[\phi(0.85 f'_c A_1)\right]$$

$$= 2\ [0.70\ (0.85 \times 3 \times 12 \times 30)] = 1285 \text{ kips} > 957.5 \text{ kips} \quad \text{O.K.}$$

3. Required dowel bars between column and footing:

Even though bearing strength on the column and footing concrete is adequate to transfer the factored loads, a minimum area of reinforcement is required across the interface. *15.8.2.1*

A_s (min) = 0.005 (30 × 12) = 1.80 in.^{2}

Provide 4-No. 7 bars as dowels (A_s = 2.40 in.^{2})

4. Development of dowel reinforcement in compression: *12.3.2*

In column:

$$\ell_d = \frac{0.02 d_b f_y}{\sqrt{f'_c}} \geq 0.0003 d_b f_y$$

For No. 7 bars:

$$\ell_d = \frac{0.02 \times 0.875 \times 60{,}000}{\sqrt{5000}} = 14.9 \text{ in.}$$

$$\ell_{d(min)} = 0.0003 \times 0.875 \times 60{,}000 = 15.8 \text{ in.} \quad \text{(governs)}$$

In footing:

$$\ell_d = \frac{0.02 \times 0.875 \times 60{,}000}{\sqrt{3000}} = 19.2 \text{ in.} \quad \text{(governs)}$$

$$\ell_{d(min)} = 0.0003 \times 0.875 \times 60{,}000 = 15.8 \text{ in.}$$

Available length for development in footing

= footing thickness - cover - 2 (footing bar diameter) - dowel bar diameter

= 33 - 3 - 2 (1.0) - 0.875 = 27.1 in. > 19.2 in.

Therefore, the dowels can be fully developed in the footing.

Example 24.5—Design for Transfer of Force by Reinforcement

For the design conditions given below, provide for transfer of force between the column and footing.

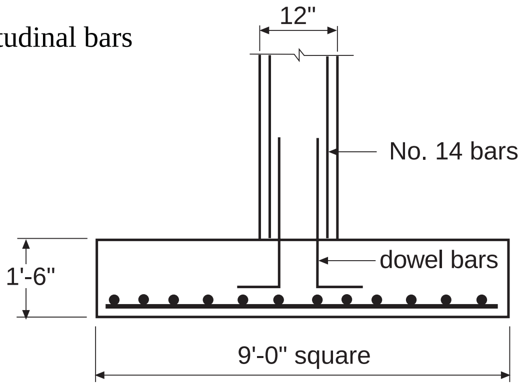
12 × 12 in. tied reinforced column with 4-No. 14 longitudinal bars

$f'_c = 4000$ psi (column and footing)

$f_y = 60{,}000$ psi

$P_D = 200$ kips

$P_L = 100$ kips



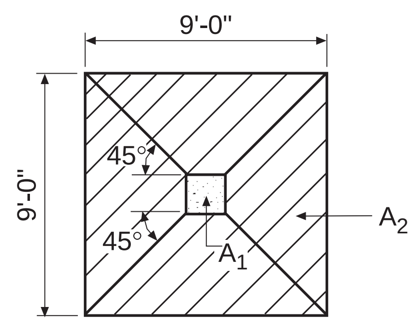
Calculations and Discussion	Code Reference
1. Factored load $P_u = (1.4 \times 200) + (1.7 \times 100) = 450$ kips	*Eq. (9-1)*
2. Bearing strength on column concrete:	*15.8.1.1*

$$\phi P_{nb} = \phi(0.85 f'_c A_1) = 0.70\,(0.85 \times 4 \times 12 \times 12)$$ *10.17.1*

$$= 342.7 \text{ kips} < 450 \text{ kips} \quad \text{N.G.}$$

The column load cannot be transferred by bearing on concrete alone. The excess load (450 - 342.7 = 107.3 kips) must be transferred by reinforcement. *15.8.1.2*

3. Bearing strength on footing concrete: *15.8.1.1*



$$\phi P_{nb} = \sqrt{\frac{A_2}{A_1}}\,[\phi(0.85 f'_c A_1)]$$

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{9 \times 9}{1 \times 1}} = 9 > 2, \text{ use } 2$$

$$\phi P_{nb} = 2\,(342.7) = 685.4 \text{ kips} > 450 \text{ kips} \quad \text{O.K.}$$

4. Required area of dowel bars: *15.8.1.2*

$$A_s\,(\text{required}) = \frac{(P_u - \phi P_{nb})}{\phi f_y}$$

$$= \frac{107.3}{0.70 \times 60} = 2.55 \text{ in.}^2$$

$A_s\,(\text{min}) = 0.005\,(12 \times 12) = 0.72$ in.^{2} *15.8.2.1*

Try 4-No. 8 bars ($A_s = 3.16$ in.^{2})

5. Development of dowel reinforcement

a. For development into the column, the No. 14 column bars may be lap spliced with the No. 8 footing dowels. The dowels must extend into the column a distance not less than the development length of the No. 14 column bars or the lap splice length of the No. 8 footing dowels, whichever is greater. *15.8.2.3*

For No. 14 bars:

$$\ell_d = \frac{0.02 d_b f_y}{\sqrt{f_c'}} = \frac{0.02 \times 1.693 \times 60{,}000}{\sqrt{4000}} = 32.1 \text{ in.} \quad \text{(governs)}$$

$$\ell_{d(min)} = 0.0003 d_b f_y = 0.0003 \times 1.693 \times 60{,}000 = 30.5 \text{ in.}$$

For No. 8 bars:

$$\text{lap length} = 0.0005 f_y d_b$$ *12.16.1*

$$= 0.0005 \times 60{,}000 \times 1.0 = 30.0 \text{ in.}$$

Development length of No. 14 bars governs.

The No. 8 dowel bars must extend not less than 33 in. into the column.

b. For development into the footing, the No. 8 dowels must extend a full development length. *15.8.2.3*

$$\ell_d = \frac{0.02 d_b f_y}{\sqrt{f_c'}} = \frac{0.02 \times 1.0 \times 60{,}000}{\sqrt{4000}} = 19.0 \text{ in.} \quad \text{(governs)}$$

$$\ell_{d(min)} = 0.0003 d_b f_y = 0.0003 \times 1.0 \times 60{,}000 = 18.0 \text{ in.}$$

This length may be reduced to account for excess reinforcement. *12.3.3.1*

$$\frac{A_s \text{ (required)}}{A_s \text{ (provided)}} = \frac{2.55}{3.16} = 0.81$$

Required $\ell_d = 19.0 \times 0.81 = 15.4$ in.

If the footing dowels are bent for placement on top of the footing reinforcement (as shown in the figure), the bent portion cannot be considered effective for developing the bars in compression. Available length for development above footing reinforcement $\approx$ 18 - 6 = 12 in. < 15.4 in. required. Either the footing depth must be increased or a larger number of smaller-sized dowels used. *12.5.5*

Increase footing depth to 1 ft-9 in. and provide 4-No. 8 dowels, extended 33 in. into the column and bent 90-deg. for placement on top of the footing reinforcement. Total length of No. 8 dowels = 32 + 16 = 48 in.

Example 24.6—Design for Transfer of Horizontal Force at Base of Column

For the column and footing of Example 24.5, design for transfer of a horizontal factored force of 95 kips acting at the base of the column.

Design data:

Footing: size = 9 × 9 ft
thickness = 1ft-9 in.

Column: size = 12 × 12 in. (tied)
4-No. 14 longitudinal reinforcement

f'_c = 4000 psi (footing and column)

f_y = 60,000 psi

Calculations and Discussion	Code Reference
1. The shear-friction design method of 11.7 is applicable.	*15.8.1.4*
Check maximum shear transfer permitted:	*11.7.5*
$V_u \le \phi(0.2f'_cA_c)$ but not greater than $\phi(800A_c)$	
$\phi V_n = 0.85\,(0.2 \times 4 \times 12 \times 12) = 97.9$ kips	
$\phi(800A_c) = 0.85 \times 800 \times 12 \times 12/1000 = 97.9$ kips	
$V_u < \phi(0.2f'_cA_c)$ and $\phi(800A_c)$ O.K.	
The shear transfer of 95 kips is permitted at the base of 12 × 12 in. column.	
Strength requirement for shear:	
$V_u \le \phi V_n$	*Eq. (11-1)*
$V_n = V_u/\phi = A_{vf}f_y\mu$	*Eq. (11-25)*
Use μ = 0.6 (concrete not intentionally roughened)	*11.7.4.3*
and ϕ = 0.85 (shear)	
Required $A_{vf} = \dfrac{V_u}{\phi f_y\mu} = \dfrac{95}{0.85 \times 60 \times 0.6} = 3.10 \text{ in.}^2$	*Eq. (11-25)*
Note that this steel area (3.10 in.^{2}) is less than that required for transfer of vertical force (3.6 in.^{2}) in Example 24.5. Therefore, the 4-No. 8 dowels would be adequate for transfer of horizontal shear also.	

If the 4-No. 8 dowels were not adequate for transfer of horizontal shear, the footing concrete in contact with the column concrete could be roughened to an amplitude of approximately 1/4 in. to take advantage of the higher coefficient of friction of 1.0:

$$\text{Required } A_{vf} = \frac{95}{0.85 \times 60 \times 1.0} = 1.86 \text{ in.}^2$$

2. Tensile development of No. 8 dowels

   a. Within the column

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)}$$ *Eq. (12-1)*

Clear cover to No. 8 bar ≈ 3.25 in.

Center-to-center bar spacing of No. 8 bars ≈ 4.5 in.

$$c = \text{minimum of} \begin{cases} 3.25 + 0.5 = 3.75 \text{ in.} \\ \dfrac{4.5}{2} = 2.25 \text{ in.} \quad \text{(governs)} \end{cases}$$ *12.2.4*

Assume $K_{tr} = 0$ (no transverse reinforcement)

$$\frac{c + K_{tr}}{d_b} = \frac{2.25 + 0}{1.0} = 2.25 < 2.5\text{, use } 2.25$$ *12.2.3*

$\alpha = 1.0$

$\beta = 1.0$

$\alpha\beta = 1.0 < 1.7$

$\gamma = 1.0$

$\lambda = 1.0$

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{60{,}000}{\sqrt{4000}}\frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.25} = 31.6$$

$\ell_d = 31.6 \times 1.0 = 31.6$ in.

Provide at least 32 in. of embedment into the column.

   b. Within the footing

Use standard hooks at the ends of the No. 8 bars

$\ell_{hb} = 1200 d_b / \sqrt{f'_c}$ 12.5.2

$= 1200 \times 1/\sqrt{4000} = 19$ in.

Modifications: 12.5.3

cover normal to plane of 90° hook > 2.5 in.

cover on bar extension beyond hook ≥ 2 in.

$\ell_{dh} = 0.7 \times 19 = 13.3$ in. 12.5.3.2

Min. $\ell_{dh} = 8 \times d_b = 8$ in. < 13.3 in. 12.5.1

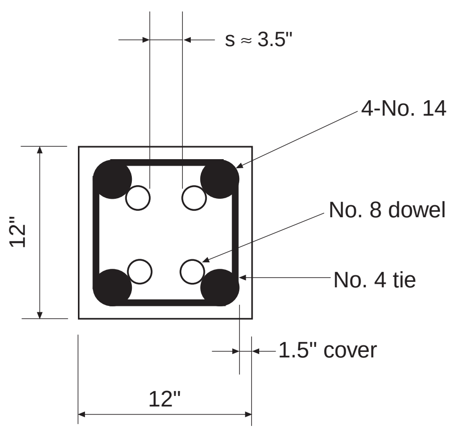
Available development in 1 ft-9 in. depth of footing

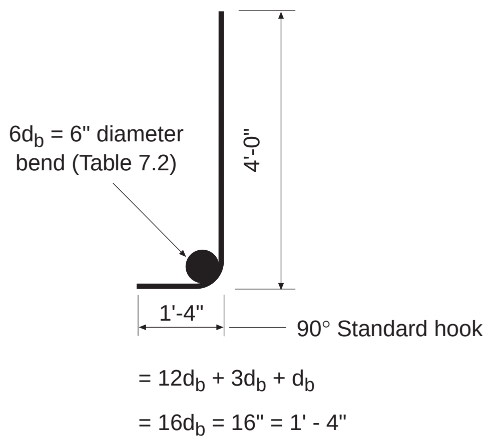
$= 21 - 6 = 15$ in. > 13.3 in. O.K.

Use 15 in. hook embedment into footing to secure hook on top of footing reinforcement for placement.

Total length of No. 8 dowel = 32 + 15 = 47 in. Use 4 ft-0 in. long dowels.

Note: The top of the footing at the interface between column and footing must be clean and free of laitance before placement of the column concrete. 11.7.9





No. 8 dowel detail

Example 24.7—Design for Depth of Footing on Piles

For the footing supported on the piles shown, determine the required thickness of the footing (pile cap).

Footing size $= 8.5 \times 8.5$ ft

Column size $= 16 \times 16$ in.

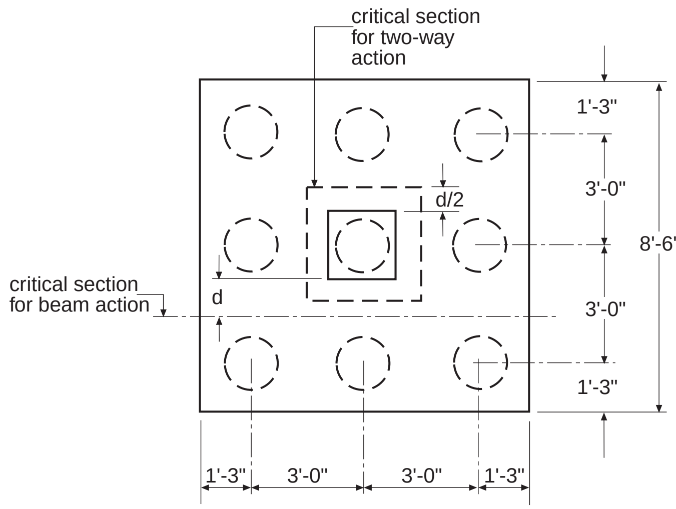
Pile diameter $= 12$ in.

$f'_c = 4000$ psi

Load per pile:

$P_D = 20$ kips

$P_L = 10$ kips



Calculations and Discussion	Code Reference

1. Depth required for shear usually controls footing thickness. Both wide-beam action and two-way action for the footing must be investigated. *11.12*

   Assume an overall footing thickness of 1 ft-9 in. with an average d $\approx$ 14 in. *15.7*

2. Factored pile loading:

   $P_u = 1.4\,(20) + 1.7\,(10) = 45$ kips *Eq. (9-1)*

3. Strength requirements for shear

   $V_u \leq \phi V_n$ *Eq. (11-1)*

   a. Wide-beam action for footing: *11.12.1.1*

   3 piles fall within tributary area

   V_u (neglecting footing wt.) $= 3 \times 45 = 135$ kips

   $\phi V_n = \phi\left(2\sqrt{f'_c}\,b_w d\right)$ *Eq. (11-3)*

   $b_w = 8$ ft-6 in. $= 102$ in.

   $\phi V_n = 0.85\left(2\sqrt{4000} \times 102 \times 14\right)/1000 = 153.5 \text{ kips} > V_u$ O.K.

   b. Two-way action: *11.12.1.2*

   8 piles fall within the tributary area

$$V_u = 8 \times 45 = 360 \text{ kips}$$

$$\frac{V_c}{\sqrt{f'_c}\,b_o d} = \text{minimum of} \begin{cases} 2 + \dfrac{4}{\beta_c} & \textit{Eq. (11-35)} \\ \dfrac{\alpha_s d}{b_o} + 2 & \textit{Eq. (11-36)} \\ 4 & \textit{Eq. (11-37)} \end{cases}$$

$$\beta_c = \frac{16}{16} = 1.0$$

$$b_o = 4\,(16 + 14) = 120 \text{ in.}$$

$$\alpha_s = 40 \text{ for interior columns}$$

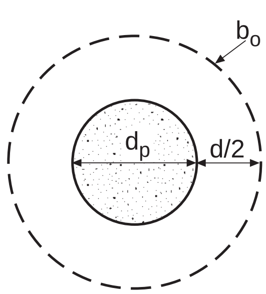
$$\frac{b_o}{d} = \frac{120}{14} = 8.6$$

$$\frac{V_c}{\sqrt{f'_c}\,b_o d} = \begin{cases} 2 + \dfrac{4}{1} = 6 \\ \dfrac{40}{8.6} + 2 = 6.7 \\ 4 \quad \text{(governs)} \end{cases}$$

$$\phi V_c = 0.85 \times 4\sqrt{4000} \times 120 \times 14/1000$$

$$= 361 \text{ kips} > 360 \text{ kips} \quad \text{O.K.}$$

4. Check "punching" shear strength at corner piles. With piles spaced at 3 ft-0 in. on center, critical perimeters do not overlap.



$$V_u = 45 \text{ kips per pile}$$

$$\frac{V_c}{\sqrt{f'_c}\,b_o d} = \text{minimum of} \begin{cases} 2 + \dfrac{4}{\beta_c} & \textit{Eq. (11-35)} \\ \dfrac{\alpha_s d}{b_o} + 2 & \textit{Eq. (11-36)} \\ 4 & \textit{Eq. (11-37)} \end{cases}$$

β_c = 1.0 (square reaction area of equal area)

$b_o = \pi(12 + 14) = 81.7$ in.

$\alpha_s = 20$ 11.12.2.1

$$\frac{b_o}{d} = \frac{81.7}{14} = 5.8$$

$$\frac{V_c}{\sqrt{f'_c}\,b_o d} = \begin{cases} 2 + \dfrac{4}{1} = 6 \\ \dfrac{20}{5.8} + 2 = 5.4 \\ 4 \quad \text{(governs)} \end{cases}$$

$\phi V_c = 0.85 \times 4\sqrt{4000} \times 81.7 \times 14/1000 = 246 \text{ kips} > V_u$ O.K.

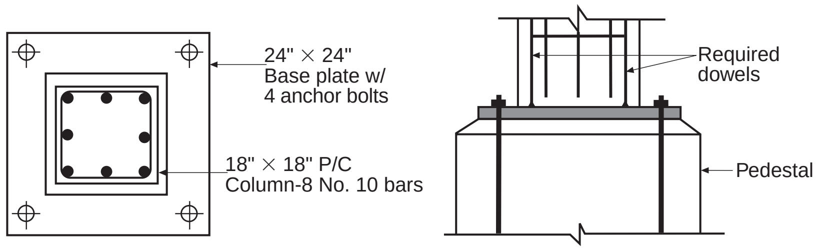
Example 24.8—Design for Transfer of Force at Base of Precast Column

For the 18 × 18 in. precast column and base plate detail shown, design for force transfer between column and pedestal for a factored load P_u = 1050 kips.

f'_c = 5000 psi (column)

f'_c = 3000 psi (pedestal)

f_y = 60,000 psi



The base plate is to be secured to the column by dowels. (Column steel not welded to base plate.)

Calculations and Discussion	Code Reference

1. Bearing strength requirement 10.17.1

$\phi P_{nb} \geq P_u$

a. Bearing strength on column concrete (between precast column and base plate), f'_c = 5000 psi

$\phi P_{nb} = \phi(0.85 f'_c A_1)$

$= 0.70\,(0.85 \times 5 \times 18 \times 18) = 964$ kips < 1050 kips N.G.

b. Bearing strength on pedestal concrete (between base plate and pedestal), f'_c = 3000 psi

$\phi P_{nb} = 0.70\,(0.85 \times 3 \times 24 \times 24) = 1028$ kips < 1050 kips N.G.

Factored load cannot be transferred by bearing on concrete alone for either column or pedestal. The excess load between column and base plate (1050 - 964 = 86 kips), and between base plate and pedestal (1050 - 1028 = 22 kips) must be transferred by reinforcement.

2. In the manufacture of precast columns it is common practice to cast the base plate with the column. The base plate may be secured to the column either by deformed bar anchors (dowels) or column bars welded to the base plate (see page 6-22 of Ref. 24.1).

For the base plate connected to column by dowels (column steel not welded to base plate)

Required area of dowel bars

$$A_s\ (\text{required}) = \frac{(P_u - \phi P_{nb})}{\phi f_y} = \frac{(1050 - 964)}{0.7 \times 60} = 2.04\ \text{in.}^2$$ 9.3.2.4

Also, connection between precast column and base plate must have a tensile strength not less than $200A_g$ in pounds, where A_g is area of precast column — 15.8.3.1

$$A_s\ (\min) = \frac{200A_g}{f_y} = \frac{200 \times 18 \times 18}{60{,}000} = 1.08\ \text{in.}^2 < 2.04\ \text{in.}^2$$

$$\text{Number of No. 5 deformed bars required} = \frac{2.04}{0.31} = 6.6$$

Use 8-No. 5 dowels ($A_s = 2.48$ in.^{2}), 2 at each corner (see following figure)

Basic development length of dowels:

$$\ell_{db} = \frac{0.02 d_b f_y}{\sqrt{f'_c}} = \frac{0.02 \times 0.625 \times 60{,}000}{\sqrt{5000}}$$ — 12.3.2

$$= 10.6\ \text{in.}$$

and $\ell_{db} = 0.003 d_b f_y = 0.0003 \times 0.625 \times 60{,}000 = 11.25$ in. (governs)

Modification for ℓ_{db} due to excess steel area provided — 12.3.3

$$\ell_d = \frac{11.25 \times 2.04}{2.48} = 9.25\ \text{in.}$$

Use 8-No. 5 deformed bars 10 in. long. Anchors are automatically welded (similar to headed studs) to base plate. The base plate and bar anchor assembly is then cast with the column.

3. Excess load between base plate and pedestal = 1050 - 1028 = 22 kips must also be transferred by reinforcement, with an area not less than $200A_g/f_y$. — 16.5.1.3(a)

Check 4 anchor bolts, ASTM A36 steel.

$$A_s\ (\text{required}) = \frac{(1050 - 1028)}{0.7 \times 36} = 0.873\ \text{in.}^2$$

but not less than $A_s\ (\min) = \dfrac{200 \times 18 \times 18}{36{,}000} = 1.80$ in.^{2} (governs)

Note: The code minimum of $200A_g/f_y$ applies also to the connection between base plate and pedestal.

$$\text{Required number of 3/4 in. anchor bolts} = \frac{1.8}{\frac{\pi}{4}(0.75)^2} = 4.1$$

Use four 3/4 in. anchor bolts.

The anchor bolts must be embedded into the pedestal to develop their design strength in bond. Determine embedment length of smooth anchor bolt as 2 times the embedment length for a deformed bar.

$$\ell_d = 2\left(0.02 \times 0.75 \times 36{,}000/\sqrt{3000}\right) = 19.7 \text{ in.} \quad \text{(governs)}$$ 12.3.2

but not less than 2 (0.0003 × 0.75 × 36,000) = 16.2 in.

Use four 3/4 in.× 1 ft-9 in. anchor bolts. Enclose anchor bolts with 4-No. 3 ties at 3 in. centers (see connection detail below).

Note: The reader should refer to Ref. 24.1 for an in-depth treatise on design and construction details for precast column connections. Design for the base plate thickness is also addressed in Ref. 24.1.