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.

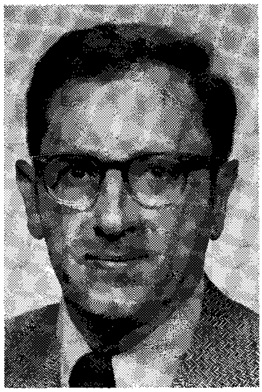
A soil exploration program should be included in the foundation design. Sufficient borings should be taken to insure that the subsurface conditions can be accurately established. In addition to providing data for developing pile capacities, the soil study normally includes an investigation of estimated 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.



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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.

$$N = (0.33f_c' - 0.27f_{pc})A_c$$

*Table 1. Summary of Design Loads.**

A. Typical interior column	Axial load, kips		Lateral load, kips
	Service	Ultimate	Service
Dead load	1358	1902	—
Live load	862	1463	—
Wind	56	—	4
B. Perimeter wall	**Interior panel load at mullion, kips**		**Corner panel load, kips per ft**
Dead load	577		25.6
Live load	86		2.2
Wind (max)	—		34.4

**Note:* In both the interior column and perimeter wall the seismic load effect is not critical.

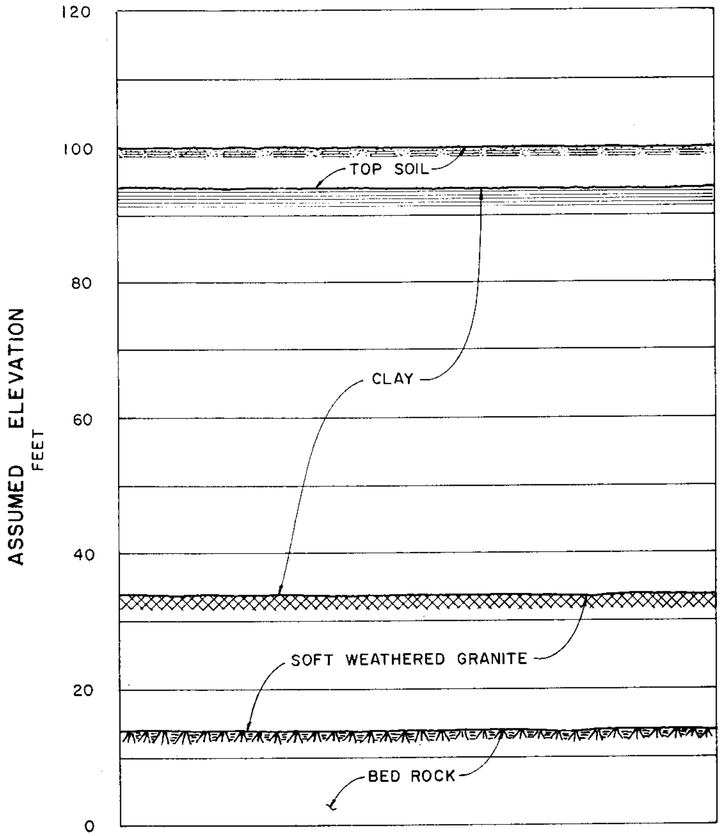


Fig. 1. Assumed soil conditions.

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.

Table 2. Bearing Capacity for Various Pile Sizes.

Pile size (square) in.	Bearing capacity, tons
10	87
12	126
14	172
16	225
18	286
20	353

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.

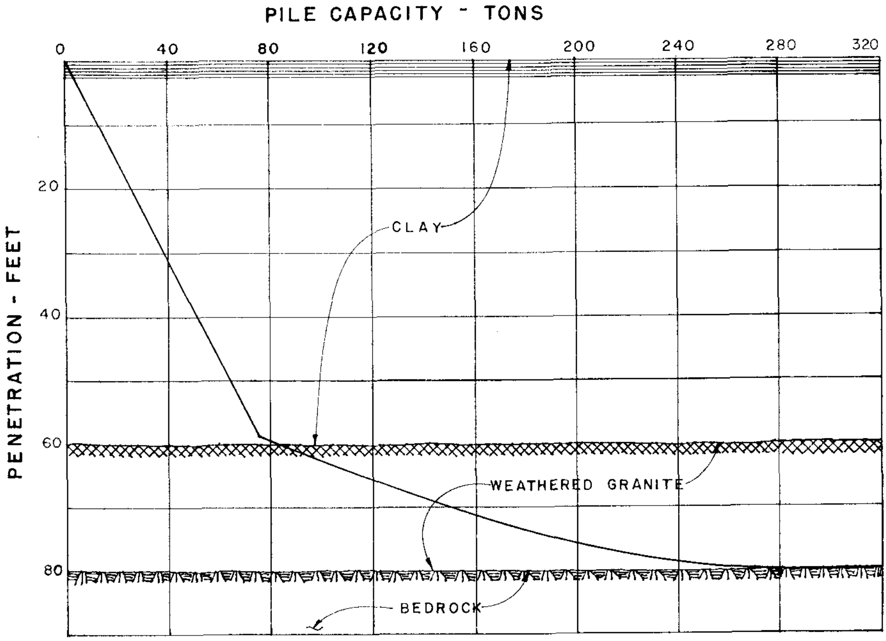


Fig. 2. Assumed pile capacity curve (16-in. prestressed concrete pile).

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 3. Length of Piling for Various Pile Sizes.

Pile size (square), in.	Length, ft
10	100
12	110
14	120
16	150
18	170
20	190

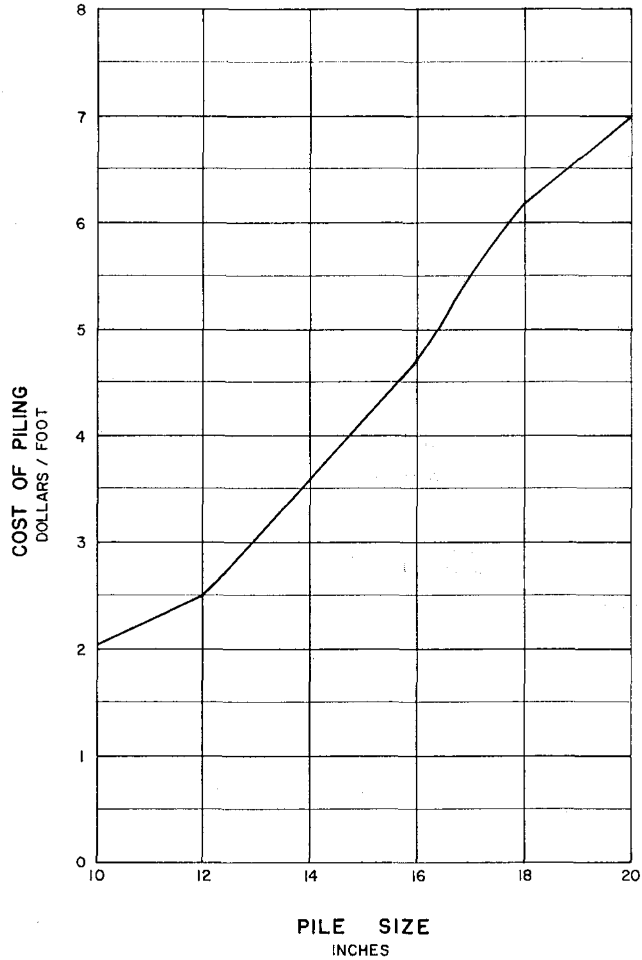


Fig. 3. Cost of piling in relation to pile size.

and contractors in selecting proper size pile hammers that will insure the pile is driven efficiently and without damage.

For further details of this method see E. A. L. Smith, "Pile-Driving Analysis by the Wave Equation," Transactions–ASCE, 3306, Vol. 127, 1962, pp. 1145-1193.

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

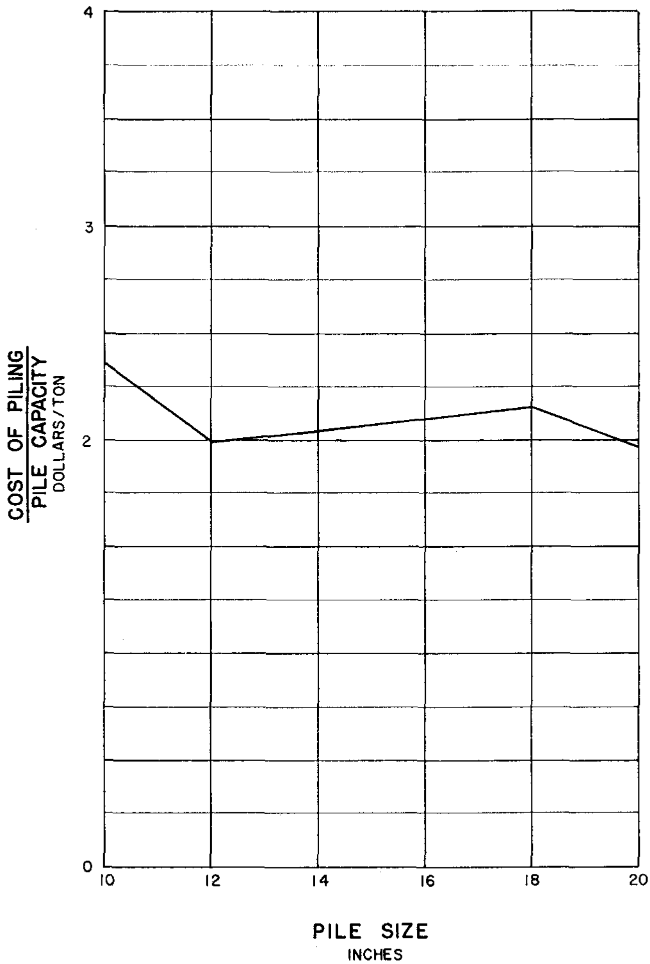


Fig. 4. Ratio of cost of piling to pile capacity in relation to pile size. (Note that the cost is based on 100-ft pile lengths.)

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:

$$\begin{aligned} N &= (0.33f_c' - 0.27f_{pc})\,A_c \\ &= [0.33(6000) - 0.27(750)]\,256 \\ &= 450 \text{ kips (or 225 tons)} \end{aligned}$$

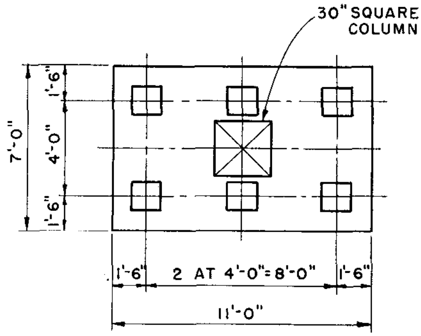
Hence, our initial premise of using 200-ton 16-in. square piles is satisfactory.

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:

$$\begin{aligned} V_u &= 2(DL + LL) \\ &= 2(317 + 244) \\ &= 1122 \text{ kips} \end{aligned}$$

From Section 11.4.1, ACI 318-71:

$v_c = 2\sqrt{f_c'} = 2\sqrt{4000} = 127$ psi
$b = 7(12) = 84$ in.

$$\begin{aligned} d_{min} &= \frac{V_u}{\phi b v_u} \\ &= \frac{1122}{0.85(84)127} \\ &= 124 \text{ in.} \end{aligned}$$

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.
	33 in.
Pile misalignment	6 in.
One-half pile width	8 in.
	47 in.

Therefore, the minimum depth d for beam shear is 47 in. For design purposes use $d_{min} = 48$ in.

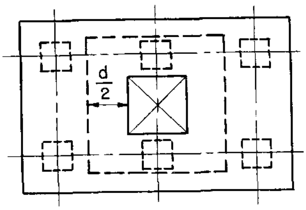
Punching shear

Follow Sections 11.10.2 and 11.10.3, ACI 318-71.

$$v_c = 4\sqrt{f_c'} = 4\sqrt{4000} = 253 \text{ psi}$$

$$b_o = 4(30 + d) = 4(30 + 48) = 312 \text{ in.}$$

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



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

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

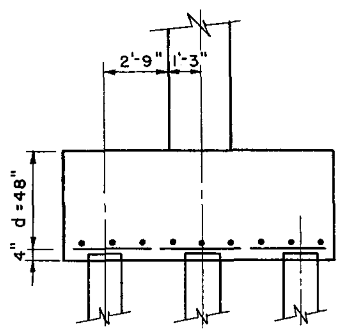
Moment steel (long direction)

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

$f_y = 60$ ksi, $f_c' = 4000$ psi

The moment at the column face is:

$$PL = 2(561)(2.75 + 0.5) = 3640 \text{ ft-kips}$$



Allow a 6-in. tolerance.

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

$$M_u = \phi bd^2 f_c' q\,(1 - 0.59q)$$

Rearranging:

$$\frac{M_u}{f_c' bd^2} = \frac{(3640)(12{,}000)}{(0.9)(4000)(84)(48)^2}$$

from which $q = 0.065$.

$$p = qf_c'/f_y = 0.065(4000)/60{,}000 = 0.00433$$

The required area of steel is:

$$A_s = pbd = 0.00433(84)48 = 17.5 \text{ sq in.}$$

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

This gives a reinforcement ratio of:

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

But, the minimum allowable steel ratio is:

$$q_{min} = 200/f_y = 200/60{,}000 = 0.0033$$

Therefore, the steel furnished is satisfactory.

Development length

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

$$L = 0.04A_b f_y/\sqrt{f_c'} = 0.04(1.56)(60{,}000)/\sqrt{4000} = 59 \text{ in.}$$

but not smaller than:

$$0.0004\, d_b f_y = 0.0004(1.375)60{,}000 = 33 \text{ in.}$$

The development length provided is 4 ft 3 in. minus 3 in. clear length, that is 48 in., which does not meet the ACI specifications.

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

$L = (1.27/1.56)59 = 48$ in. (ok)

Use 14 #10 bars ($A_s = 17.8$ sq in.)

Check the spacing for one layer.

$[84 - 2(4)]/13 = 5.85$ in. center to center.

The clear distance is $5.85 - 1.37 = 4.48$ in.

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

Moment steel (short direction)

The moment at the column face is:

$$PL = 3(561)(0.75 + 0.5) = 1683 \text{ ft-kips}$$

This value is less than one-half the longitudinal moment.

Therefore, use the minimum amount of steel, Eq. (10-1), ACI 318-71:

$$A_s(\text{min.}) = pbd = 0.0033(12 \times 11)48 = 20.91 \text{ sq in.}$$

Try using 21 #9 bars ($A_s = 21$ sq in.)

Development length

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

$$L = 0.04A_b f_y/\sqrt{f_c'} = 0.04(1.00)60{,}000/\sqrt{4000} = 38 \text{ in.}$$

By strength considerations, approximately twice the steel area is already present.

Therefore, $L_{req} = 0.5(38) = 19$ in.

The development length provided is: 2 ft 4 in. minus 3 in., or 25 in. This is greater than the required 19 in. and hence the development length furnished is satisfactory.

Steel distribution

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

$$2A_s/(\beta + 1) = 2(21)/[(11/7) + 1] = 16.34 \text{ sq in.}$$

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

DL	=	577 kips
LL	=	86 kips
Footing	=	50 kips
		713 kips

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

$DL = 25.6 + 3$ (footing weight) $= 28.6$ kips per ft

$LL = 2$ kips per ft

$W = 34.4$ kips per ft (maximum at corner)

$DL + LL = 28.6 + 2.2 = 30.8$ kips per ft

$(DL + LL + W)/1.33 = 49.0$ kips per ft

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.

$DL = 25.6$ kips per ft
$LL = \ \ 2.2$ kips per ft

27.8 kips per ft

Total load $= 27.8 \times 12 = 334$ kips
Transfer girder $= 15$ kips

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

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), $V = 369$ kips

Number of piles in wall, $N = 14$

$V/N = 369/14 = 26.4$ kips

Minimum vertical load at corner:

$$1075 - 668 = 407 \text{ kips}$$

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:

$$713/2 = 356 \text{ kips per pile}$$

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

The following are two suggested references:

1. Broms, B. B., "Design of Laterally Loaded Piles," *Journal of Soil Mechanics and Foundations,* ASCE, May 1965.
2. Reese, L. C., and Matlock, H., "Non-Dimensional Solutions for Laterally Loaded Piles," Bureau of Engineering Research No. 29, University of Texas, September 1956.

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

Minimum condition

$P = 68$ kips, $M = 75$ ft-kips

The pile stresses are calculated from:

$$f = P/A + Pe/Z \pm M/Z$$
$$= (68/254) + 0.750 \pm 75(12)/668$$
$$= -2365 \text{ psi (comp) and}$$
$$+329 \text{ psi (tens)}$$

Now, since the allowable f_c is:

$$0.45(6000)1.33 = 3600 \text{ psi}$$

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

$P = 356$ kips, $M = 75$ ft-kips

Similarly, the pile stresses are computed as above:

$$f = 356/254 + 0.750 \pm 75(12)/668$$
$$= -3499 \text{ psi (comp) and}$$
$$-805 \text{ psi (comp)}$$

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

Check the interaction formula:

$$f_a/F_a + f_b/F_b \leqslant 1.33$$

From the Uniform Building Code:

$$F_a = 0.33 f_c' - 0.27 f_{pc}$$
$$= 0.33(6000) - 0.27(750)$$
$$= 1778 \text{ psi}$$

$$F_b = 0.45 f_c'$$
$$= 0.45(6000)$$
$$= 2700 \text{ psi}$$

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., 1½ percent the pile area).

$0.015(254) = 3.81$ 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:

$38(60 - 34)/60 = 16$ 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:

½-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 ½-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.

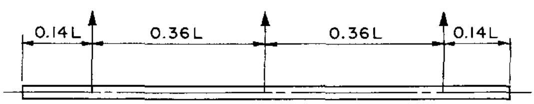
Pickup points

$f_e = 835$ 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:

$$
\begin{aligned}
M_{max} &= (0.0095)\,(w)\,(L)^2\,(1.5)\\
&= (0.0095)\,(0.273)\,(90)^2\,(1.5)\\
&= 31.5 \text{ ft-kips}
\end{aligned}
$$

The stress is found from:

$$
\begin{aligned}
f &= M/Z\\
&= (31.5)\,(12)/668\\
&= 565 \text{ psi } (<835 \text{ psi, ok})
\end{aligned}
$$

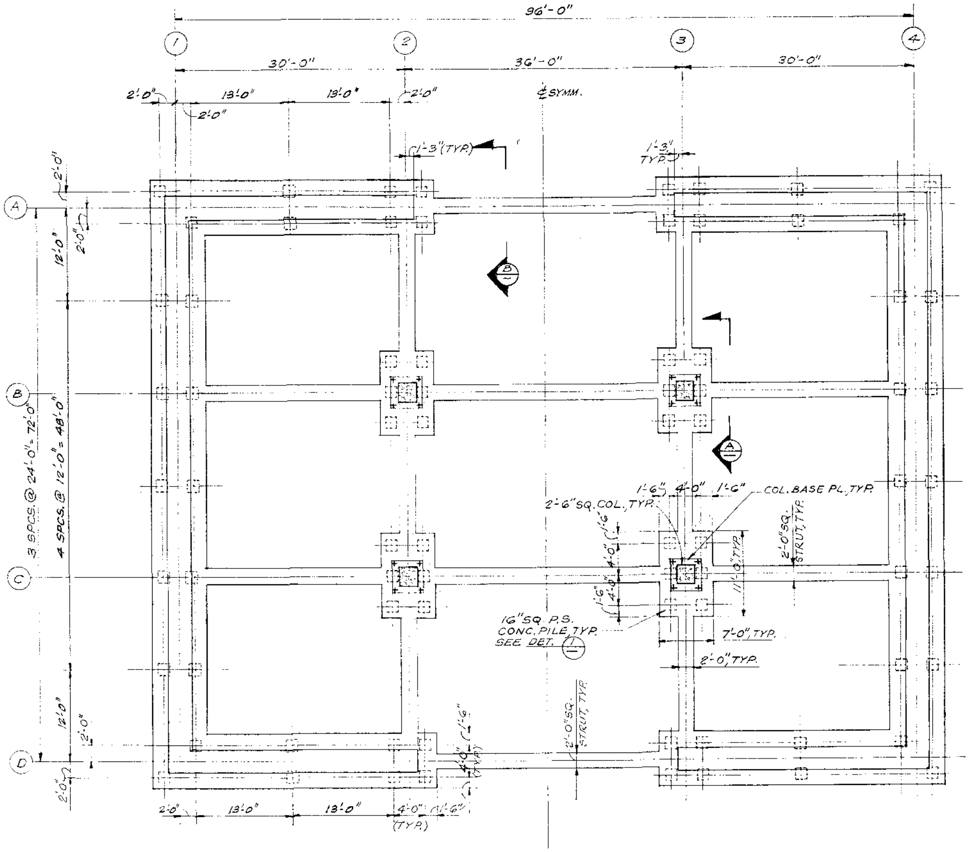
The allowable maximum length is computed from:

$$
\begin{aligned}
L &= \sqrt{\frac{M}{(0.0095)\,(w)\,(1.50)}}\\
&= \sqrt{\frac{(0.835)\,(668)}{(0.0095)\,(0.273)\,(1.50)\,(12)}}\\
&= 109 \text{ ft}
\end{aligned}
$$

Therefore, the selected length is satisfactory.

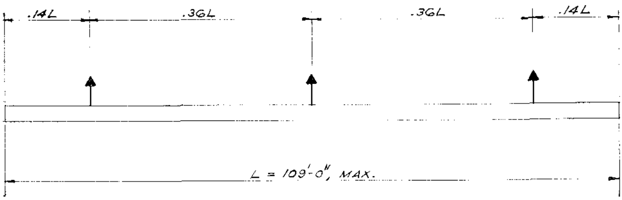
A detailed foundation plan of the building, including elevation and cross sections, as well as the pile elevation and pile pickup arrangement, is given on the following two pages.

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.



FOUNDATION PLAN

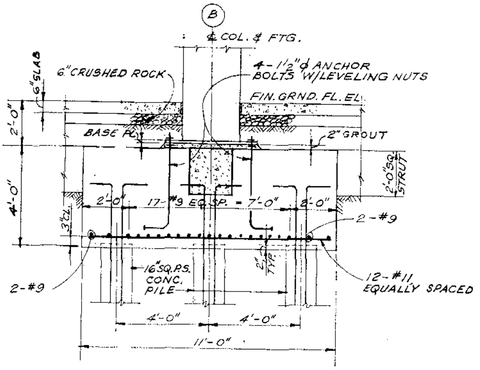
1/8" = 1'-0"



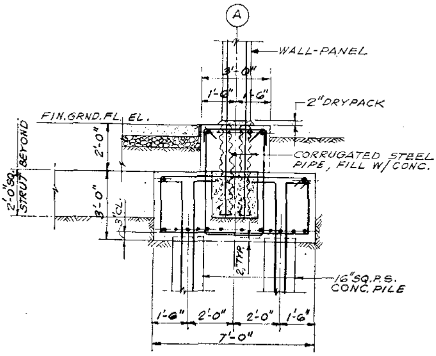
1. PICKING ARRANGEMENT SHOWN ABOVE BASED ON VERTICAL PICK POINTS OF EQUAL REACTIONS.
2. CONTRACTOR SHALL SUBMIT PICK-UP ARRANGEMENT FOR HANDLING PILES DURING DRIVING PROCEDURE.

PILE PICK-UP POINTS

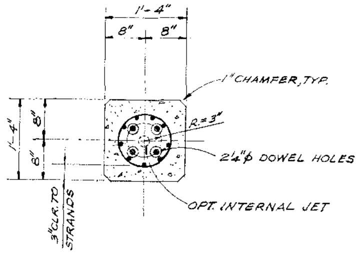
NO SCALE



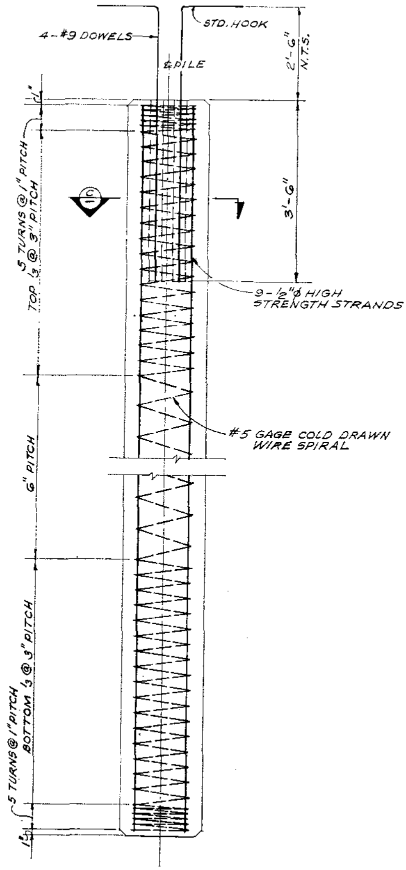
SECTION (A)
3/8" = 1'-0"



SECTION (B)
3/8" = 1'-0"



SECTION (C)
1" = 1'-0"



ELEVATION
16" SQ. P.S. CONC. PILE
DETAIL (1)
1" = 1'-0"

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 WIRE HIGH STRENGTH STRAND TYPE 270K.
   ULTIMATE TENSILE STRENGTH . . 270,000 PSI
   INITIAL FORCE AT TRANSFER . . . 28,910 #/STRAND
   DESIGN FORCE AFTER LOSSES . . . 23,550 #/STRAND