DESIGN OF PRESTRESSED CONCRETE PILE FOUNDATIONS

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This sixth paper in the series of articles on "Design Considerations for a Precast Prestressed Apartment Building" covers the design of the prestressed concrete pile foundations.

The first part of the paper describes the typical design considerations that enter into proper pile selection. Such factors as pile capacities, pile lengths, driving conditions, and pile prices are discussed.

The second part of the paper presents the detailed design calculations for a typical prestressed pile foundation.

Detailed computations are given for an interior footing, a perimeter wall footing, and a footing at transfer beam.

Step-by-step calculations are included for finding the number of piles, pile layout, beam shear, punching shear, moment steel, development length, and steel distribution. A middle section is devoted to lateral load analysis.

Finally, the last section presents the pile dowel and pile prestress calculations including a method for finding the pile pickup points.

A detailed foundation plan of the building, including sections, is included at the end of the design computations.

DESIGN CONSIDERATIONS

General

The design of a building foundation is affected by several factors. For purposes of designing the foundation for this building, certain assumptions have been made relating to site conditions and economic considerations.

Design loads

As a starting point we will use the design loads from the lateral load analysis presented earlier by John V. Christiansen in "Analysis of Lateral Load Resisting Elements" (see Paper 2 in this series).

Table 1 shows a summary of the design loads that will be used in developing the foundation design.
Note that wind, and not seismic effects, controls the design of the structure.

SITE CONDITIONS

In order to select the proper size prestressed concrete pile to use in the foundation, it is necessary to determine estimated pile lengths and maximum pile loads that can be developed by the soil.

In many instances, the maximum pile load is limited by the soil conditions rather than the structural pile capacity. For example, a dense sand layer underlain by clay may offer sufficient bearing for moderate pile loads while not being able to support the full structural load that the pile may be able to offer.

A soil exploration program should be included in the foundation design. Sufficient borings should be taken to insure that the subsurface conditions can be accurately established. In addition to providing data for developing pile capacities, the soil study normally includes an investigation of estimated settlements. Length of piling is often controlled by settlement requirements rather than bearing capacity.

For purposes of preparing the foundation design for this building, the soil profile shown in Fig. 1 is assumed to represent the soil conditions at the building site.

FACTORS AFFECTING PILE SELECTION

Pile capacities

Piles driven into the weathered granite with the proper size hammer will develop the maximum pile capacity available. Normally pile capacities ranging from 75 to 250 tons are specified for driven prestressed concrete piles used in building foundations. The higher capacities generally become more economical as the length of pile increases.

The most common prestressed concrete pile sizes used in building foundations are the square and octagonal shapes from the 10-in. through the 20-in. size. Table 2 indicates allowable bearing capacities for square piles based on the Uniform Building Code.

\[ N = (0.33f' - 0.27f'_{pu})A_c \]

Table 1. Summary of Design Loads.*

<table>
<thead>
<tr>
<th></th>
<th>Axial load, kips</th>
<th>Lateral load, kips</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A. Typical interior column</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dead load</td>
<td>1358</td>
<td>1902</td>
</tr>
<tr>
<td>Live load</td>
<td>862</td>
<td>1463</td>
</tr>
<tr>
<td>Wind</td>
<td>56</td>
<td>-</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>B. Perimeter wall</strong></th>
<th>Interior panel load at mullion, kips</th>
<th>Corner panel load, kips per ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load</td>
<td>577</td>
<td>25.6</td>
</tr>
<tr>
<td>Live load</td>
<td>86</td>
<td>2.2</td>
</tr>
<tr>
<td>Wind (max)</td>
<td>-</td>
<td>34.4</td>
</tr>
</tbody>
</table>

*Note: In both the interior column and perimeter wall the seismic load effect is not critical.
The effective prestress in Table 2 is assumed to be 750 psi. Concrete strength was entered at 6000 psi in 28 days.

Fig. 2 represents a typical bearing capacity curve which would be developed upon completion of the soil exploration program.

For this particular problem it has been assumed that a gradual increase in bearing capacity is available with depth until the pile reaches the weathered granite at which point high bearing capacity is available for small additional penetration.

### Pile lengths

Maximum economy can be achieved in the use of prestressed concrete piles if piles can be driven full length without field splices. While splices can be installed in the pile to facilitate handling, their use will normally increase the pile cost.

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**Table 2. Bearing Capacity for Various Pile Sizes.**

<table>
<thead>
<tr>
<th>Pile size (square) in.</th>
<th>Bearing capacity, tons</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>87</td>
</tr>
<tr>
<td>12</td>
<td>126</td>
</tr>
<tr>
<td>14</td>
<td>172</td>
</tr>
<tr>
<td>16</td>
<td>225</td>
</tr>
<tr>
<td>18</td>
<td>286</td>
</tr>
<tr>
<td>20</td>
<td>353</td>
</tr>
</tbody>
</table>
Fig. 2. Assumed pile capacity curve (16-in. prestressed concrete pile).

Table 3 indicates practical lengths to be used in selecting pile sizes. These lengths are controlled by the transportation and handling of full length piling.

Piles longer than those shown above may be used under certain conditions. Extra long piles may require additional prestress in the pile for handling purposes.

Since the pile capacity curve indicates pile lengths will be in the 80 to 90-ft range, any of the piles noted above would be satisfactory in meeting the practical length for handling piles.

**Driving conditions**

If hard driving is anticipated at the site, the prestressed pile is sometimes fitted with a steel tip to enable the pile to penetrate the dense material a minimum distance for uplift or other considerations. This tip usually consists of a bearing pile section cast into the lower section of the pile. Since the end area is substantially less than the prestressed pile, the resistance due to end bearing is reduced thus allowing the pile to reach required elevations.

Use of the wave equation method of predicting driving resistance and stresses will assist the designer in predicting the driving behavior of the proposed pile. This analysis, which considers the elastic behavior of the pile during driving has proven to be a useful and reliable aid to both engineers

<table>
<thead>
<tr>
<th>Table 3. Length of Piling for Various Pile Sizes.</th>
</tr>
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<tbody>
<tr>
<td><strong>Pile size</strong></td>
</tr>
<tr>
<td>(square), in.</td>
</tr>
<tr>
<td>10</td>
</tr>
<tr>
<td>12</td>
</tr>
<tr>
<td>14</td>
</tr>
<tr>
<td>16</td>
</tr>
<tr>
<td>18</td>
</tr>
<tr>
<td>20</td>
</tr>
</tbody>
</table>

PCI Journal/September-October 1974
and contractors in selecting proper size pile hammers that will insure the pile is driven efficiently and without damage.


**Pile prices**

Probably the most important factors influencing the selection of prestressed pile are pile prices and availability. When designing a foundation a review of current unit prices for prestressed piles available in the area should be made before selection is made.

Fig. 3 illustrates typical unit prices for square prestressed concrete piles FOB fabrication yard. Fig. 4 compares the cost of a 100-ft length of pile divided by the allowable bearing capacity for the pile.

This is a measure of cost per ton of capacity available for the various
square piles. In this particular analysis, the 12, 14, and 16 in. square piles are all about equal to cost per ton of capacity.

In addition to unit prices based on furnishing at point of fabrication, transportation and driving costs should also be considered. Since it is beyond the scope of this paper to consider the many variables in transportation and driving costs, the 16 in. square pile was arbitrarily chosen for the following design analysis. Actual evaluation of these factors, plus availability of various pile sizes, may indicate other sizes to be the most economical.

**Design calculations**

Step-by-step calculations for designing the footings and pile follow this section. A detailed foundation plan of the building, including sections, is included at the end of the design computations.
TYPICAL INTERIOR FOOTING

Assumed data

Service loads
DL = 1358 kips
LL = 862 kips
Wind load = 56 kips
Base shear = 4 kips

Ultimate loads
DL = 1902 kips
LL = 1463 kips

Design service loads
DL ± LL = 2220 kips
Footing = 80 kips
Total = 2300 kips

For preliminary design purposes try using a 200-ton, 16-in. square pile.

Also, let $f'_{c} = 6000$ psi and $f_{pc} = 750$ psi.

Number of piles

From Section 2909, Uniform Building Code, the allowable axial load is:

$$N = 0.33f'_{c} - 0.27f_{pc} A_{c}$$

$$= [0.33(6000) - 0.27(750)] 256$$

$$= 450$$ kips (or 225 tons)

Hence, our initial premise of using 200-ton 16-in. square piles is satisfactory.

Consequently, the number of piles required is 2300/400 = 5.75. Therefore, for design purposes use six piles.

Pile layout

The pile spacing is:
$3d = 3(16) = 48$ in., or 4 ft on center.

The ultimate load per pile (without considering the footing weight) is:
$DL = 1902/6 = 317$ kips
$LL = 1463/6 = 244$ kips

Beam shear

The beam shear is calculated using Section 11.10.1 of ACI 318-71.

Assume that two piles are outside the distance $d$. Then:

$$V_{u} = 2(DL + LL)$$

$$= 2(317 + 244)$$

$$= 1122$$ kips

From Section 11.4.1, ACI 318-71:

$$\Delta V_{u} = 2\sqrt{V_{c}} = 2\sqrt{400} = 127$$ psi

$$b = 7(12) = 84$$ in.

$$d_{min} = \frac{V_{u}}{b\Delta V_{u}}$$

$$= \frac{1122}{0.85(84)127}$$

$$= 124$$ in.

This depth exceeds the distance $d$ from the support.

Therefore, we must find the minimum depth $d$ required to keep the pile load within the failure plane.

Centerline column to centerline pile ............ 48 in.
One-half column width ............ 15 in.

Pile misalignment ............ 6 in.
One-half pile width ............ 8 in.
Therefore, the minimum depth $d$ for beam shear is 47 in. For design purposes use $d_{\text{min}} = 48$ in.

**Punching shear**

Follow Sections 11.10.2 and 11.10.3, ACI 318-71.

$v_o = 4\sqrt{f'_c} = 4\sqrt{4000} = 253$ psi

$b_o = 4(30 + d) = 4(30 + 48) = 312$ in.

$V_u = 4(317 \times 244) = 2240$ kips

From Eq. (11-25), ACI 318-71:

$v_u = \frac{V_u}{\phi b_o d} = \frac{2240}{0.85(312)48} = 176$ psi (<253 psi, ok)

**Moment steel (long direction)**

$d = 48$ in., $b = 84$ in.

$f_y = 60$ ksi, $f'_c = 4000$ psi

The moment at the column face is:

$PL = 2(561)(2.75 + 0.5) = 3640$ ft-kips

Allow a 6-in. tolerance.

To find the required amount of steel we use the familiar equation for ultimate moment:

$M_u = \phi b d^2 f'_c q(1 - 0.59q)$

Rearranging:

$M_u = \frac{(3640)(12,000)}{f'_c b d^2} = \frac{(0.9)(4000)(84)(48)^2}{p}$

from which $q = 0.065$.

$p = q f'_c / f_y = 0.065(4000)/60,000 = 0.00433$

The required area of steel is:

$A_s = p b d$

$= 0.00433(84)48 = 17.5$ sq in.

Try using twelve #11 bars which gives an area of 18.7 sq in.

This gives a reinforcement ratio of:

$q = A_s / b d = 18.7/(84 \times 48) = 0.00463$

But, the minimum allowable steel ratio is:

$q_{\text{min}} = \frac{200}{f_y} = \frac{200}{60,000} = 0.0033$

Therefore, the steel furnished is satisfactory.

**Development length**

From Section 12.5, ACI 318-71, the required development length of deformed bars must be:

$L = 0.04 A_s f_y / \sqrt{f'_c}$

$= 0.04(1.56)(60,000)/\sqrt{4000}$

$= 59$ in.

but not smaller than:

$0.0004 d f_y = 0.0004(1.375)60,000 = 33$ in.

The development length provided is 4 ft 3 in. minus 3 in. clear length, that

PCI Journal/September-October 1974 47
is 48 in., which does not meet the ACI specifications.

Therefore, try using smaller bars (for example #10 bars).

\[ L = \frac{1.27}{1.56} \times 59 = 48 \text{ in. (ok)} \]

Use 14 #10 bars \( A_s = 17.8 \text{ sq in.} \)

Check the spacing for one layer.

\[ \left[ 84 - 2(4) \right] /13 = 5.85 \text{ in. center to center.} \]

The clear distance is \( 5.85 - 1.37 = 4.48 \text{ in.} \)

\( (4/3) \times 1\frac{1}{2}-\text{in. aggregate} = 2 \text{ in. (ok)} \)

Steel distribution

Follow Section 15.4.4, ACI 318-71. The required reinforcement in band width \( b \) from Eq. (15-1) is:

\[ 2A_s/(\beta + 1) = 221/[(11/7) + 1] = 16.34 \text{ sq in.} \]

Use 17 #9 bars in center band 7 ft wide.

PERIMETER WALL FOOTING

Interior panel at floor beams

First find the number of piles required.

Service loads

<table>
<thead>
<tr>
<th>Service load</th>
<th>Kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>577</td>
</tr>
<tr>
<td>LL</td>
<td>86</td>
</tr>
<tr>
<td>Footing</td>
<td>50</td>
</tr>
<tr>
<td>Total</td>
<td>713</td>
</tr>
</tbody>
</table>

Therefore, the number of piles required is 713/400 = 1.78. For design purposes use two piles under the mullion area with floor beam.

(Note that in the above computation, the wind load was not a controlling factor.)

Panels not carrying floor beams

Service loads

<table>
<thead>
<tr>
<th>Service load</th>
<th>Kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL + LL</td>
<td>30.8</td>
</tr>
<tr>
<td>DL + LL + W</td>
<td>49.0</td>
</tr>
</tbody>
</table>

With the above controlling load, the spacing of 200-ton piles is 400/49 = 8.16 ft.

Therefore, use two piles near corner at 16 ft maximum spacing.
FOOTING AT TRANSFER BEAM

The added load to column at transfer beam is equal to one full panel (12 ft) loading.

\[ DL = 25.6 \text{ kips per ft} \]
\[ LL = 2.2 \text{ kips per ft} \]
\[ 27.8 \text{ kips per ft} \]

Total load = \( 27.8 \times 12 = 334 \text{ kips} \)
Transfer girder = 15 kips

Total load to foundation
349 + 577 + 86 = 1012 kips
Footing = 50 kips

Total = 1062 kips

Therefore, the number of piles required is 1062/400 = 2.65. That is, for design purposes use four piles.

LATERAL LOAD ANALYSIS

Wall shear (transverse), \( V = 369 \text{ kips} \)
Number of piles in wall, \( N = 14 \)
\[ V/N = 369/14 = 26.4 \text{ kips} \]

Minimum vertical load at corner:
1075 - 668 = 407 kips

Assume this load is distributed to six piles. Therefore, the load per pile is 407/6 = 68 kips per pile

Maximum vertical load at interior panel:
713/2 = 356 kips per pile

The maximum moment in the pile is determined from the interaction of the pile with the soil.

The following are two suggested references:

For purposes of this design assume the maximum in the pile to be 75 ft-kips.

Minimum condition
\[ P = 68 \text{ kips, } M = 75 \text{ ft-kips} \]

The pile stresses are calculated from:
\[ f = P/A + P_e/Z \pm M/Z \]
\[ = (68/254) + 0.750 \pm 75(12)/668 \]
\[ = -2365 \text{ psi (comp)} \text{ and} +329 \text{ psi (tens)} \]

Now, since the allowable \( f_a \) is:
0.45(6000)1.33 = 3600 psi
the section is satisfactory in compression. Also, because the allowable tensile stress is 600 psi, the computed tensile stress is below the required limit.

Maximum condition
\[ P = 356 \text{ kips, } M = 75 \text{ ft-kips} \]

Similarly, the pile stresses are computed as above:
\[ f = 356/254 + 0.750 \pm 75(12)/668 \]
\[ = -3499 \text{ psi (comp)} \text{ and} -805 \text{ psi (comp)} \]

As shown above, these stresses are within the allowable stress limits.

Check the interaction formula:
\[ f_a/F_a + f_b/F_b \leq 1.33 \]

From the Uniform Building Code:
\[ F_a = 0.33 f'_a - 0.27 f_{pc} \]
\[ = 0.33(6000) - 0.27(750) \]
\[ = 1778 \text{ psi} \]
\[ F_b = 0.45 f'_a \]
\[ = 0.45(6000) \]
\[ = 2700 \text{ psi} \]

Therefore, applying the above formula:
(1402/1778) + (1347/2700) = 1.287
which is less than 1.33 (ok).
Note: The passive pressure of the soil on the footing in the vicinity of the transverse wall could also be used to resist lateral load.

**Pile dowels**

Use the minimum amount of dowels in the footing (i.e., 1 1/2 percent the pile area).

\[0.015(254) = 3.81 \text{ sq in.}\]

Use four #9 bars.

From previous calculations, the development length is 38 in.

To allow for a reduced prestress at the pile end use a 3 ft 6 in. embedment.

Use a standard hook on top of the bar.

From Section 12.8, ACI 318-71, the required tensile stress in the bar is:

\[f_h = \xi \sqrt{f_c} = 540 \sqrt{4000} = 34,152 \text{ psi}\]

The minimum embedment length is:

\[38(60 - 34)/60 = 16 \text{ in.}\]

**Pile prestress**

From Section 2909, Uniform Building Code, the effective minimum prestress for piles greater than 50 ft in length is 700 psi.

For design purposes, including handling and driving resistance use a minimum prestress of 750 psi. Then the effective prestress force is:

\[P_e = (0.750)(254) = 191 \text{ kips}\]

The design prestress force is computed from:

- 1/2-in. diameter strand (70 percent ultimate) ... 28.91 kips
- Losses (35,000 psi) ........... 5.36 kips
- Design force ............ 23.55 kips

Therefore, the required number of strands is 191/23.55 = 8.1.

For design purposes use nine 1/2-in. diameter strands.

The average compressive force then is:

\[f_c = 9(23.55)/254 = 835 \text{ psi}\]

With reference to Section 2909, Uniform Building Code, use a #5 gage spiral.

Top and bottom ...... 5 turns at 1 in.
Bottom and top (one-third) ....... at 3 in. pitch.
Middle of pile ....... at 6 in. pitch.

The head of the pile should also be checked for stresses using conventional column design since prestress is not present at the pile head. In general, this check is not a critical part of the design procedure.

**Pickup points**

\[f_e = 835 \text{ psi}\]

Use 50 percent impact with no tension.

Try a three-point pick (with equal reactions).

Let the maximum pile length be 90 ft.
The maximum moment is calculated from:

\[ M_{max} = (0.0095) \cdot (w) \cdot (L)^2 \cdot (1.5) \]
\[ = (0.0095) \cdot (0.273) \cdot (90)^2 \cdot (1.5) \]
\[ = 31.5 \text{ ft-kips} \]

The stress is found from:

\[ f = \frac{M}{Z} \]
\[ = \frac{(31.5) \cdot (12)}{668} \]
\[ = 565 \text{ psi} \ (< 835 \text{ psi, ok}) \]

The allowable maximum length is computed from:

\[ L = \sqrt{\frac{M}{(0.0095) \cdot (w) \cdot (1.50)}} \]
\[ = \sqrt{\frac{(0.835) \cdot (668)}{(0.0095) \cdot (0.273) \cdot (1.50) \cdot (12)}} \]
\[ = 109 \text{ ft} \]

Therefore, the selected length is satisfactory.

A detailed foundation plan of the building, including elevation and cross sections, as well as the pile elevation and pile pickup arrangement, is given on the following two pages.

Discussion of this paper is invited. Please forward your discussion to PCI Headquarters by February 1, 1975, to permit publication in the March-April 1975 PCI JOURNAL.
1. Picking arrangement shown above based on vertical pick points of equal reactions.
2. Contractor shall submit pick-up arrangement for handling piles during driving procedure.

PILE PICK-UP POINTS
No scale
NOTES

1. CONCRETE STRENGTH
   ULTIMATE COMPRESSIVE STRENGTH
   CAST-IN-PLACE CONCRETE: 4,000 PSI
   @ 28 DAYS
   PRESTRESSED PILES: 3,500 PSI
   @ TRANSFER 6,000 PSI
   @ 28 DAYS

2. MILD STEEL REINFORCING SHALL BE DEFORMED
   BARS CONFORMING TO ASTM A615-68 GRADE 60.

3. PRESTRESSING STRANDS SHALL BE SEVEN WIRES
   HIGH STRENGTH STEEL TYPE 270.
   ULTIMATE TENSILE STRENGTH: 270,000 PSI
   INITIAL FORCE AT TRANSFER: 26,500 LBS.
   DESIGN FORCE AFTER LOSES: 22,000 LBS.