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16. Abstract

In this report, a detailed description is made of the established procedures to design deep foundations subjected to vertical loads on the basis of preboring pressuremeter tests. Both the ultimate capacity and load-settlement calculations are outlined in the form of step-by-step procedures. Design examples are given and solved. An indication of the precision of the methods is presented by comparing the predicted behavior to the measured behavior for actual case histories.

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PRESSUREMETER DESIGN OF VERTICALLY LOADED PILES

by

Jean-Louis Briaud and Joe Anderson

Research Report 340-2

The Pressuremeter and the Design of Highway Related Foundations
Research Study 2-5-83-340

Sponsored by

State Department of Highways and Public Transportation
In Cooperation with the
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June 1983

SUMMARY

In this report a detailed description is made of the established procedures to design deep foundations subjected to vertical loads on the basis of preboring pressuremeter tests. Both the ultimate capacity and settlement calculations are presented in the form of step-by-step design procedures.

The ultimate point bearing capacity, q_{max}, is given by

$$q_{max} = kp^*_{Le} + q_{ov}$$

where k is the pressuremeter bearing capacity factor, p^*_{Le} is the equivalent net limit pressure obtained from preboring pressuremeter tests performed near the pile point, and q_{ov} is the vertical total pressure at the pile point. The bearing capacity factor k depends on the relative depth of embedment of the foundation, the type of soil, the shape of the foundation, and the method of installation. The ultimate side friction, f_{max}, is also a function of the type of soil and the method of installation as well as the type of foundation material. Charts for k and f_{max} have been proposed by Menard and Gambin in 1963, Baguelin, Jezequel, and Shields in 1978, and Bustamante and Gianeselli in 1982.

The charts for the three methods are presented and used to solve several example problems. The results of those examples show that generally the Bustamante-Gianeselli method gives the lowest ultimate capacity values, that the Menard-Gambin method gives higher values and

that the Baguelin-Jezequel-Shields method give values which are slightly higher than the values obtained with the Menard-Gambin method.

For the calculation of settlement for deep foundations, the load transfer approach has been used. The unit point bearing-point movement (q-w) curve and unit side friction-pile movement (f-w) curves have been modeled as linear elastic-plastic. The ultimate values, q_{max}, are obtained by the three methods mentioned above. Each of these methods also propose values for the slope of the elastic portion of the transfer curves. This slope is given as a function of either the pressuremeter first loading modulus, E_o, or the pressuremeter reload modulus, E_r, and the pile width and shape. The Menard-Gambin and the Baguelin-Jezequel-Shields methods are a simple linear elastic-plastic model, whereas Bustamante-Gianeselli propose a bilinear elastic-plastic model. These q-w and f-w curves are to be input into a conventional beam-column computer program to obtain the complete load-settlement curve for the pile. An approximate hand calculation method is also presented for obtaining the load-settlement curve.

Examples are used to illustrate the design procedures for various cases. An example of the hand method for calculation of the load-settlement curve is given for each of the three design procedures. Experimental evidence is presented for comparison between predicted and measured behavior. The results of 192 pile load tests are presented for the Bustamante-Gianeselli method for ultimate pile capacity. It must be emphasized that one of the critical elements in the accuracy of the predictions is the performance of quality pressuremeter tests and that such quality pressuremeter tests can only be performed by trained professionals.

IMPLEMENTATION STATEMENT

This report gives the details of existing pressuremeter methods for the design of vertically loaded piles. These methods require the use of a new piece of equipment: a preboring pressuremeter. These methods are directly applicable to design practice and should be used in parallel with current methods for a period of time until a final decision can be made as to their implementation.

ACKNOWLEDGEMENTS

The authors are grateful for the continued support and encouragement of Mr. George Odom of the Texas State Department of Highways and Public Transportation. Mr. Larry Tucker's help in completing the report was very valuable.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration, or the State Department of Highways and Public Transportation. This report does not constitute a standard, a specification, or a regulation.

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GLOSSARY OF TERMS

A = Area of section, ft^2*

A_p = area of section at point, ft^2

C = coefficient of strain dependent on the ratio He/R and the method of installation of the pile, dimensionless

D = pile diameter, ft

E = Young's Modulus for the pile, lb/ft^2

E_o = the pressuremeter first loading modulus, lb/ft^2

E_R = the pressuremeter reload modulus, lb/ft^2

f_{max} = the ultimate skin friction, lb/ft^2

H_e,h = the equivalent depth of embedment of the pile, ft

k = the pressuremeter bearing capacity factor, dimensionless

L = the length of the pile, ft

P = the load in the pile, lb

p_{OH} = the total horizontal stress at rest (estimated), lb/ft^2

p_L^* = the net limit pressure = $p_L - p_{OH}$

p_L = the limit pressure (from pressuremeter test) lb/ft^2

p_{Le}^* = the equivalent net limit pressure at the point, lb/ft^2

Q_p = the point bearing capacity, lb

Q_S = the skin friction, lb

Q_T = the total vertical capacity, lb

q_{max} = the ultimate bearing capacity at the point, lb/ft^2

q_{ov} = vertical total pressure at the pile point, lb/ft^2

R = the pile radius, ft

* The units shown are not the only ones used in the report.

R_o = 1.0 if using U.S. units, ft

0.30 if using S.I. units, m

W = the weight of the pile, lb

w = the movement of the pile shaft, ft

ΔZ_i = the thickness of a layer i, ft

α = the rheological coefficient, dimensionless

α' = 0.76 R when R is in feet

2.50 R when R is in meters

λ = the pile shape factor, dimensionless

ν = Poisson's Ratio (approx. 0.33), dimensionless

CHAPTER 1 - INTRODUCTION

The established procedures to design deep foundations subjected to vertical loads on the basis of preboring pressuremeter tests are presented in detail in this report. In Chapter 2 the ultimate capacity calculations are described in step-by-step procedures. The procedures for calculating settlement are described in detail in Chapter 3. Some design examples are then given and solved in Chapter 4 for various cases. Finally, in Chapter 5, the accuracy of the methods are evaluated by comparing predicted and measured behavior for numerous case histories.

It must be emphasized that one of the critical elements for the successful prediction of deep foundation behavior using these design rules is the performance of quality pressuremeter tests. Such quality pressuremeter tests can only be performed by trained professionals.

CHAPTER 2 - VERTICAL ULTIMATE LOAD

2.1. Point Capacity

The point bearing capacity is calculated as follows:

$Q_p = A_p q_{max}$

A_p = area of the point

q_{max} = ultimate bearing capacity at the point

$$q_{max} = k p^*_{Le} + q_{ov}$$

k = pressuremeter bearing capacity factor

p^*_L = net limit pressure = $p_L - p_{oh}$

p_{oh} = total horizontal stress at rest (estimated)

p_L = limit pressure (from test)

p^*_{Le} = equivalent net limit pressure near the point

q_{ov} = vertical total pressure at the pile point

2.1.1 Calculating p^*_{Le}, The Equivalent Limit Pressure

$$p^*_{Le} = \sqrt[n]{p^*_{L1} \times p^*_{L2} \times \ldots \times p^*_{Ln}}$$

where $p^*_{L1}, \ldots, p^*_{Ln}$ are the net limit pressures obtained from tests performed within the + 1.5B/ - 1.5B zone near the point.

2.1.2 Calculating H_e (or D), The Equivalent Depth of Embedment

$$H_e = \sum_{1}^{n} \frac{\Delta Z_i \, p^*_{Li}}{p^*_{Le}}$$

where p_{Li} are the limit pressures obtained from tests between the ground surface and the tip of the pile, ΔZ_i are the thicknesses of the elementary layers corresponding to the pressuremeter tests.

2.1.3 Determining k, The Pressuremeter Bearing Capacity Factor

The pressuremeter bearing capacity factor, k, is a function of the type of soil, and the embedment and shape of the pile. This factor may be determined using one of three methods.

The first method was proposed by Menard (1) and shall be referred to as Method A. In this method soils are broken down into four categories which are found in Figure 1. After calculating the penetration depth to radius ratio, k is obtained using Figure 2a for piles or Figure 2b for cast-in-situ walls.

The second method, proposed by Baguelin, Jezequel and Shields (2), shall be referred to as Method B. Method B uses several graphs. This method plots k vs the ratio of penetration depth to foundation width. Values of k for bored piles may be obtained from Figures 3a through 3d; each figure represents one type of soil. Similarly, Figures 4a through 4d are used for driven piles.

The third method shall be referred to as Method C. This method was proposed by Bustamante and Gianeselli (3). Method C uses soil categories which are found in Figure 5. As in

Ranges of Pressures Limit p_L	Nature of Soil	Soil Categories
0 - 25100 psf (0 - 12 bars)	Clay	Category I
0 - 14600 psf (0 - 7 bars)	Silt	
37600 - 83500 psf (18 - 40 bars)	Firm Clay or Marl	Category II
14600 - 62700 psf (12 - 30 bars)	Compact Silt	
8400 - 16700 psf (4 - 8 bars)	Compressible Sand	
20900 - 62700 psf (10 - 30 bars)	Soft or Weathered Rock	
20900 - 41800 psf (10 - 20 bars)	Sand and Gravel	Category III
83500 - 20900 psf (40 - 100 bars)	Rock	
62700 - 125000 psf (30 - 60 bars)	Very Compact Sand and Gravel	Category IIIA

Fig. 1 - Soil Categories for Bearing Capacity Determination by Method A (from Reference 1).

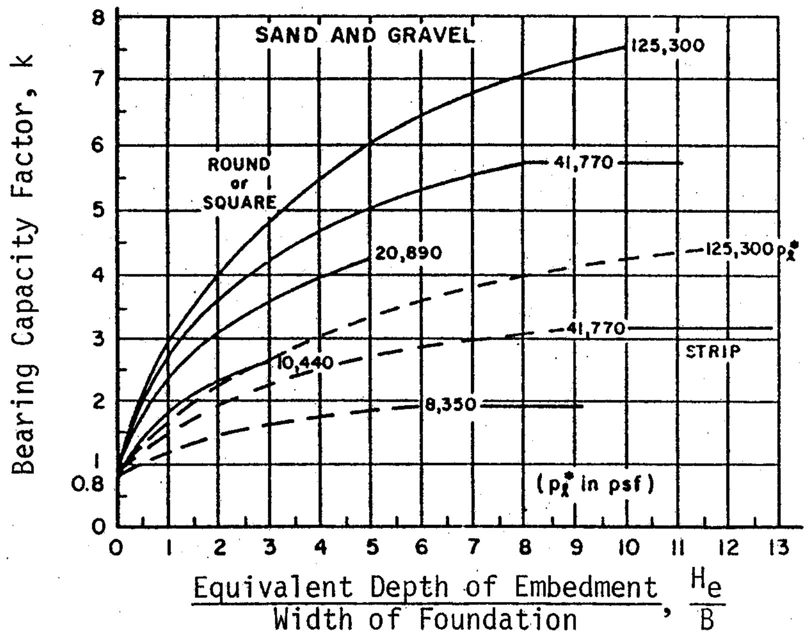


Fig. 3a

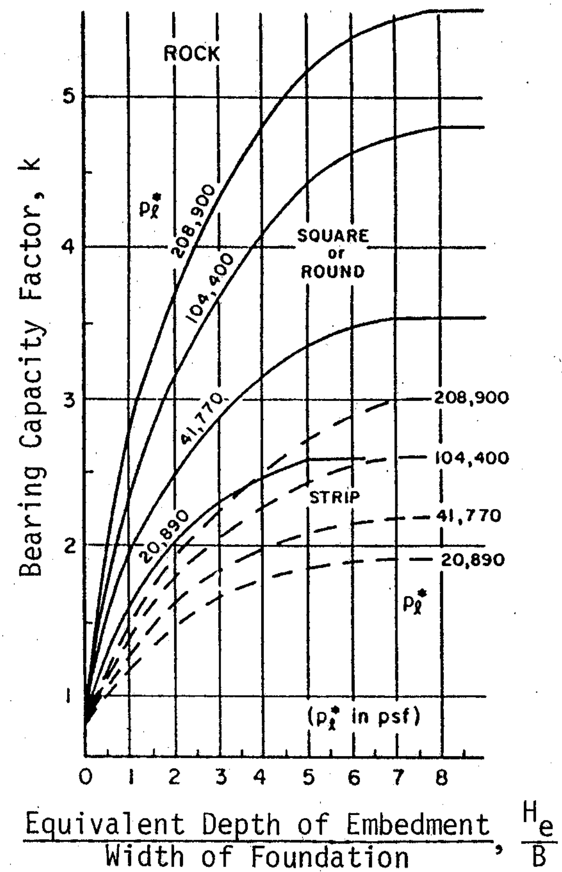


Fig. 3b

Fig. 3 - Bearing Capacity Factor Charts for Bored Piles: for use Method B (From Reference 2).

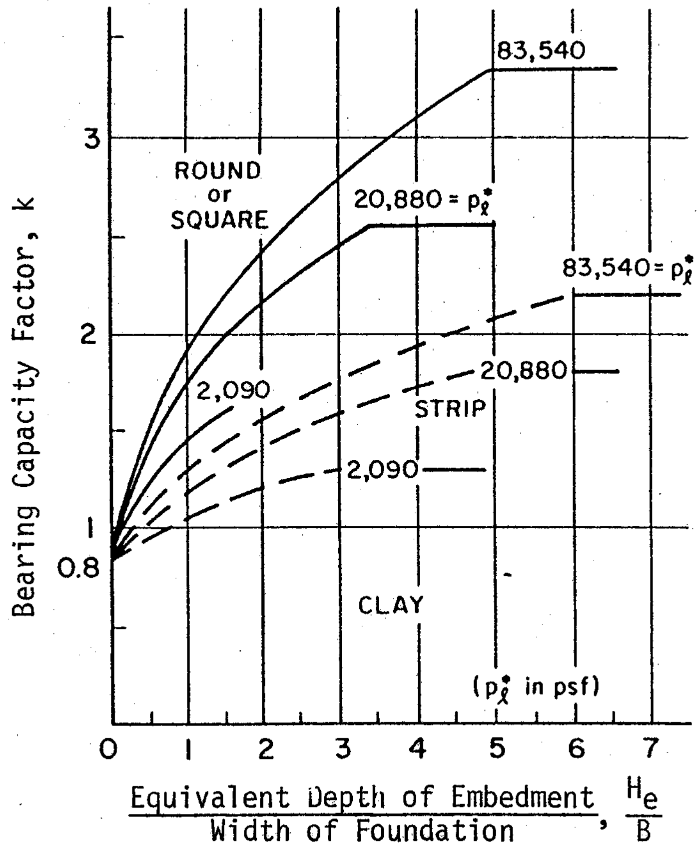


Fig. 3c

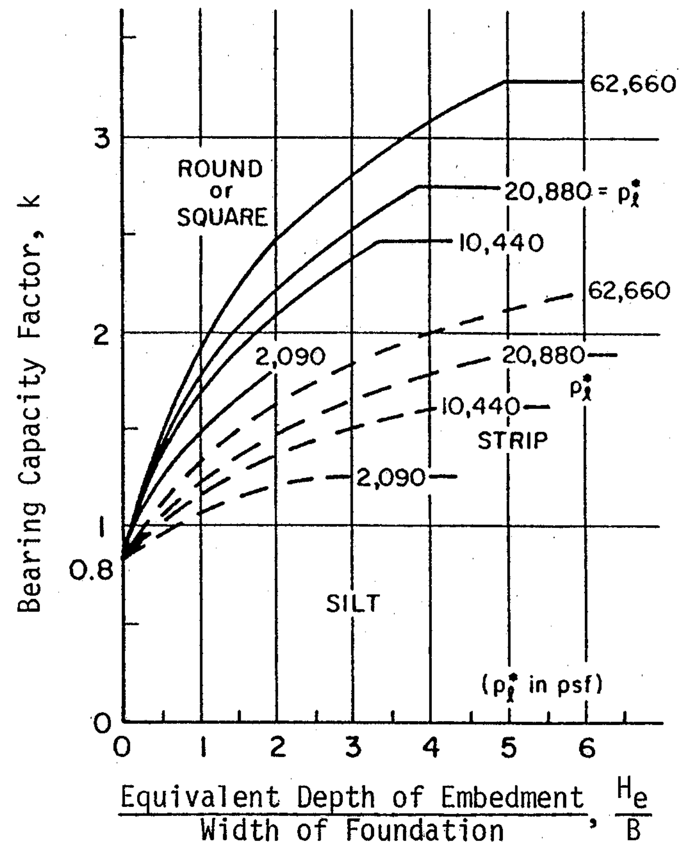


Fig. 3d

Fig. 3 – Continued

k = Bearing Capacity Factor

$$D/B = \frac{\text{Depth of Embedment}}{\text{Width of Foundation}}$$

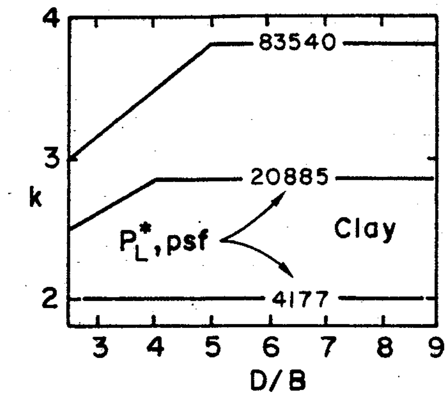


Fig. 4a

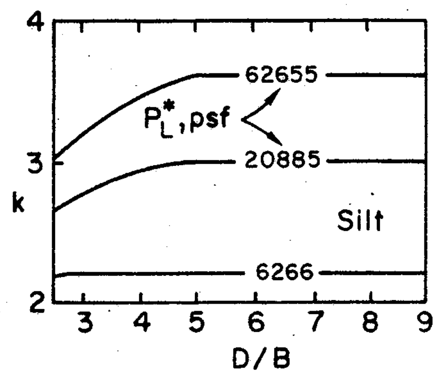


Fig. 4b

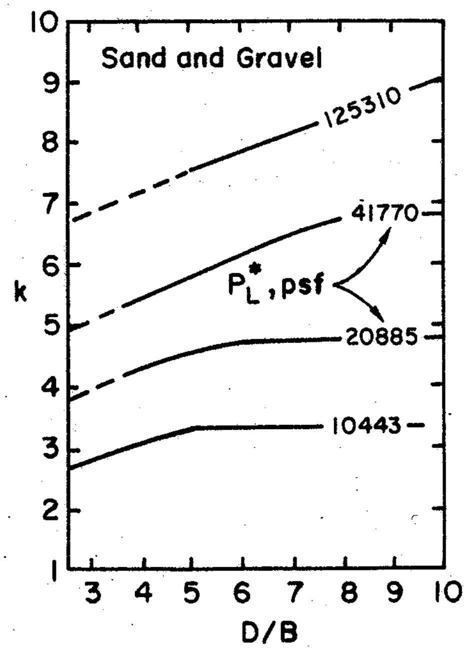


Fig. 4c

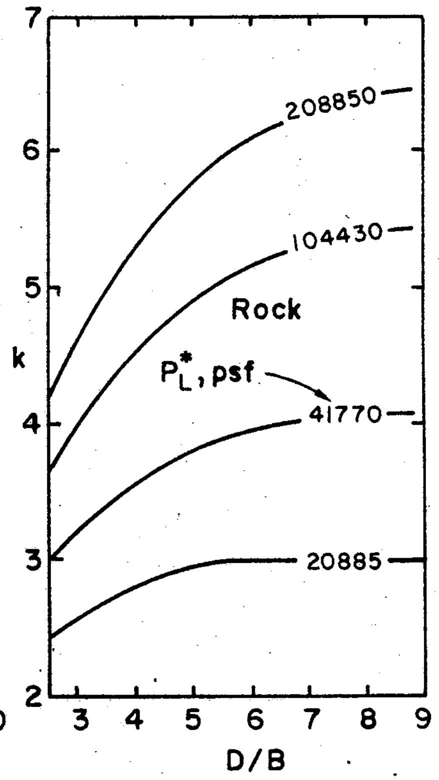


Fig. 4d

Fig. 4 - Bearing Capacity Factor Charts for Driven Piles; for Use With Method B (from Reference 2)

Limit Pressure p_L	Soil Type	Category
0 - 14600 psf (0 - 7 bars)	Soft Clay	1
0 - 16700 psf (0 - 8 bars)	Silt and Soft Chalk	1
0 - 14600 psf (0 - 7 bars)	Loose Clayey, Silty or Muddy Sand	1
20900 - 41800 psf (10 - 20 bars)	Medium Dense Sand and Gravel	2
25100 - 62700 psf (12 - 30 bars)	Clay and Compact Silt	2
31300 - 83500 psf (15 - 40 bars)	Marl and Limestone-Marl	2
20900 - 52200 psf (10 - 25 bars)	Weathered Chalk	2
52200 - 83500 psf (25 - 40 bars)	Weathered Chalk	2
> 62700 psf (> 30 bars)	Fragmented Chalk	2
> 94000 psf (> 45 bars)	Very Compact Marl	2
> 52200 psf (> 25 bars)	Dense to Very Dense Sand and Gravel	3
> 94000 psf (> 45 bars)	Fragmented Rock	3

Fig. 5 - Soil Categories for Bearing Capacity Determination by Method C (from Reference 3).

As in Method A, the penetration depth to radius ratio is calculated. The k value is then determined from Figure 6. This figure has separate curves for driven and bored piles.

2.2. Side Friction

The skin friction is determined as follows:

$$Q_s = \sum_{1}^{n} f_{max} \pi D \Delta Z_i$$

f_{max} = ultimate skin friction in layer i

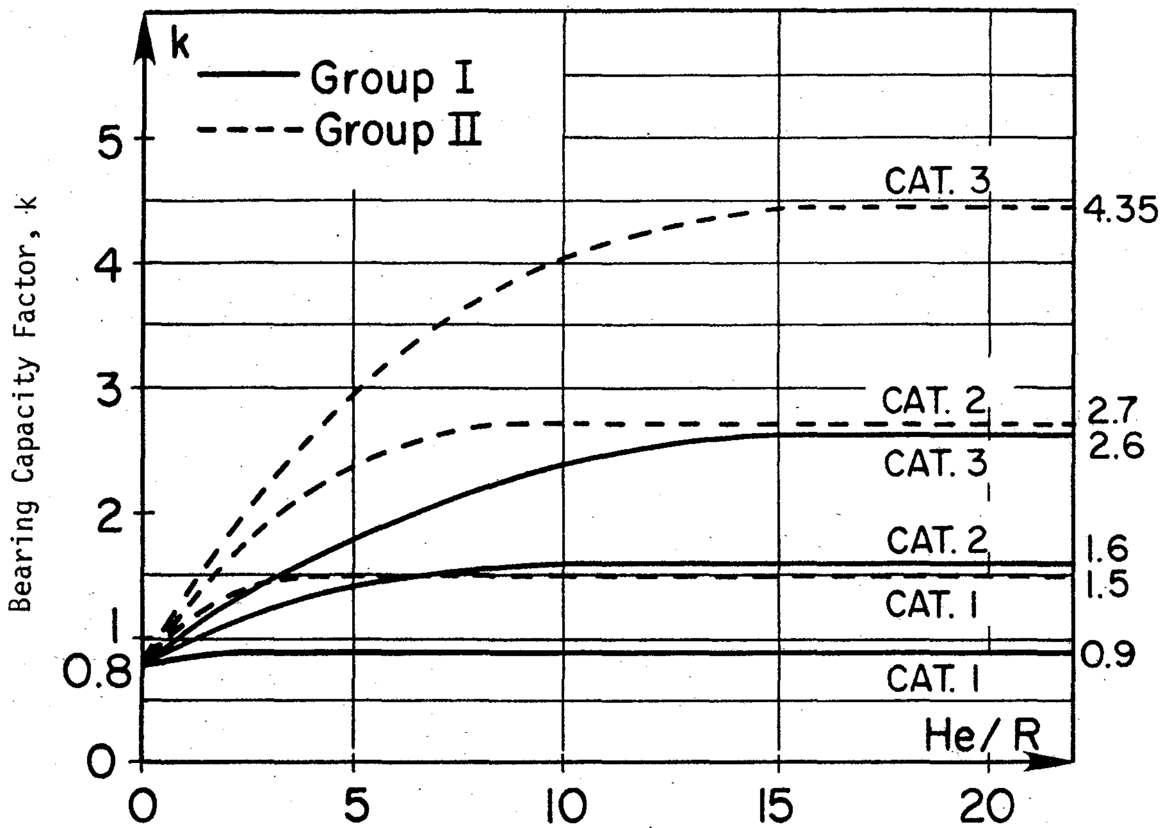
ΔZ_i = thickness of layer i

D = pile diameter

2.2.1 Obtaining f_{max}, The Ultimate Skin Friction.

As for the bearing capacity factor, three methods may be used to determine the ultimate skin friction, f_{max}.

The first method was proposed by Menard (1), and will be referred to as Method A. In this method it is assumed that an increase in skin friction will occur near the tip of the pile up to a height of three diameters from the point, due to increased confining pressures in this region. Using p_L^* for the soil, f_{max} is obtained using the appropriate curve on Figure 7. Menard recommends that for steel piles or piles with a permanent lining, the values obtained form Curve A and Curve B be reduced by 20% in cohesive soils and 30% in sands or submerged sands and gravels. It must be noted that the values in Figure 7 are for a pile diameter of up to 60 cm and should be reduced by 10% for a diameter of



$$\frac{\text{Equivalent Depth of Embedment}}{\text{Radius of Foundation}}, \frac{H_e}{R}$$

Group I - Bored Piles
Group II - Driven Piles

Fig. 6 - Bearing Capacity Factor Chart for Use With Method C (from Reference 3).

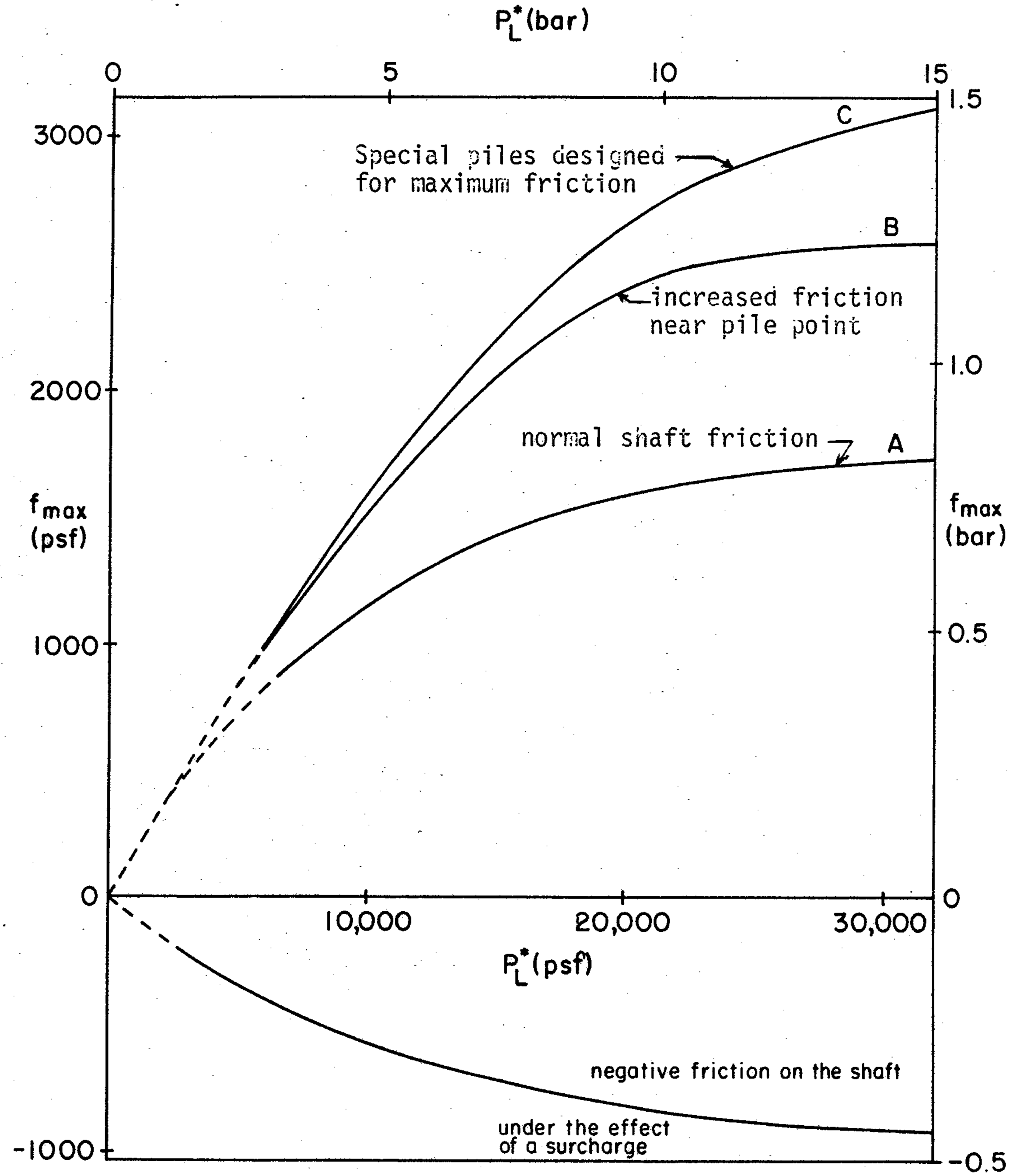


Fig. 7 - Skin Friction Design Chart for Use With Method A (from Reference 1).

80 cm and 30% for a diameter of 120 cm.

The second method, Method B, was proposed by Baguelin, Jezequel and Shields (2). The value of f_{max} may be obtained from Figure 8 using p_L^* of the soil and the appropriate curve. Each of the four curves corresponds to a soil type and installation procedure.

The third method, which was proposed by Bustamante and Gianeselli (3), shall be referred to as Method C. The soil and foundation type $(A, A_{bis}, B, C, D, E, F)$ must first be obtained from Figure 9. The value of f_{max} is then obtained for the corresponding value of p_L^* and from the appropriate curve on Figure 10.

2.3. Total Vertical Capacity

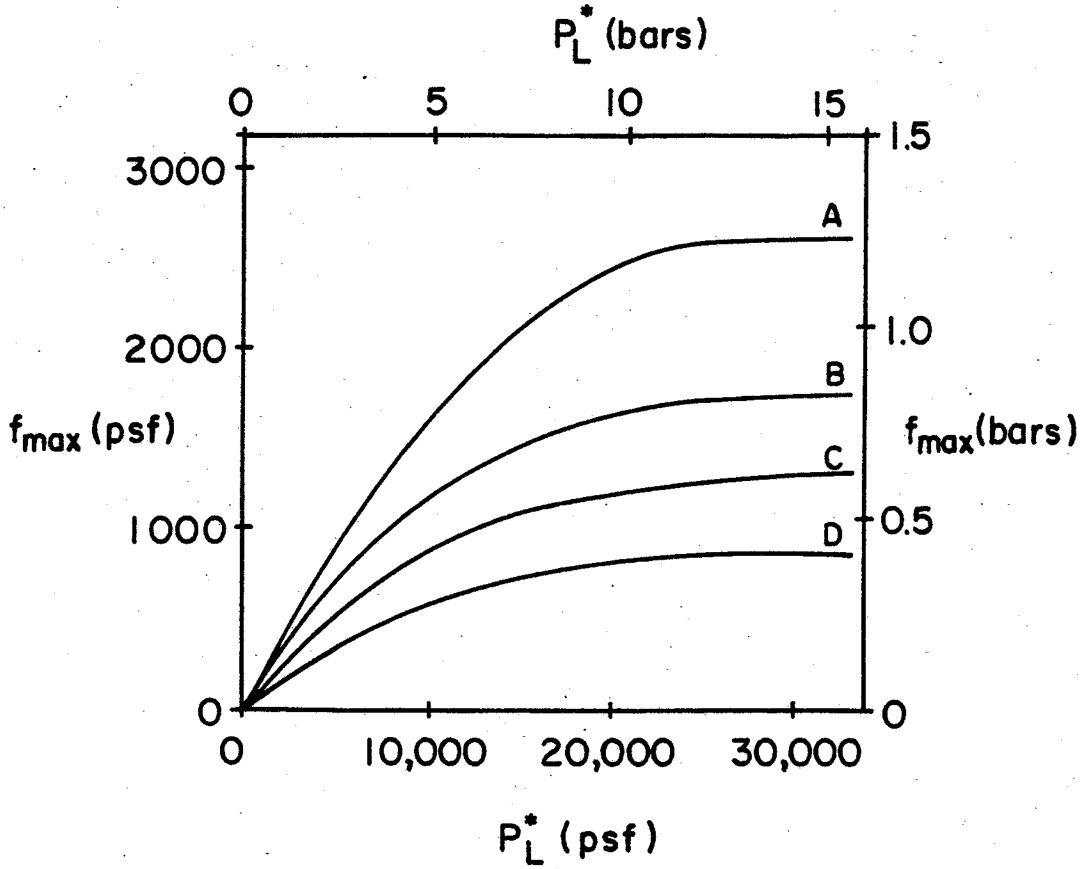
$$Q_T = Q_p + Q_s$$

the recommended load at the ground surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

where W is the weight of the pile.

The factor of safety of 3 for the point load is due to the fact that it is difficult to transfer load to the point of a pile.



A - Displacement/Concrete/Granular

B - No Displacement/Concrete/Any Soil or
Displacement/Steel/Granular or
Displacement/Concrete/Cohesive

C - Displacement/Steel/Cohesive

D - No Displacement/Steel/Any Soil

Fig. 8 - Skin Friction Design Chart for Use With Method B (from Reference 2).

SOIL TYPE	LIMIT PRESSURE p_L (psf)	INSTALLATION PROCEDURE AND PILE MATERIAL						
		DRILLED	DRILLED WITH CASING		DRIVEN		INJECTED	
		CONCRETE	CONCRETE	STEEL	CONCRETE	STEEL	LOW PRESSURE	HIGH PRESSURE
Clayey, Silty or Muddy Sand	< 14600	A bis	A bis	A bis	A bis	A bis	A	–
Soft Chalk	< 14600	A bis	A bis	A bis	A bis	A bis	A	–
Soft to Stiff Clay	≤ 62700	(A)[1] A bis	(A)[1] A bis	A bis	(A)[1] A bis	A bis	A	D[2]
Silt and Compact Silt	≤ 62700	(A)[1] A bis	(A)[1] A bis	A bis	(A)[1] A bis	A bis	A	D[2]
Medium Dense Sand and Gravel	20900 to 41800	(B)[1] A	(A)[1] A bis	A bis	(B)[1] A	A	B	≥ D
Dense to Very Dense Sand and Gravel	> 52200	(C)[1] B	(B)[1] A	A	(C)[1] B	B	C	≥ D
Weathered to Fragmented Chalk	> 20900	(C)[1] B	(B)[1] A	A	(C)[1] B	B	C	≥ D
Marl and Limestone Marl	31300 to 83500	(E)[1] C	(C)[1] B	B	E[3]	E[3]	E	F
Very Compact Marl	> 94000	E	–	–	–	–	F	> F
Weathered Rock	52200 to 83500	F	F	–	F[3]	F[3]	≥ F	> F
Fragmented Rock	> 94000	F	–	–	–	–	≥ F	> F

[1]Use the letter in bracket for a careful execution of the drilled shaft with a low disturbance drilling technique or for a soil which will set up or densify around the driven pile.
[2]For soils with $p_L \geq 31328$ psf.
[3]Only if driving is possible.

FIG. 9. - Choosing the Skin Friction Design Curve for Method C (from Reference 3).

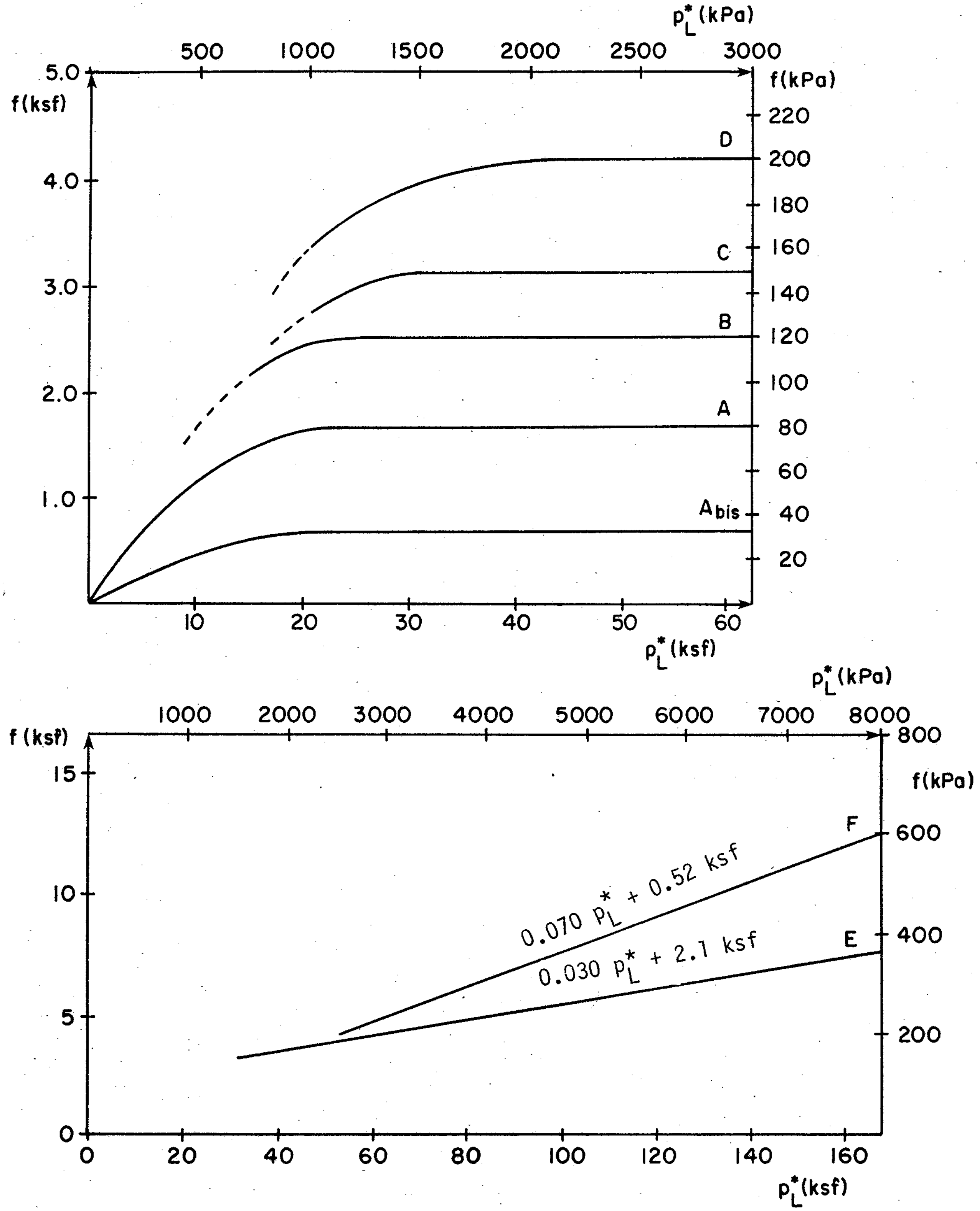


Fig. 10 - Skin Friction Design Chart For Use With Method C (from Reference 3).

CHAPTER 3 - VERTICAL SETTLEMENT

3.1 Obtaining the q-w and f-w curves

The q-w curve is the load transfer curve at the point of the pile. The parameter q is the average pressure exerted by the pile point on the soil for a movement w of the pile point. An f-w curve is a load transfer curve along the shaft of the pile. The parameter f is the friction developed between the soil and the pile for a movement w of the pile shaft. In order to determine the vertical settlement of a pile the q-w and f-w curves must first be obtained. These curves may be determined using one of three methods.

The first method is the Menard-Gambin method. It shall be referred to as Method A. In this method both q-w and f-w curves are represented by elastic-plastic models (Figure 11). The ultimate values of q and f called q_{max} and f_{max} are found by using Method A for point bearing and side friction as described in Chapter 2. The slopes $\frac{q}{w}$ and $\frac{f}{w}$ of the elastic parts of the curves are given by:

a) q-w curve

drilled shafts $R \leq 1$ ft. or $R \leq 0.30$m
$$\frac{q}{w} = \frac{2E}{\lambda R}$$

1 ft $< R \leq 2.5$ ft. or $0.30\text{m} < R \leq 0.75$m
$$\frac{q}{w} = \frac{2E_o}{(R_o)\left(\frac{\lambda R}{R_o}\right)^{\alpha}}$$

Method A (Menard Gambin)

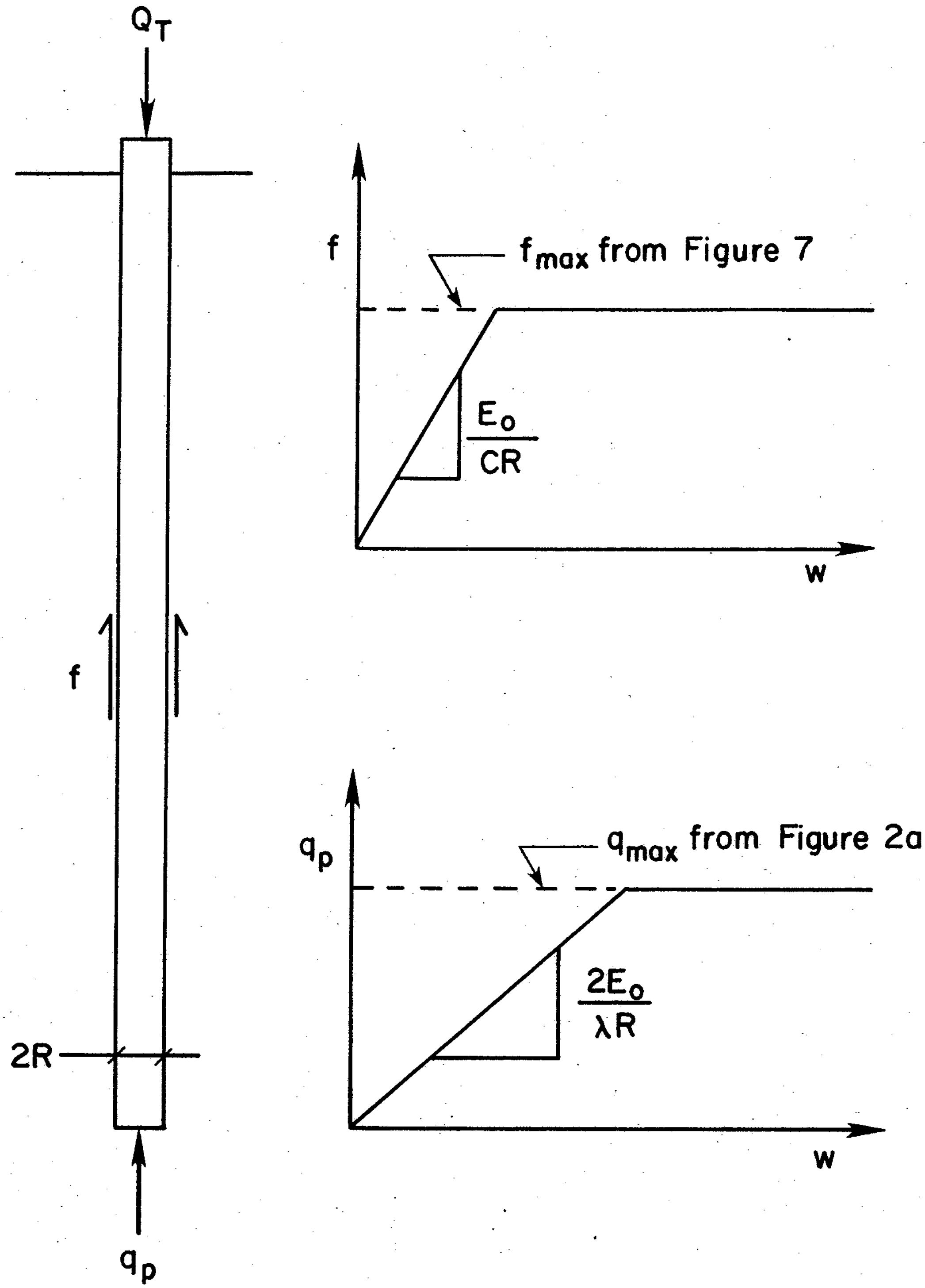


Fig. 11 - q-w and f-w Curves For Use With Method A.

where R_o = 1.0 with R in feet

or R_o = 0.30 with R in meters

driven piles $R \leq 2.5$ ft or $R \leq 0.75$ m $\quad \frac{q}{w} = \frac{2E_R}{\lambda R}$

where:

E_o is the pressuremeter first loading modulus

λ is a shape factor = 1.00 for circular

1.12 for square

1.53 for length/width = 2

2.65 for length/width = 10

R is the pile radius in feet or meters

α is a rheological coefficient (Figure 12)

E_R is the pressuremeter reload modulus

b) f-w curve

$R \leq 1.0$ ft, $R \leq 0.30$ m $\quad \frac{f}{w} = \frac{E_o}{CR}$

$R > 1.0$ ft, $R > 0.30$ m $\quad \frac{f}{w} = \frac{E_o}{C(R_o)(\frac{R}{R_o})^{\alpha}}$

where R_o = 1.0 with R in feet

R_o = 0.30 with R in meters

is given in Figure 12

C is a coefficient of strain, dependent on the ratio h/R and the method of installation of the pile (Figure 13)

Soil Type	Peat		Clay		Silt		Sand		Sand and Gravel	
	E_o/p_L^*	α	E_o/p_L^*	α	E_o/p_L^*	α	E_o/p_L^*	α	E_o/p_L^*	α
Over-consolidated			> 16	1	> 14	2/3	> 12	1/2	> 10	1/3
Normally consolidated		1	9–16	2/3	8–14	1/2	7–12	1/3	6–10	1/4
Weathered and/or remoulded			7–9	1/2		1/2		1/3		1/4

Rock	Extremely fractured	Other	Slightly fractured or extremely weathered
	α = 1/3	α = 1/2	α = 2/3

Fig. 12 – α Values (From Reference 3)

Type of Pile	Friction Pile		End Bearing Pile
	h/R = 10	h/R = 20	
Drilled Pile	4.5 - 5.0	5.2 - 5.6	2.8 - 3.2
Driven Pile	1.8 - 2.0	2.1 - 2.3	1.1 - 1.3

Fig. 13. – Coefficient of Friction Mobilization (from Reference 4).

The second method, Method B, was proposed by Baguelin, Frank, and Jezequel (6) using a selfboring pressuremeter modulus. Because the reload preboring pressuremeter modulus, E_R, correlates favorably with that selfboring pressuremeter modulus, E_R was used in the calculations. Thus Method B is not exactly Baguelin's method and was called the Pseudo-Baguelin method. An elastic-plastic model is also used for both the q-w and f-w curves (Figure 14). The ultimate values, q_{max} and f_{max}, are obtained by using Method B for point bearing and side friction as described in Chapter 2. The slopes of the elastic part of the curves are given by:

$$\frac{q}{w} = \frac{2E_R}{\pi(1-\nu^2)R}$$

$$\frac{f}{w} = \frac{E_R}{2(1+\nu)[1+\ln(\frac{L}{2R})]R}$$

E_R is the pressuremeter reload modulus

L is the pile length

R is the pile radius

The third method is the Frank-Bustamante method which will be referred to as Method C. The q-w and f-w curves (Figure 15) are modelled as bilinear elastic-plastic curves.

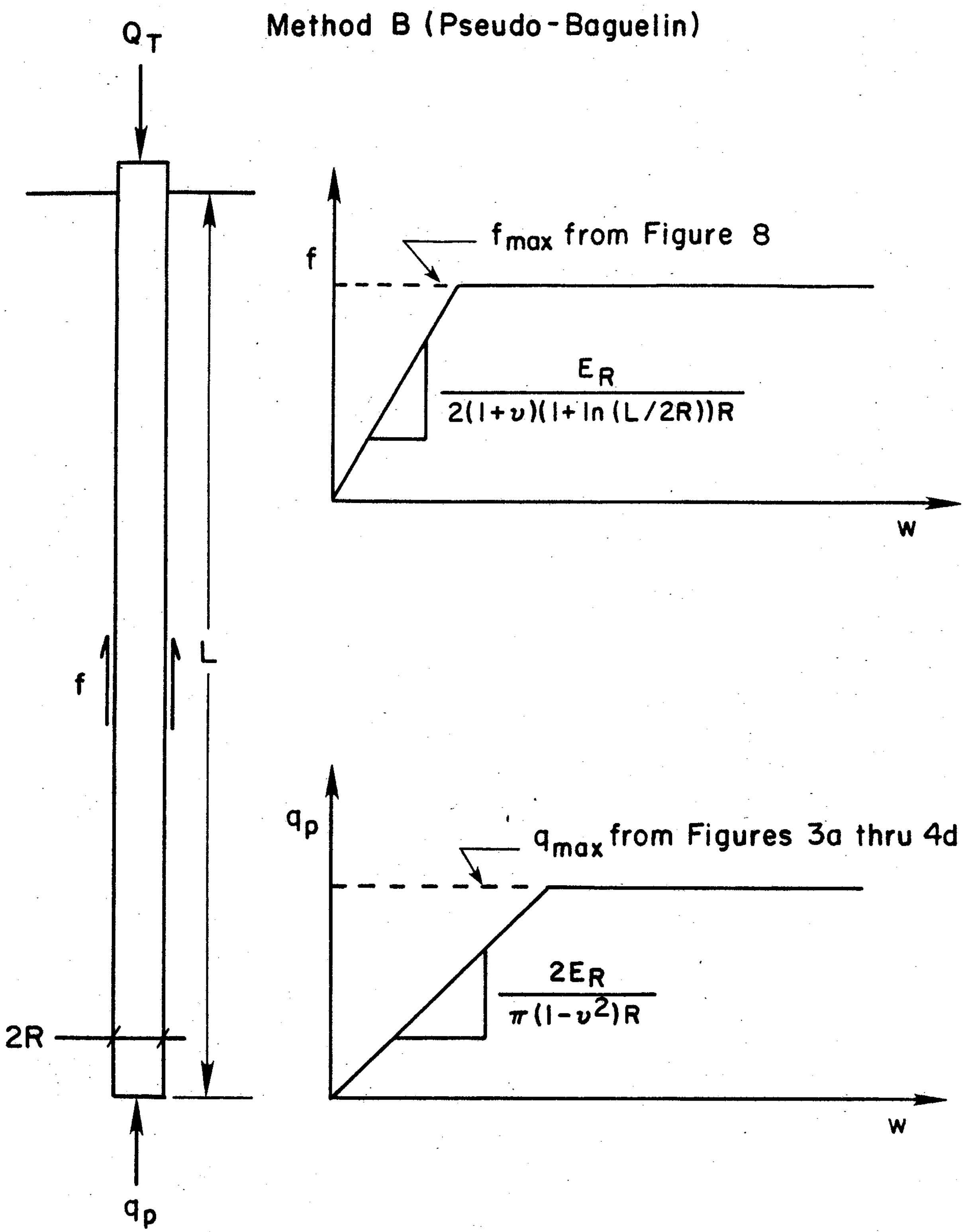


Fig. 14 - q-w and f-w Curves for Use With Method B.

Method C (Frank Bustamante)

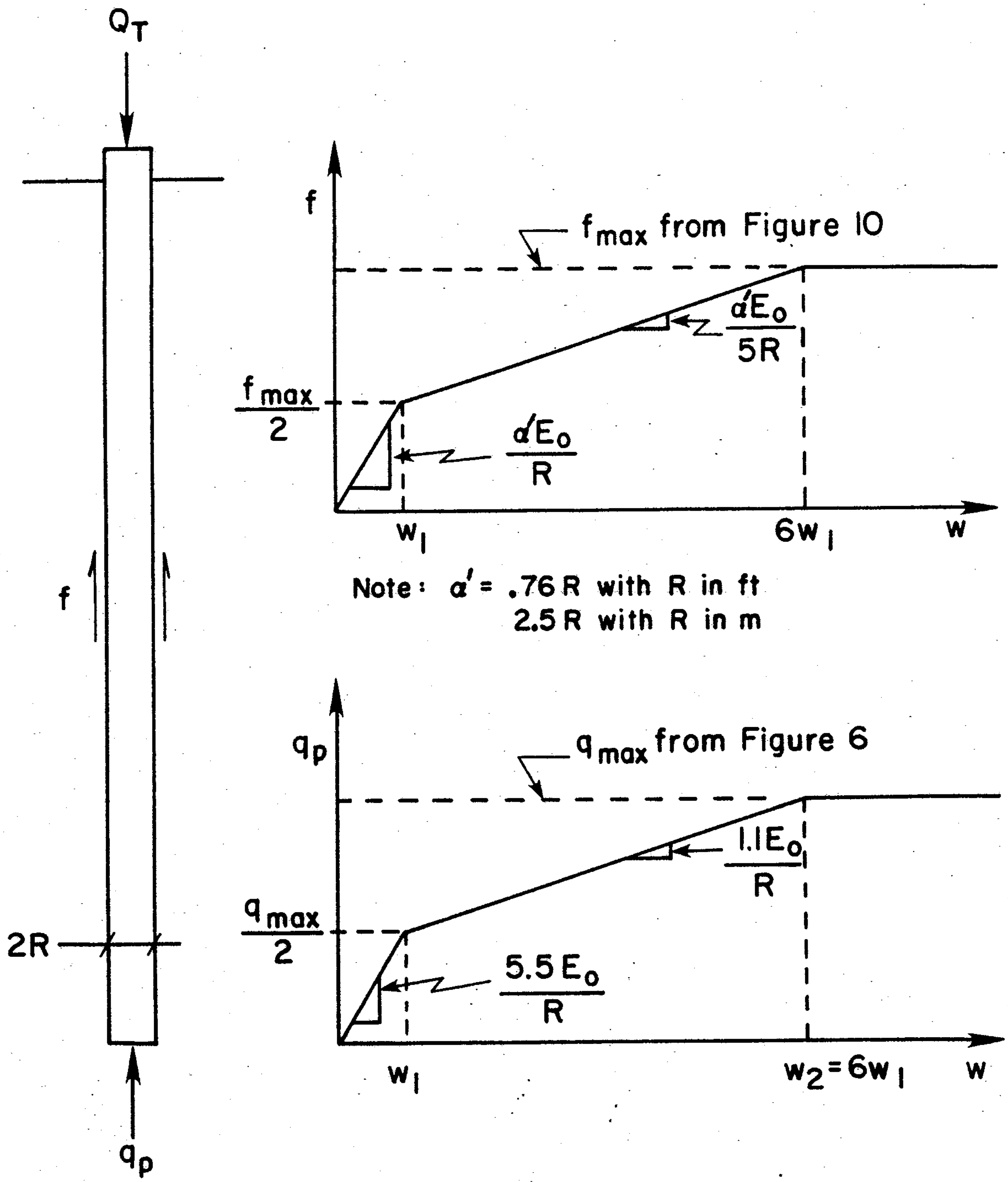


Fig. 15 - q-w and f-w Curves for Use With Method C.

The ultimate values q_{max} and f_{max} are found by using Method C for point bearing and side friction as described in Chapter 2. The first slope in the elastic region is given by:

$$\frac{q}{w} = \frac{5.5\ E_o}{R}$$

$$\frac{f}{w} = \frac{\alpha' E_o}{R} \text{ with } \alpha' = 0.76\ R \text{ with R in feet}$$

$$= 2.5\ R \text{ with R in meters}$$

E_o is the pressuremeter first loading modulus. The second slope in the elastic range is 5 times softer than the first one and the change in slope occurs at one half of the ultimate values q_{max} or f_{max}.

3.2. Obtaining the load-settlement curve

The approximate load-settlement curve is obtained point by point in the following manner:

1. Divide the pile into segments (about 10).
2. Assume a point pressure.
3. Read the corresponding point displacement w from the q-w curve.
4. Assume that the load in the pile segment closest to the point (segment n) is equal to the point load.
5. Calculate the compression of segment n under that load by:

$$\Delta w = \frac{PL}{AE}$$

6. Calculate the settlement of the top of segment n by:

$$w_n = w_{n-1} + \Delta w_n$$

7. Use the f-w curve to read the friction f_n on segment n at the displacement w_n.

8. Calculate the load in pile segment (n-1) by:

$$Q_{n-1} = f_n \Delta Z_n \pi D_n + Q_p$$

9. Do 4 through 8 up to the top segment. The load and displacement at the top of the pile provide one point on the load-settlement curve.

10. Repeat 1 through 9 for other assumed values of the point pressure.

The q-w and f-w curves can also be input into a conventional beam column program in order to obtain a more accurate load-settlement curve.

CHAPTER 4 - EXAMPLES OF DESIGN

In this chapter a series of examples have been solved to show the detailed steps of the Pressuremeter Design Method for deep foundations subjected to vertical loads.

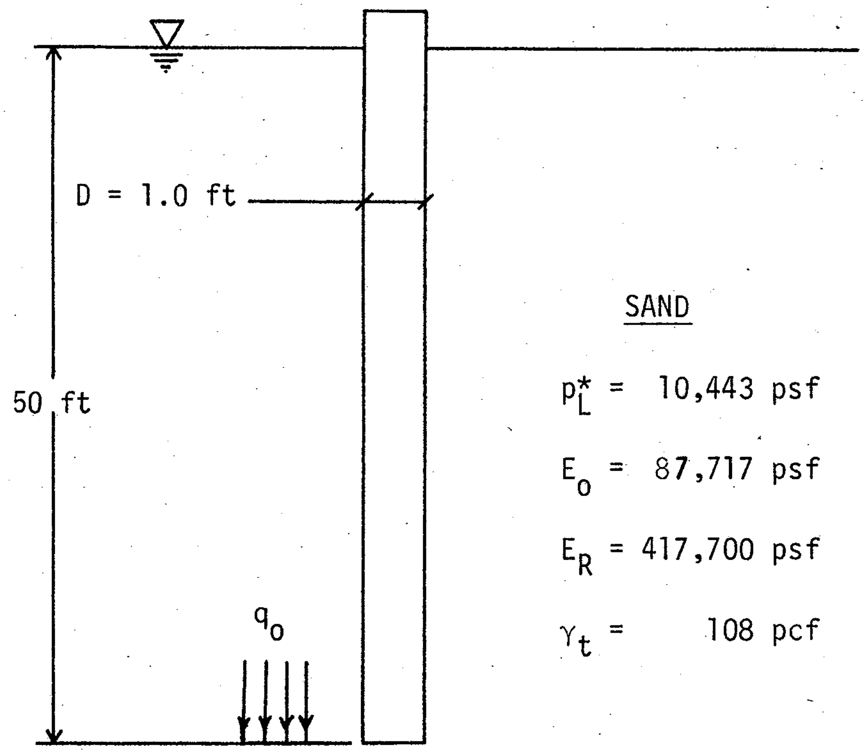
Example 4.1 - Pile in Uniform Sand: Ultimate Capacity.

Example 4.2 - Pile in Uniform Clay: Ultimate Capacity.

Example 4.3 - Pile through Loose Silt into Dense Sand: Ultimate Capacity.

Example 4.4 - Pile in Layered Clay: Ultimate Capacity and Settlement.

4.1 Pile in Uniform Sand: Ultimate Capacity



Driven, Circular, Concrete Pile

VERTICAL CAPACITY BY METHOD A

Point Bearing Capacity

Driven Pile, in Sand with $p_L^* = 10{,}443$ psf

Soil is Category II (From Fig. 1)

$$H_e/R = {}^{50}/0.5 = 100$$

$$k = 3.6 \text{ (From Fig. 2a)}$$

$$q_{max} = kp_L^* + q_o$$

$$= 3.6\ (10{,}443) + 50\ (108)$$
$$= 42{,}995 \text{ psf}$$

$$Q_p = A_p\ q_{max}$$

$$= \pi(0.5)^2\ (42{,}995)$$
$$= 33768 \text{ lb} = 16.88 \text{ tons}$$

Friction Capacity

1 ft Diameter Concrete Pile

Top 1.5 ft of Pile, $f = 0$

>3 dia. from point, $f_{max} = 1190$ psf

(From Fig. 7)

≤ 3 dia. from point, $f_{max} = 1316$ psf

$$Q_s = \sum_1^n f_{max\ i}\ \pi D \Delta Z_i$$

$$= 1190\ (\pi)(1.0)(45.5) + 1316\ (\pi)\ (1.0)(3)$$
$$= 182{,}505 \text{ lb} = 91.25 \text{ tons}$$

Total Vertical Capacity

$$Q_T = Q_p + Q_s$$

$$= 16.88 + 91.25 = 108.13 \text{ tons}$$

the recommended load at the ground surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{16.88}{3} + \frac{91.25}{2} - (150)(\pi)(.5)^2(50)/2000$$

$$= 48.31 \text{ tons}$$

VERTICAL CAPACITY BY METHOD B

Point Bearing Capacity

Driven Pile in Sand with $p_L^* = 10{,}443$ psf

$$D/B = {}^{50}/1.0 = 50$$

$$k = 3.3 \text{ (From Fig. 4c)}$$

$$q_{max} = kp_L^* + q_o$$

$$= 3.3(10{,}443) + 50(108)$$

$$= 39862 \text{ psf}$$

$$Q_p = A_p \; q_{max}$$

$$= \pi(0.5)^2(39{,}862)$$

$$= 31{,}308 \text{ lb} = 15.65 \text{ tons}$$

Friction Capacity

Concrete Displacement Pile in Sand; Use Curve A (Fig. 8)

$$f_{max} = 1608 \text{ psf (From Fig. 8)}$$

$$Q_s = \sum_1^n f_{max\ i} \; \pi D \Delta Z_i$$

$$= 1608 \; (\pi)(1.0)(48.5)$$

$$= 245{,}006 \text{ lb} = 122.50 \text{ tons}$$

Total Vertical Capacity

$$Q_T = Q_p + Q_s$$

$$= 15.65 + 122.50 = 138.15 \text{ tons}$$

the recommended load at the surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{15.65}{3} \; \frac{122.50}{2} - 2.95$$

$$= 63.52 \text{ tons}$$

VERTICAL CAPACITY BY METHOD C

Point Bearing Capacity

Driven Pile in Sand with $p_L^* = 10,443$ psf

Soil is category 1 (From Fig. 5)

$$H_e/R = {}^{50}/0.5 = 100$$

$$k = 1.5 \text{ (From Fig. 6)}$$

$$q_{max} = kp_L^* + q_o$$

$$= 1.5(10,443) + 50(108)$$

$$= 21065 \text{ psf}$$

$$Q_p = A_p \, q_{max}$$

$$= \pi(.5)^2 \; (21065)$$

$$= 16,544 \text{ lb} = 8.27 \text{ tons}$$

Friction Capacity

Use A_{bis} Curve (From Fig. 9)

$$f_{max} = 460 \text{ psf (From Fig. 10)}$$

$$Q_s = \sum_{1}^{n} f_{max\ i} \, \pi D \Delta Z_i$$

$$= 460 \; (\pi)(1.0)(48.5)$$

$$= 70,089 \text{ lb} = 35.04 \text{ tons}$$

Total Vertical Capacity

$$Q_T = Q_p + Q_s$$

$$= 8.27 + 35.04 = 43.31 \text{ tons}$$

the recommended load at the surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{8.27}{3} + \frac{35.04}{2} - 2.95 = 17.33 \text{ tons}$$

q-w and f-w CURVES BY METHOD A

1.0 ft Diameter, Driven, Circular Pile

q-w Curve

$$\frac{q}{w} = \frac{2E_R}{\lambda R}$$

$$= \frac{2(417,700)}{(1.0)(0.5)}$$

$$= 1,670,800$$

$$w = .5985 \times 10^{-6}\ q\ \text{ft}$$

$$q_{max} = 42,995\ \text{psf}$$

f-w Curve

$Q_s = 94.06$ tons $> Q_p = 16.88$ tons, therefore friction pile

$h/R = {}^{50}/0.5 = 100$

$C = 2.3$ (From Fig. 13)

$$\frac{f}{w} = \frac{E_o}{CR}$$

$$= \frac{87,717}{2.3(.5)}$$

$$= 76,276$$

$$w = 13.11 \times 10^{-6}\ f\ \text{ft}$$

$f_{max} = 1190$ psf $>$ 3 dia. from point

$f_{max} = 1316$ psf within 3 dia. of point

q-w AND f-w CURVES BY METHOD B

q-w Curve

$$\frac{q}{w} = \frac{2E_R}{\pi(1-\nu^2)R}$$

$$= \frac{2\ (417,700)}{\pi(1-.33^2)(0.5)}$$

$$= 596,827$$

$$w = 1.676 \times 10^{-6}\ q \text{ ft}$$

$$q_{max} = 39,862 \text{ psf}$$

f-w Curve

$$\frac{f}{w} = \frac{E_R}{2(1+\nu)\,[1+\ln(L/2R)]\ R}$$

$$= \frac{417,700}{2(1+.33)[1+\ln(50/1)]0.5}$$

$$= 63,937$$

$$w = 15.64 \times 10^{-6}\ f \text{ ft}$$

$$f_{max} = 1608 \text{ psf}$$

q-w AND f-w CURVES BY METHOD C

q-w Curve

$$0 \leq q \leq 1/2\ q_{max},\ \frac{q}{w} = \frac{5.5E_o}{R}$$

$$= \frac{5.5(87,717)}{0.5}$$

$$= 964,887$$

$$w = 1.036 \times 10^{-6}\ q\ \text{ft}$$

$$1/2\ q_{max} < q < q_{max}\ ,\ w = \frac{(5q - 2q_{max})R}{5.5E_o}$$

$$= \frac{(5q - 2q_{max})(0.5)}{5.5(87,717}$$

$$= (1.036 \times 10^{-6})(5q - 2q_{max})\ \text{ft}$$

$$q_{max} = 21,065\ \text{psf}$$

f-w Curve

$$0 \leq f \leq 1/2\ f_{max},\ \frac{f}{w} = \frac{\alpha' E_o}{R}$$

$$= \frac{(0.76)(0.5)(87,717)}{(0.5)}$$

$$= 66,665$$

$$w = 15.00 \times 10^{-6}\ f\ \text{ft}$$

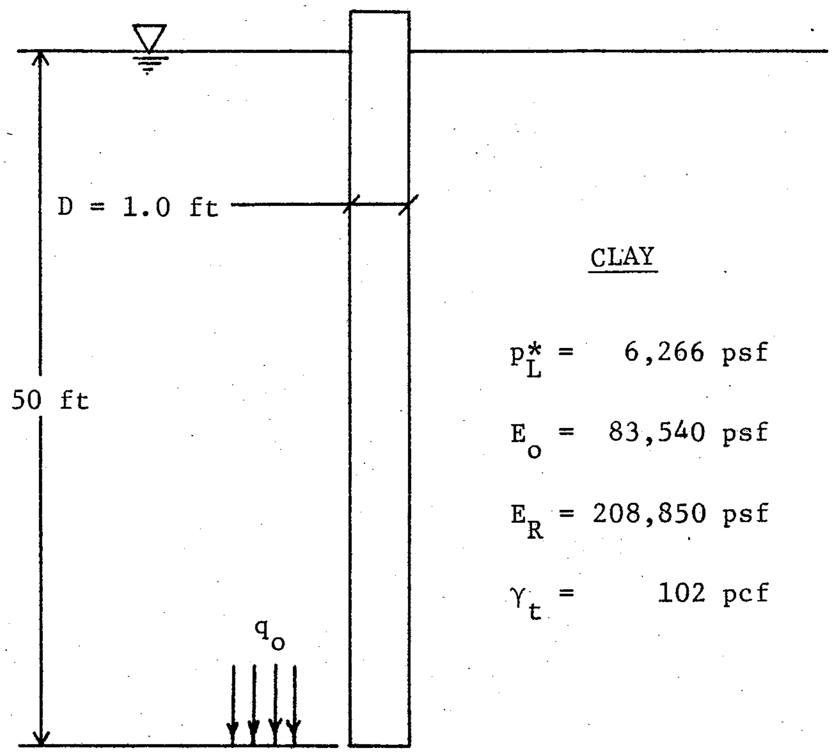
$$1/2\ f_{max} < f \leq f_{max},\ w = \frac{(5f - 2f_{max})R}{\alpha E_o}$$

$$= \frac{(5f - 2f_{max})(0.5)}{(0.76)(0.5)(87,717)}$$

$$= (15.00 \times 10^{-6})(5f - 2f_{max})\ \text{ft}$$

$$f_{max} = 460\ \text{psf}$$

4.2 Pile in Uniform Clay: Ultimate Capacity



Driven, Circular, Concrete Pile

VERTICAL CAPACITY BY METHOD A

Point Bearing Capacity

Driven Pile in Clay with $p_L^* = 6266$ psf

Soil is Category I (From Fig. 1)

$$H_e/R = 50/0.5 = 100$$

$$k = 2.0 \text{ (From Fig. 2a)}$$

$$q_{max} = kp_L^* + q_o$$

$$= 2.0(6266) + 50\ (102)$$

$$= 17632 \text{ psf}$$

$$Q_p = A_p\ q_{max}$$

$$= (0.5)^2(17632)$$

$$= 13848 \text{ lb} = 6.92 \text{ tons}$$

Friction Capacity

1 ft Diameter Concrete Pile

$>$ 3 dia. from point, $f_{max} = 877$ psf (From Fig. 7)

$\leq$ 3 dia. from point, $f_{max} = 1044$ psf

$$Q_s = \sum_1^n f_{max}\ i\ \pi D \Delta Z_i$$

$$= 877(\ )(1.0)(45.5) + 1044(\ )(1.0)(3)$$

$$= 135{,}200 \text{ lb} = 67.60 \text{ tons}$$

Total vertical capacity

$$Q_T = Q_p + Q_s$$

$$= 6.92 + 67.60$$

$$= 74.52 \text{ tons}$$

the recommended load at the ground surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{6.92}{3} + \frac{67.60}{2} - 150\ (\pi)(.5)^2(50)/2000$$

$$= 33.16 \text{ tons}$$

VERTICAL CAPACITY BY METHOD B

Point Bearing Capacity

Driven Pile in Clay with $p_L^* = 6266$ psf

$$D/B = 50/1.0 = 50$$

$$k = 2.1 \text{ (From Fig. 4a)}$$

$$q_{max} = kp_L^* + q_o$$

$$= 2.1(6266) + 50(102)$$

$$= 18259 \text{ psf}$$

$$Q_p = A_p\, q_{max}$$

$$= \pi(0.5)^2(18259)$$

$$= 14341 \text{ lb} = 7.17 \text{ tons}$$

Friction Capacity

Concrete, Displacement Pile in Cohesive Soil; Use Curve B (Fig. 8)

$$f_{max} = 835 \text{ psf (From Fig. 8)}$$

$$Q_s = \sum_{1}^{n} f_{max\, i}\, \pi D \Delta Z_i$$

$$= 835(\pi)(1.0)(48.5)$$

$$= 127{,}227 \text{ lb} = 63.61 \text{ tons}$$

Total Vertical Capacity

$$Q_T = Q_p + Q_s$$

$$= 7.17 + 63.61$$

$$= 70.78 \text{ tons}$$

the recommended load at the surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{7.17}{3} + \frac{63.61}{2} - 2.95$$

$$= 31.25 \text{ tons}$$

VERTICAL CAPACITY BY METHOD C

Point Bearing Capacity

Driven Pile in Clay with $p_L^* = 6266$ psf

Soil is Category 2 (From Fig. 5)

$$H_e/R = 50/0.5 = 100$$

$$k = 2.7 \text{ (From Fig. 6)}$$

$$q_{max} = kp_L^* + q_o$$
$$= 2.7(6266) + (50)(102)$$
$$= 22018 \text{ psf}$$

$$Q_p = A_p\, q_{max}$$
$$= \pi(0.5)^2(22018)$$
$$= 17293 \text{ lb} = 8.65 \text{ tons}$$

Friction Capacity

High Value (Use Curve A; from Fig. 9)

$$f_{max} = 835 \text{ psf (From Fig. 10)}$$

$$Q_s = \sum_1^n f_{max\ i}\ \pi D \Delta Z_i$$
$$= 835\ (\pi)(1.0)(48.5)$$
$$= 127{,}227 \text{ lb} = 63.61 \text{ tons}$$

Low Value (Use Curve A_{bis}; From Fig. 9)

$$f_{max} = 292 \text{ psf (From Fig. 10)}$$

$$Q_s = 292\ (\pi)(1.0)(48.5)$$
$$= 44{,}491 \text{ lb} = 22.25 \text{ tons}$$

Total Vertical Capacity

$$Q_T = Q_p + Q_s$$

High $Q_T = 8.65 + 63.61 = 72.26$ tons

Low $Q_T = 8.65 + 22.25 = 30.90$ tons

the recommended load at the surface is

$$Q = Q_p/3 + Q_s/2 - W$$

High $Q = \dfrac{8.65}{3} + \dfrac{63.61}{2} - 2.95 = 31.74$ tons

Low $Q = \dfrac{8.65}{3} + \dfrac{22.25}{2} - 2.95 = 11.06$ tons

q-w AND f-w CURVES BY METHOD A

1.0 ft Diameter, Driven, Circular Pile

q-w Curve

$$\frac{q}{w} = \frac{2E_R}{\lambda R}$$

$$= \frac{2(208,850)}{1.0(0.5)}$$

$$= 835,400$$

$$w = 1.197 \times 10^{-6}\ q\ \text{ft}$$

$$q_{max} = 17632\ \text{psf}$$

f-w Curve

Q_s = 69.67 tons > Q_p = 6.92 tons, Therefore Friction Pile

$h/R = 50/0.5 = 100$

$C = 2.3$ (From Fig. 13)

$$\frac{f}{w} = \frac{E_o}{CR}$$

$$= \frac{83540}{(2.3)(0.5)}$$

$$= 72643$$

$$w = 13.76 \times 10^{-6}\ f\ \text{ft}$$

$\leq$ 3 dia. from Point, f_{max} = 877 psf

Within 3 dia of point, f_{max} = 1044 psf

q-w AND f-w CURVES BY METHOD B

q-w Curve

$$= \frac{2E_R}{\pi(1-\nu^2)R}$$

$$= \frac{2(208,850)}{\pi(1-.33^2)(0.5)}$$

$$= 298,413$$

$$w = 3.351 \times 10^{-6}\ q\ \text{ft}$$

$$q_{max} = 18259\ \text{psf}$$

f-w Curve

$$= \frac{E_R}{2(1+\nu)(1+\ln(L/_{2R})R}$$

$$= \frac{208,850}{2(1+.33)(1+\ln(^{50}/1))05}$$

$$= 31969$$

$$w = 31.281 \times 10^{-6}\ f\ \text{ft}$$

$$q_{max} = 835\ \text{psf}$$

q-w AND f-w CURVES BY METHOD C

q-w Curve

$$o \le q \le 1/2\ q_{max},\quad \frac{q}{w} = \frac{5.5\ E_o}{R}$$

$$= \frac{5.5(83,540)}{0.5}$$

$$= 918,940$$

$$= 1.088 \times 10^{-6}\ q\ \text{ft}$$

$$1/2\ q_{max} < q \le q_{max},\quad w = \frac{(5q - 2q_{max})R}{5.5\ E_o}$$

$$= \frac{(5q - 2q_{max})(0.5)}{5,5(83,540)}$$

$$= (1.088 \times 10^{-6})(5q - 2q_{max})\text{ft}$$

q_{max} = 22,018 psf

f-w Curve

$$o \le f \le 1/2\ f_{max},\quad \frac{f}{w} = \frac{\alpha E_o}{R}$$

$$= \frac{(0.76)(0.5)(83,540)}{0.5}$$

$$= 63,490$$

$$= 15.750 \times 10^{-6}\ f\ \text{ft}$$

$$1/2\ f_{max} < f \le f_{max},\quad w = \frac{(5f - 2f_{max})R}{\alpha E_o}$$

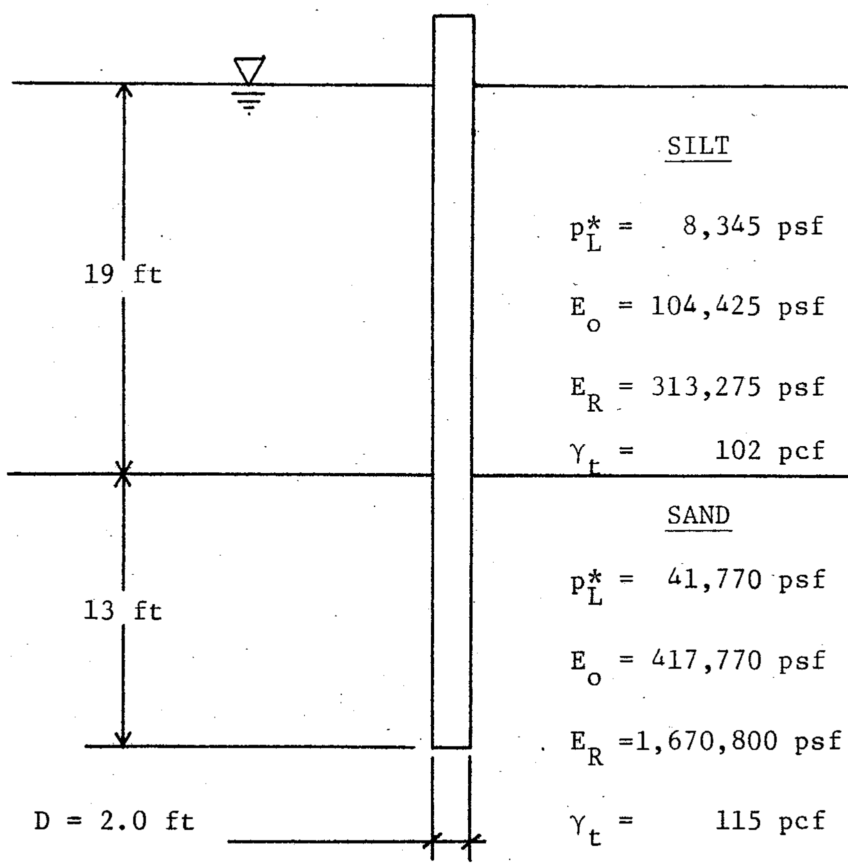
$$= \frac{(5f - 2f_{max})(0.5)}{(0.76)(0.5)(83540)}$$

$$= (15.750 \times 10^{-6})(5f - 2f_{max})\ \text{ft}$$

High Value, f_{max} = 835 psf

Low Value, f_{max} = 292 psf

4.3 Pile Through Loose Silt, Into Dense Sand: Ultimate Capacity



Drilled, Circular, Concrete Pile

VERTICAL CAPACITY BY METHOD A

Point Bearing Capacity

$$H_e = \sum_{1}^{n} \Delta Z_i \frac{p^*_{Li}}{p^*_{Le}}$$

$$= \frac{(8345 \times 19) + (41770 \times 13)}{41770}$$

$$= 16.8 \text{ ft}$$

$$H_e/R = 16.8/1.0 = 16.8$$

Point in Sand with p^*_L = 41770 psf

Soil is Category III (From Fig. 1)

$$k = 5.25 \text{ (From Fig. 2a)}$$

$$q_{max} = kp^*_L + q_o$$

$$= 5.25(41770) + (19 \times 102 + 13 \times 115)$$
$$= 222{,}726 \text{ psf}$$

$$Q_p = 222{,}726(\pi)(1.0)^2$$
$$= 699{,}713 \text{ lb} = 350 \text{ tons}$$

Friction Capacity

2 ft Diameter, Concrete Pile

Silt Layer f_{max} = 1044 psf

Sand Layer, > 3 dia. from point f_{max} = 1713 psf

Sand Layer, Within 3 dia. of point f_{max} = 2527 psf

$$Q_s = \sum_{1}^{n} f_{max\ i}\ \pi D \Delta Z_i$$

$$= (1044)(\pi)(2)(17) + (1713)(\pi)(2)(7) + (2527)(\pi)(2)(6)$$
$$= 282{,}121 \text{ lb} = 141 \text{ tons}$$

Total Vertical Capacity

$$Q_T = Q_p + Q_s$$

$$= 350 + 141 = 491 \text{ tons}$$

the recommended load at the ground surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{350}{3} + \frac{141}{2} - (150)(\pi)(1.0)^2(32)/2000$$

$$= 180 \text{ tons}$$

VERTICAL CAPACITY BY METHOD B

Point Bearing Capacity

$$D = \sum_{1}^{n} \Delta Z_i \frac{p^*_{Li}}{p^*_{Le}}$$

$$= \frac{(8345 \times 19) + (41770 \times 13)}{41770}$$

$$= 16.8 \text{ ft}$$

$$D/B = 16.8/2.0 = 8.4$$

Bored Pile, Point in Sand with $p^*_L = 41770$ psf

$$k = 5.7 \text{ (From Fig. 3a)}$$

$$q_{max} = kp^*_L + q_o$$

$$= 5.7(41770) + (19 \times 102 + 13 \times 115)$$
$$= 241{,}522 \text{ psf}$$

$$Q_p = 241522\ (\pi)(1.0)^2$$
$$= 758{,}764 \text{ lb} = 379 \text{ tons}$$

Friction Capacity

Concrete, Non-Displacement Pile in Sand; Use Curve B (Fig. 8)

Silt Layer $f_{max} = 1044$ psf

Sand Layer $f_{max} = 1713$ psf

$$Q_s = \sum_{1}^{n} f_{max\ i}\ \pi D \Delta Z_i$$
$$= (1044)(\pi)(2)(17) + (1713)(\pi)(2)(13)$$
$$= 251{,}434 \text{ lb} = 126 \text{ tons}$$

Total Vertical Capacity

$$Q_T = Q_p + Q_s$$

$$= 379 + 126 = 505 \text{ tons}$$

the recommended load at the ground surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$
$$= \frac{379}{3} + \frac{126}{2} - (150)(\pi)(1.0)^2(32)/2000$$

$$= 182 \text{ tons}$$

VERTICAL CAPACITY BY METHOD C

Point Bearing Capacity

$$H_e = \sum_{1}^{n} \Delta Z_i \frac{p^*_{Li}}{p^*_{Le}}$$

$$= \frac{(8354 \times 19) + (41770 \times 13)}{41770}$$

$$= 16.8 \text{ ft}$$

$$H_e/R = 16.8/1.0 = 16.8$$

Point in Sand With $p^*_L = 41770$

Soil is Category 2 (From Fig. 5)

$$k = 1.6 \text{ (From Fig. 6)}$$

$$q_{max} = kp^*_L + q_o$$

$$= 1.6(41770) + (19x102 + 13x115)$$

$$= 70,265 \text{ psf}$$

$$Q_p = 70265(\pi)(1.0)^2$$
$$= 221875 \text{ lb} = 111 \text{ tons}$$

Friction Capacity and Total Vertical Capacity

1. Low Value

Silt Layer (A_{bis}) (From Fig. 9) $f_{max} = 418$ psf (From Fig 10)

Sand Layer (A) $f_{max} = 1617$ psf

$$Q_s = \sum_{1}^{n} f_{max\ i}\ \pi D \Delta Z_i$$

$$= (418)(\pi)(2.0)(17) + (1671)(\pi)(2.0)(13)$$

$$= 181,138 \text{ lb} = 91 \text{ tons}$$

$$Q_T = Q_p + Q_s$$

$$= 111 + 91 = 202 \text{ tons}$$

the recommended load at the ground surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{111}{3} + \frac{91}{2} - (150)(\pi)(1.0)^2(32)/2000$$

$$= 75 \text{ tons}$$

2. High Value

Silt Layer (A) (From Fig. 9) f_{max} = 1044 psf (From Fig. 10)

Sand Layer (B) f_{max} = 2506 psf

Q_s = (1044)(π)(2.0)(17) + (2506)(π)(2.0)(13)

= 316,208 lb = 158 tons

Q_T = 111 + 158 = 269 tons

the recommended load at the ground surface is

$$Q = \frac{111}{3} + \frac{158}{2} - 7.54$$

= 108 tons

q-w AND f-w CURVES BY METHOD A

2 ft Diameter, Circular Pile with Drilled Shaft

q-w Curve

$$\frac{q}{w} = \frac{2\,E_o}{\lambda R}$$

$$= \frac{2(417,700)}{(1.0)(1.0)}$$

$$= 835,400$$

$$w = 1.197 \times 10^{-6}\, q \text{ ft}$$

$$q_{max} = 222,726 \text{ psf}$$

f-w Curves

Q_s = 141 tons < Q_p = 350 tons, Therefore End Bearing

C = 3.0 (From Fig. 13)

$$\frac{f}{w} = \frac{E_o}{CR}$$

Silt, $\frac{f}{w} = \frac{104,425}{(3.0)(1.0)}$

$$= 34,808$$

$$w = 28.729 \times 10^{-6}\, f \text{ ft}$$

$$f_{max} = 1044 \text{ psf}$$

Sand, $\frac{f}{w} = \frac{417,700}{(3.0)(1.0)}$

$$= 139,233$$

$$w = 7.182 \times 10^{-6}\, f \text{ ft}$$

f_{max} = 1713 psf (>3 dia. from point)

f_{max} = 2527 psf (within 3 dia. of point)

q-w AND f-w CURVES BY METHOD B

q-w Curve

$$\frac{q}{w} = \frac{2E_R}{\pi(1-\nu^2)R}$$

$$= \frac{2(1,670,800)}{\pi(1-.33^2)1.0}$$

$$= 1,193,653$$

$$w = 0.838 \times 10^{-6}\ q \text{ ft}$$

$$q_{max} = 241,522 \text{ psf}$$

f-w Curve

$$\frac{f}{w} = \frac{E_R}{2(1+\nu)[1+\ln(^L/2R)]R}$$

Silt, $$= \frac{313,275}{2(1+.33)[1+\ln(^{32}/2)]1.0}$$

$$= 31,218$$

$$w = 32.033 \times 10^{-6}\ f \text{ ft}$$

$$f_{max} = 1044$$

Sand, $$\frac{f}{w} = \frac{1,670,800}{2(1+.33)[1+\ln(^{32}/2)]1.0}$$

$$= 166,496$$

$$w = 6.006 \times 10^{-6}\ f \text{ ft}$$

$$f_{max} = 1713 \text{ psf}$$

q-w AND f-w CURVES BY METHOD C

q-w Curve

$$0 \le q \le 1/2\ q_{max},\quad \frac{q}{w} = \frac{5.5E_o}{R}$$

$$= \frac{5.5(417.700)}{1.0}$$

$$= 2,297,350$$

$$w = 0.435 \times 10^{-6}\ q\ \text{ft}$$

$$1/2\ q_{max} < q \le q_{max},\quad w = \frac{(5q - 2q_{max})R}{5.5\ E_o}$$

$$= \frac{(5q - 2q_{max})(1.0)}{5.5(417,700)}$$

$$= (0.435 \times 10^{-6})(5q - 2q_{max})\ \text{ft}$$

$$q_{max} = 70,265\ \text{psf}$$

f-w Curves

$$0 \le f \le 1/2\ f_{max},\quad \frac{f}{w} = \frac{\alpha' E_o}{R}$$

$$\text{Silt},\quad \frac{f}{w} = \frac{(0.76)(104,425)}{1.0}$$

$$= 79,363$$

$$w = 12.600 \times 10^{-6}\ f\ \text{ft}$$

$$\text{Sand},\quad \frac{f}{w} = \frac{(0.76)(417,700)}{1.0}$$

$$= 317,452$$

$$w = 3.150 \times 10^{-6}\ f\ \text{ft}$$

$$1/2\ f_{max} < f \le f_{max},\quad w = \frac{(5f - 2f_{max})R}{\alpha E_o}$$

$$\text{Silt},\quad w = \frac{(5f - 2f_{max})(1.0)}{(0.76)(1.0)(104,425)}$$

$$= (12.600 \times 10^{-6})(5f - 2f_{max})\ \text{ft}$$

$$\text{Sand, } w = \frac{(5f - 2f_{max})(1.0)}{(0.76)(1.0)(417{,}700)}$$

$$= (3.150 \times 10^{-6})(5f - 2f_{max})\,\text{ft}$$

Low Values

Silt, f_{max} = 418 psf

Sand, f_{max} = 1671 psf

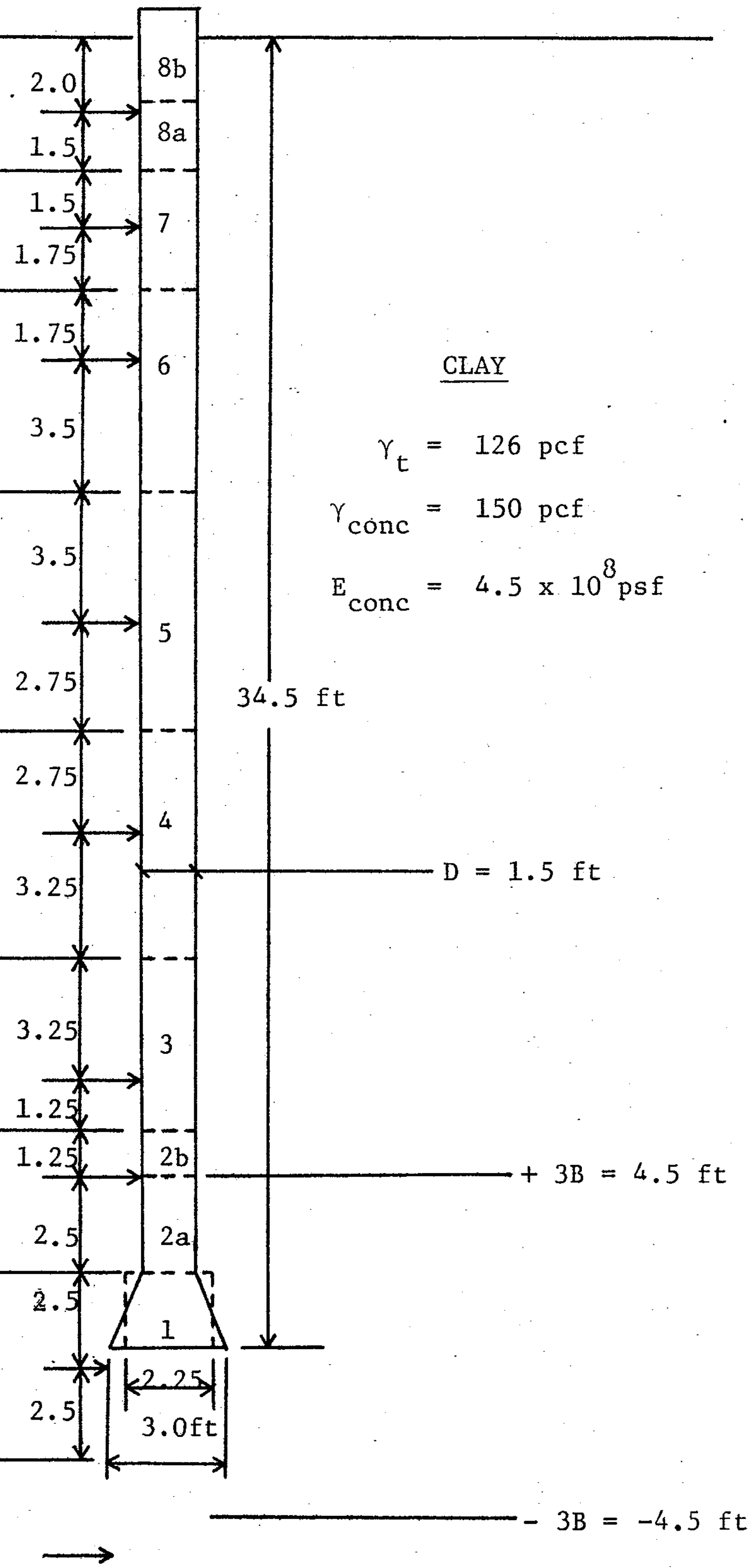
High Values

Silt, f_{max} = 1044 psf

Sand, f_{max} = 2506 psf

4.4 Pile in Layered Clay: Ultimate Capacity and Settlement

p^*_L (psf)	E_o (psf)	E_R (psf)
8354*	125310*	710090*
7205	68921	148284
11487	83540	227647
12740	127399	524214
361311	407258	998303
33416	236001	1142410
50124	346691	1743898
62655	891790	2088500
71001		



* The large number of significant figures in all the pressuremeter parameters is not meant to imply a great precision for that test; it comes from a conversion of those parameters from SI units to english units.

VERTICAL CAPACITY BY METHOD A

Point Bearing Capacity

$$p_{Le} = \sqrt[n]{p^*_{L1} \times p^*_{L2} \times \ldots \times p^*_{Ln}}$$

$$= \sqrt[3]{50{,}124 \times 62{,}655 \times 71{,}001}$$

$$= 60{,}639 \text{ psf}$$

$$H_e = \sum_{1}^{n} \Delta Z_i \frac{p^*_{Li}}{p^*_{Le}}$$

[(3.5 x 8354)+(3.25)x7705)+5.25x11487)+(6.25x12740)+

(6.00x36131)+(3.75x50124)+(20x62655)] /(60639)

$= 14.40$ ft

$H_e/R = 14.40/1.5$

$= 9.60$

Point in clayey shale with $p^*_L = 60{,}339$ psf

Soil is Category II (From Fig. 1)

$k = 3.1$ (From Fig. 2a)

$q_{max} = kp^*_L + q_o$

$= 3.2(60{,}639)+(126 \times 34.5)$

$= 198{,}392$ psf

$Q_p = 198{,}392(\pi)(1.5)^2$

$= 1{,}402{,}349$ lb $= 701$ tons

Friction Capacity

1.5 ft Diameter, Concrete Pile

Depth (ft)	p_ℓ^*	f_{max}(psf)
0.00 - 1.75	8,354	0
1.75 - 3.50	8,354	1044
3.50 - 6.75	7,205	961
6.75 - 12.00	11,487	1253
12.00 - 18.25	12,740	1316
18.25 - 24.25	361,311	1713
24.25 - 28.75	33,416	1713
28.75 - 30.00	50,124	1713
30.00 - 32.50	50,124	2548
32.50 - 34.50	62,655	2548

$$Q_s = \pi(1.5)[(1044x1.75)+(961x3.21)+(1253x5.25)+(1316x6.25)+$$

$$1713x11.75)+(2548x4.5)]$$

$$= 241{,}787 \text{ lb} = 120.9 \text{ tons}$$

<u>Total Vertical Capacity</u>

$$Q_T = Q_p + Q_s$$

$$= 701 + 121 = 822 \text{ tons}$$

the recommended load at the ground surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{701}{3} + \frac{121}{2} - 150(\pi)(0.75^2)(34.5)/2000$$

$$= 290 \text{ tons}$$

VERTICAL CAPACITY BY METHOD B

Point Bearing Capacity

p^*_{Le} = 60,639 psf (see Method A)

$D = H_e$ = 14.40 ft (see Method A)

Bored Pile with Point in Clayey Shale with p^*_L = 60,639 psf

$k = 3.1$ (from Fig. 3c)

$$q_{max} = kp^*_L + q_o$$

$$= 3.1(60,639) + (126 \times 34.5)$$

$$= 192,328 \text{ psf}$$

$$Q_p = 192,328(\pi)(1.5)^2$$

$$= 1,359,487 \text{ lb} = 680 \text{ tons}$$

Friction Capacity

Concrete, Non-Displacement Piles; use Curve B (Fig. 8)

Depth (ft)	p^*_L	f_{max}
0.00 - 1.75	8354	0
1.75 - 3.50	8354	1044
3.50 - 6.75	7205	940
6.75 - 12.00	11487	1253
12.00 - 18.25	12740	1316
18.25 - 24.25	361311	1713
24.25 - 28.75	33416	1713
28.75 - 32.50	50124	1713
32.50 - 34.50	62655	1713

$$Q_s = \pi(1.5)[(1044 \times 1.75)+(940 \times 3.25)+(1253 \times 5.25)+$$
$$(1316 \times 6.25(+1713 \times 16.25)]$$
$$= 223,940 \text{ lb} = 112 \text{ tons}$$

Total Vertical Capacity

$$Q_T = Q_p + Q_s$$

$$= 680 + 112 = 792 \text{ tons}$$

The recommended load at the ground surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{680}{3} + \frac{112}{2} - 4.57$$

$$= 278 \text{ tons}$$

VERTICAL CAPACITY BY METHOD C

Point Bearing Capacity

p^*_{Le} = 60,639 psf (see Method A)

H_e = 14.40 ft (see Method A)

Bored pile, point in clay with p^*_L = 60,639 psf.

Soil is category 2 (from Fig. 5)

k = 1.6 (from Fig. 6)

$q_{max} = kp^*_L + q_o$

$= 1.6(60,369) + (126 \times 34.5)$

$= 100,937$ psf

$Q_p = 100,937\ (\pi)(1.5)^2$

$= 713,482$ lb $= 357$ tons

Friction Capacity & Total Vertical Capacity

1. Low value is A_{bis} (from Fig. 9).
2. High value is A (from Fig. 9).

Depth (ft)	p^*_L	f_{max} Low	f_{max} High
0.00 - 1.75	8,354	0	0
1.75 - 3.50	8,354	397	1023
3.50 - 6.75	7,205	355	940
6.75 - 12.00	11,478	480	1253
12.00 - 18.25	12,740	522	1316
18.25 - 24.25	361,311	668	1671
24.25 - 28.75	33,416	668	1671
28.75 - 32.50	50,124	668	1671
32.50 - 34.50	62,655	668	1671

Low Value

$Q_s = \pi(1.5)[(397 \times 1.75)+(355 \times 3.25)+(480 \times 5.25)+(522 \times 6.25)+$

$668 \times 16.25)] = 87,113$ lb $= 44$ tons

$Q_T = Q_p + Q_s$

$= 357+44 = 401$ tons

The recommended load at the ground surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{357}{3} + \frac{44}{2} - 4.57$$

$$= 136 \text{ tons}$$

<u>High Value</u>

$$Q_s = \pi(1.5)[(1023x1.75)+(940x3.25)+(1253x5.25)+(1316x6.25)+$$

$$(1671x16.25)]$$

$$= 220{,}550 \text{ lb} = 110 \text{ tons}$$

$$Q_T = Q_p + Q_s$$

$$= 357 + 110 = 467 \text{ tons}$$

The recommended load at the ground surface is

$$Q = \frac{Q_p}{3} + \frac{Q_s}{2} - W$$

$$= \frac{357}{3} + \frac{110}{2} - 4.57$$

$$= 169 \text{ tons}$$

q-w AND f-w CURVES BY METHOD A

1.5 ft diameter, Circular Pile, with drilled shaft in clayey shale

q-w Curve

$$E_o/p^* = 891,700/62,655$$

$$= 14.2$$

$$\alpha = 2.3 \text{ (from Fig. 12)}$$

$$\lambda = 1.0$$

$$\frac{q}{w} = \frac{2E_o}{R_o(\lambda R/R_o)^\alpha}$$

$$= \frac{2(891,700)}{1.0(1.5/1.0)^{2/3}}$$

$$= 1,360,989$$

$$w = 0.735 \times 10^{-6}\ q \text{ ft}$$

$$q_{max} = 198,392 \text{ psf}$$

f-w Curve

Q_s = 121 tons < Q_p = 701 tons, THEREFORE ENDBEARING

C = 3.0 (from Fig. 13)

$$\frac{f}{w} = \frac{E_o}{CR}$$

Segment	Depth (ft)	E_o (psf)	w (ft)	f_{max}(psf)	Δ L (ft)
8b	0.00 - 1.75	125,310	$(17.96\times10^{-6})f$	0	1.75
8a	1.75 - 3.50	125,310	$(17.96\times10^{-6})f$	1044	1.75
7	3.50 - 6.75	69,921	$(32.65\times10^{-6})f$	961	3.25
6	6.75 - 12.00	83,540	$(26.93\times10^{-6})f$	1253	5.25
5	12.00 - 18.25	127,399	$(17.66\times10^{-6})f$	1316	6.25
4	18.25 - 24.25	407,258	$(5.52\times10^{-6})f$	1713	6.00
3	24.25 - 28.75	236,001	$(9.53\times10^{-6})f$	1713	4.50
2b	28.75 - 30.00	346,691	$(6.49\times10^{-6})f$	1713	1.25
2a	30.00 - 32.50	346,691	$(6.49\times10^{-6})f$	2548	2.50

THE BELL IS IN THE REGION OF 32.50 - 34.50. IT WILL BE ASSUMED TO ACT LIKE A 2.25 FT DIAMETER CYLINDER.

$$\frac{f}{w} = \frac{E_o}{C(R_o)(R/R_o)^{\alpha}}$$

$$= \frac{891790}{(3.0)(1.0)(1.125/1.0)^{2/3}}$$

$$= 274{,}815$$

$$w = 3.64 \times 10^{-6} f \text{ ft}$$

$$f_{max} = 2548 \text{ psf}$$

VERTICAL LOAD-SETTLEMENT CURVE BY METHOD A

Assuming A Point Bearing Pressure $q_1 = 20,000$ psf

Then $P_1 = \pi(1.5)^2(20,000) = 141,372$ lb

W_1 = Point Settlement

$= 0.735 \times 10^{-6}\ (20,000) = 1.47 \times 10^{-2}$ ft

W_{2a} = Settlement of top of pile segment 1

$$= W_1 + \frac{P_1 \Delta L_1}{A\ E_{conc}} = 1.47 \times 10^{-2} + \frac{141372}{\pi(1.125)^2} \times \frac{2.0}{4.5 \times 10^8} = 1.49 \times 10^{-2} \text{ ft}$$

$$f_1 = \frac{W_1}{3.64 \times 10^{-6}} = \frac{1.47 \times 10^{-2}}{3.64 \times 10^{-6}} = 4038 \text{ psf; use } f_{max} = 2548 \text{ psf}$$

$$\Delta\sigma_1 = \frac{2\pi R_1 \Delta L_1 f_{max}}{\pi R_2^2} = \frac{2(1.125)(2.0)(2548)}{(0.75)^2} = 20,384 \text{ psf}$$

$$\sigma_{2a} = \sigma_1 + \Delta\sigma_1 = \frac{141372}{(.75)^2} + 20,384 = 100,384 \text{ psf}$$

W_{2b} = Settlement of top of pile segment 2a

$$= W_{2a} + \frac{\sigma_{2a} + \Delta L_{2a}}{E_{conc}} = 1.49 \times 10^{-2} + \frac{(100,384)(2.5)}{4.5 \times 10^8} = 1.55 \times 10^{-2} \text{ ft}$$

$$f_{2a} = \frac{W_{2a}}{6.49 \times 10^{-6}} = \frac{1.49 \times 10^{-2}}{6.49 \times 10^{-6}} = 2296 \text{ psf}$$

$$\Delta\sigma_{2a} = \frac{2\Delta L_{2a} f_{2a}}{R} = \frac{2(2.5)(2296)}{0.75} = 15307 \text{ psf}$$

$$\sigma_{2b} = \sigma_{2a} + \Delta\sigma_{2a} = 100,384 + 15307 = 115,691 \text{ psf}$$

W_3 = Settlement of top of pile segment 2b

$$= W_{2b} + \frac{\sigma_{2b} \Delta L_{2b}}{E_{conc}} = 1.55 \times 10^{-2} + \frac{(115,691)(1.25)}{4.5 \times 10^8} = 1.58 \times 10^{-2} \text{ ft}$$

$$f_{2b} = \frac{W_{2b}}{6.49 \times 10^{-6}} = \frac{1.55 \times 10^{-2}}{6.49 \times 10^{-6}} = 2388 \text{ psf; use } f_{max} = 1713 \text{ psf}$$

$$\Delta\sigma_{2b} = \frac{2\Delta L_{2b} f_{max}}{R} = \frac{2(1.25)(1713)}{0.75} = 5710 \text{ psf}$$

$$\sigma_3 = \sigma_{2b} + \Delta\sigma_{2b} = 115,691 + 5710 + 121,401 \text{ psf}$$

W_4 = Settlement of top of pile segment 3

$$= W_3 + \frac{\sigma_3 \Delta L_3}{E_{conc}} = 1.58 \times 10^{-2} + \frac{(121{,}401)(4.50)}{4.5 \times 10^{8}} = 1.70 \times 10^{-2} \text{ ft}$$

$$f_3 = \frac{W_3}{9.53 \times 10^{-6}} = \frac{1.58 \times 10^{-2}}{9.53 \times 10^{-6}} = 1658 \text{ psf}$$

$$\Delta\sigma_3 = \frac{2\Delta L_3 f_3}{R} = \frac{2(4.5)(1658)}{(.75)} = 19896 \text{ psf}$$

$$\sigma_4 = \sigma_3 + \Delta\sigma_3 = 121{,}401 + 19896 = 141{,}297 \text{ psf}$$

W_5 = Settlement of top of pile segment 4

$$= W_4 + \frac{\sigma_4 \Delta L_4}{E_{conc}} = 1.70 \times 10^{-2} + \frac{(141{,}297)(6.00)}{4.5 \times 10^{8}} = 1.89 \times 10^{-2} \text{ ft}$$

$$f_4 = \frac{W_4}{5.52 \times 10^{-6}} = \frac{1.70 \times 10^{-2}}{5.52 \times 10^{-6}} = 3080 \text{ psf; use } f_{max} = 1713 \text{ psf}$$

$$\Delta\sigma_4 = \frac{2\Delta L_4 f_{max}}{R} = \frac{2(6.00)(1713)}{(0.75)} = 27{,}408 \text{ psf}$$

$$\sigma_5 = \sigma_4 + \Delta\sigma_4 = 141{,}297 + 27{,}408 = 161{,}705 \text{ psf}$$

W_6 = Settlement of top of pile segment 5

$$= W_5 + \frac{\sigma_5 \Delta L_5}{E_{conc}} = 1.89 \times 10^{-2} + \frac{(161{,}705)(6.25)}{4.5 \times 10^{8}} = 2.11 \times 10^{-2} \text{ ft}$$

$$f_5 = \frac{W_5}{17.66 \times 10^{-6}} = \frac{1.89 \times 10^{-2}}{17.66 \times 10^{-6}} = 1070 \text{ psf}$$

$$\Delta\sigma_5 = \frac{2\Delta L_5 f_5}{R} = \frac{2(6.25)(1070)}{(0.75)} = 17833 \text{ psf}$$

$$\sigma_6 = \sigma_5 + \Delta\sigma_5 = 161{,}705 + 17833 = 179{,}538 \text{ psf}$$

W_7 = Settlement of top of pile segment 6

$$= W_6 + \frac{\sigma_6 \Delta L_6}{E_{conc}} = 2.11 \times 10^{-2} + \frac{(179,538)(5.25)}{4.5 \times 10^8} = 2.32 \times 10^{-2} \text{ ft}$$

$$f_6 = \frac{W_6}{26.93 \times 10^{-6}} = \frac{2.11 \times 10^{-2}}{26.93 \times 10^{-6}} = 784 \text{ psf}$$

$$\Delta\sigma_6 = \frac{2\Delta L_6 f_6}{R} = \frac{2(5.25)(784)}{0.75} = 10976 \text{ psf}$$

$$\sigma_7 = \sigma_6 + \Delta\sigma_6 = 179,538 + 10976 = 190,514 \text{ psf}$$

W_{8a} = Settlement of top of pile segment 7

$$= W_7 + \frac{\sigma_7 \Delta L_7}{E_{conc}} = 2.32 \times 10^{-2} + \frac{(190,514)(3.25)}{4.5 \times 10^8} = 2.46 \times 10^{-2} \text{ ft}$$

$$f_7 = \frac{W_7}{32.65 \times 10^{-6}} = \frac{2.32 \times 10^{-2}}{32.65 \times 10^{-6}} = 711 \text{ psf}$$

$$\Delta\sigma_7 = \frac{2\Delta L_7 f_7}{R} = \frac{2(3.25)(711)}{(0.75)} = 6162 \text{ psf}$$

$$\sigma_{8a} = \sigma_7 + \Delta\sigma_7 = 190,514 + 6162 = 196,676 \text{ psf}$$

W_{8b} = Settlement of top of pile segment 8a

$$= W_{8a} + \frac{\sigma_{8a} \Delta L_{8a}}{E_{conc}} = 2.46 \times 10^{-2} + \frac{(196,676)(1.75)}{4.5 \times 10^8} = 2.54 \times 10^{-2} \text{ ft}$$

$$f_{8a} = \frac{W_{8a}}{17.96 \times 10^{-6}} = \frac{2.46 \times 10^{-2}}{17.96 \times 10^{-6}} = 1370 \text{ psf; use } f_{max} = 1044 \text{ psf}$$

$$\Delta\sigma_{8a} = \frac{2\Delta L_{8a} f_{max}}{R} = \frac{2(1.75)(1044)}{0.75} = 4872 \text{ psf}$$

$$\sigma_{8b} = \sigma_{8a} + \Delta\sigma_{8a} = 196,676 + 4872 = 201,548 \text{ psf}$$

W_T = Settlement of top of pile segment 8b

$$= W_{8b} + \frac{\sigma_{8b} \Delta L_{8b}}{E_{conc}} = 2.54 \times 10^{-2} + \frac{(201,548)(1.75)}{4.5 \times 10^8} = 2.62 \times 10^{-2} \text{ ft}$$

$f_{8b} = 0$

$\Delta\sigma_{8b} = 0$

$\sigma_T = 201{,}548$ psf

$$Q_T = \sigma_T \times \pi R^2$$

$$= 201{,}548 \times \pi(0.75)^2$$

$$= 356{,}165 \text{ lb} = 178 \text{ tons}$$

q - w AND f-w CURVES BY METHOD B

q-w Curve

$$\frac{q}{w} = \frac{2E_R}{\pi(1-\nu^2)R}$$

$$= \frac{2(2,088,500)}{\pi(1-.33^2)(1.5)}$$

$$= 994,711$$

$$w = 1.01 \times 10^{-6}\ q\ \text{ft}$$

$$q_{max} = 192,328\ \text{psf}$$

f-w Curves

$$\frac{f}{w} = \frac{E_R}{2(1+\nu)(1+\ln(^L/2R)R}$$

$$= \frac{E_R}{2(1+.33)(1+\ln(34.5/2(.75)).75} = \frac{E_R}{8.250}$$

$$w = \frac{8.250\ f}{E_R}$$

Segment	Depth (ft)	E_R (psf)	w (ft)	f_{max}(psf)	ΔL (ft)
8b	0.00 - 1.75	710,090	$(11.62 \times 10^{-6})f$	0	1.75
8a	1.75 - 3.50	710,090	$(11.62 \times 10^{-6})f$	1044	1.75
7	3.50 - 6.75	148,284	$(55.64 \times 10^{-6})f$	940	3.25
6	6.75 - 12.00	227,647	$(36.24 \times 10^{-6})f$	1253	5.25
5	12.00 - 18.25	524,214	$(15.74 \times 10^{-6})f$	1316	6.25
4	18.25 - 24.25	998,303	$(8.26 \times 10^{-6})f$	1713	6.00
3	24.75 - 28.75	1,142,410	$(7.22 \times 10^{-6})f$	1713	4.50
2	28.75 - 32.50	1,743,898	$(4.73 \times 10^{-6})f$	1713	3.75

1 The bell is in the region from 32.50 to 34.50. It will be assumed to behave like a 2.25 ft diameter cylinder.

$$\frac{f}{w} = \frac{2,088,500}{2(1+.33)(1+\ln(34.5/2.25))}\ 1.125$$

$$= 187,106$$

$w = 5.34\ \text{No}^{-6}\ f\ \text{ft}$ $\Delta L = 2.00\ \text{ft}$

$f_{max} = 1713\ \text{psf}$

VERTICAL LOAD SETTLEMENT CURVE BY METHOD B

Assuming A Point Bearing Pressure q = 20,000 psf,

Then $P_1 = \pi(1.5)^2(20{,}000) = 141{,}372$ lb

W_1 = Point Settlement

$= 1.01 \times 10^{-6}(20{,}000) = 2.02 \times 10^{-2}$ ft

W_2 = Settlement of Top of Pile Segment 1

$$= W_1 + \frac{P_1 \Delta L_1}{AE_{conc}} = 2.02 \times 10^{-2} + \frac{(141{,}372)(2.00)}{\pi(1.125)^2(4.5 \times 10^8)} = 2.04 \times 10^{-2} \text{ ft}$$

$$f_1 = \frac{W_1}{5.34 \times 10^{-6}} = \frac{2.04 \times 10^{-2}}{5.34 \times 10^{-6}} = 3820 \text{ psf; use } f_{max} = 1713 \text{ psf}$$

$$\Delta\sigma_1 = \frac{2\pi R_1 \Delta L_1 f_{max}}{\pi R_2^2} = \frac{2(1.125)(2.00)(1713)}{(0.75^2)} = 13{,}704 \text{ psf}$$

$$\sigma_2 = \sigma_1 + \Delta\sigma_1 = \frac{141{,}372}{\pi(.75)^2} + 13704 = 93{,}704 \text{ psf}$$

W_3 = Settlement of Top of Pile Segment 2

$$= W_2 + \frac{\sigma_2 \Delta L_2}{E_{conc}} = 2.04 \times 10^{-2} + \frac{(93{,}704)(3.75)}{4.5 \times 10^8} = 2.12 \times 10^{-2} \text{ ft}$$

$$f_2 = \frac{W_2}{4.73 \times 10^{-6}} = \frac{2.04 \times 10^{-2}}{4.73 \times 10^{-6}} = 4313 \text{ psf; use } f_{max} = 1713 \text{ psf}$$

$$\Delta\sigma_2 = \frac{2\Delta L_2 f_{max}}{R} = \frac{2(3.75)(1713)}{(0.75)} = 17{,}130 \text{ psf}$$

$$\sigma_3 = \sigma_2 + \Delta\sigma_2 = 93{,}704 + 17130 = 110{,}834 \text{ psf}$$

W_4 = Settlement of Top of Pile Segment 3

$$= W_3 + \frac{\sigma_3 \Delta L_3}{E_{conc}} = 2.12 \times 10^{-2} + \frac{(110{,}834)(4.50)}{4.5 \times 10^8} = 2.23 \times 10^{-2} \text{ ft}$$

$$f_3 = \frac{W_3}{7.22 \times 10^{-6}} = \frac{2.12 \times 10^{-2}}{7.22 \times 10^{-6}} = 2936 \text{ psf; use } f_{max} = 1713 \text{ psf}$$

$$\Delta\sigma_3 = \frac{2\Delta L_3 f_{max}}{R} = \frac{2(4.50)(1713)}{0.75} = 20556 \text{ psf}$$

$$\sigma_4 = \sigma_3 + \Delta\sigma_3 = 110{,}834 + 20556 = 131{,}390 \text{ psf}$$

W_5 = Settlement of Top of Pile Segment 4

$$= W_4 + \frac{\sigma_4 \Delta L_4}{E_{conc}} = 2.23 \times 10^{-2} + \frac{(131{,}390)(6.00)}{4.5 \times 10^8} = 2.41 \times 10^{-2} \text{ ft}$$

$$f_4 = \frac{W_4}{8.26 \times 10^{-6}} = \frac{2.23 \times 10^{-2}}{8.26 \times 10^{-6}} = 2700 \text{ psf; use } f_{max} = 1713 \text{ psf}$$

$$\Delta\sigma_4 = \frac{2\Delta L_4 f_{max}}{R} = \frac{2(6.00)(1713)}{0.75} = 27{,}408 \text{ psf}$$

$$\sigma_5 = \sigma_4 + \Delta\sigma_4 = 131{,}390 + 27408 = 158{,}798 \text{ psf}$$

W_6 = Settlement of Top of Pile Segment 5

$$= W_5 + \frac{\sigma_5 \Delta L_5}{E_{conc}} = 2.41 \times 10^{-2} + \frac{(158{,}798)(6.25)}{4.5 \times 10^8} = 2.63 \times 10^{-2} \text{ ft}$$

$$f_5 = \frac{W_5}{15.74 \times 10^{-6}} = \frac{2.41 \times 10^{-2}}{15.74 \times 10^{-6}} = 1531 \text{ psf; use } f_{max} = 1316 \text{ psf}$$

$$\Delta\sigma_5 = \frac{2\Delta L_5 f_{max}}{R} = \frac{2(6.25)(1316)}{0.75} = 21{,}933 \text{ psf}$$

$$\sigma_6 = \sigma_5 + \Delta\sigma_5 = 158{,}798 + 21{,}933 = 180{,}731 \text{ psf}$$

W_7 = Settlement of Top of Pile Segment 6

$$= W_6 + \frac{\sigma_6 \Delta L_6}{E_{conc}} = 2.63 \times 10^{-2} + \frac{(180{,}731)(5.25)}{4.5 \times 10^8} = 2.84 \times 10^{-2} \text{ ft}$$

$$f_6 = \frac{W_6}{36.24 \times 10^{-6}} = \frac{2.63 \times 10^{-2}}{36.24 \times 10^{-6}} = 726 \text{ psf}$$

$$\Delta\sigma_6 = \frac{2\Delta L_6 f_6}{R} = \frac{2(5.25)(726)}{0.75} = 10{,}164 \text{ psf}$$

$$\sigma_7 = \sigma_6 + \Delta\sigma_6 = 180{,}731 + 10{,}164 = 190{,}895 \text{ psf}$$

W_{8a} = Settlement of Top of Pile Segment 7

$$= W_7 + \frac{\sigma_7 \Delta L_7}{E_{conc}} = 2.84 \times 10^{-2} + \frac{(190{,}895)(3.25)}{4.5 \times 10^{8}} = 2.98 \times 10^{-2} \text{ ft}$$

$$f_7 = \frac{W_7}{55.64 \times 10^{-6}} = \frac{2.84 \times 10^{-2}}{55.64 \times 10^{-6}} = 510 \text{ psf}$$

$$\Delta\sigma_7 = \frac{2\Delta L_7 f_7}{R} = \frac{2(3.25)(510)}{0.75} = 4420 \text{ psf}$$

$$\sigma_{8a} = \sigma_7 + \Delta\sigma_7 = 190{,}895 + 4420 = 195{,}315 \text{ psf}$$

W_{8b} = Settlement of Top of Pile Segment 8a

$$= W_{8a} + \frac{\sigma_{8a} \Delta L_{8a}}{E_{conc}} = 2.98 \times 10^{-2} + \frac{(195{,}315)(1.75)}{4.5 \times 10^{8}} = 3.06 \times 10^{-2} \text{ ft}$$

$$f_{8a} = \frac{W_{8a}}{11.62 \times 10^{-6}} = \frac{2.98 \times 10^{-2}}{11.62 \times 10^{-6}} = 2565 \text{ psf}; \quad \text{use } f_{max} = 1044 \text{ psf}$$

$$\Delta\sigma_{8a} = \frac{2\Delta L_{8a} f_{max}}{R} = \frac{2(1.75)(1044)}{0.75} = 4872 \text{ psf}$$

$$\sigma_{8b} = \sigma_{8a} + \Delta\sigma_{8a} = 195{,}315 + 4872 = 200{,}187 \text{ psf}$$

W_T = Settlement of Top of Pile, Segment 8b

$$= W_{8b} + \frac{\sigma_{8b} \Delta L_{8b}}{E_{conc}} = 3.06 \times 10^{-2} + \frac{(200{,}187)(1.75)}{4.5 \times 10^{8}} = 3.14 \times 10^{-2} \text{ ft}$$

$$f_{8b} = 0$$

$$\Delta\sigma_{8b} = 0$$

$$\sigma_T = 200{,}187 \text{ psf}$$

$$Q_T = \sigma_T \times \pi R^2$$

$$= (200{,}187) \times \pi\ (0.75)^2$$

$$= 353{,}760 \text{ lb} = 177 \text{ tons}$$

q-w AND f-w CURVES BY METHOD C

q-w Curve

$$0 \leq q \leq \frac{1}{2} q_{max}, \quad \frac{q}{w} = \frac{5.5\ E_o}{R}$$

$$= \frac{5.5(891,700)}{1.5}$$

$$= 3,269,567$$

$$w = 0.306 \times 10^{-6}\ q\ \text{ft}$$

$$\frac{1}{2} q_{max} < q \leq q_{max}, \quad w = \frac{(5q - 2q_{max})R}{5.5\ E_o}$$

$$= 0.306 \times 10^{-6}(5q - 2q_{max})\ \text{ft}$$

$$q_{max} = 100,937\ \text{psf}$$

f-w Curves

$$0 \leq f \leq \frac{1}{2} f_{max}, \quad \frac{f}{w} = \frac{\alpha E_o}{R}$$

$$= \frac{(0.76)(R)E_o}{R}$$

$$= 0.76\ E_o$$

$$w = \frac{1.316\ f}{E_o}$$

$$\frac{1}{2} f_{max} < f \leq f_{max}, \quad w = \frac{(5f - 2f_{max})F}{\alpha E_o}$$

$$= \frac{(5f - 2f_{max})1.5}{(.76)(1.5)E_o}$$

$$= \frac{1.316(5f - 2f_{max})}{E_o}$$

Segment	Depth (ft)	E_o (psf)	w (ft)		f_{max} (psf)		ΔL (ft)
			$0 \le f \le (\frac{1}{2}) f_{max}$	$(\frac{1}{2}) f_{max} < f \le f_{max}$	Low	High	
8b	0.00 - 1.75	125,310	$10.50 \times 10^{-6} f$	$52.50 \times 10^{-6} f - 21.00 \times 10^{-6} f_{max}$	0	0	1.75
8a	1.75 - 3.50	125,310	$10.50 \times 10^{-6} f$	$52.50 \times 10^{-6} f - 21.00 \times 10^{-6} f_{max}$	397	1023	1.75
7	3.50 - 6.75	68,921	$19.09 \times 10^{-6} f$	$95.46 \times 10^{-6} f - 38.12 \times 10^{-6} f_{max}$	355	940	3.25
6	6.75 - 12.00	83,540	$15.75 \times 10^{-6} f$	$78.75 \times 10^{-6} f - 31.50 \times 10^{-6} f_{max}$	480	1253	5.25
5	12.00 - 18.25	127,399	$10.33 \times 10^{-6} f$	$51.64 \times 10^{-6} f - 20.66 \times 10^{-6} f_{max}$	522	1316	6.25
4	18.25 - 24.25	407,258	$3.23 \times 10^{-6} f$	$16.15 \times 10^{-6} f - 6.46 \times 10^{-6} f_{max}$	668	1671	6.00
3	24.25 - 28.75	236,001	$5.58 \times 10^{-6} f$	$27.88 \times 10^{-6} f - 11.15 \times 10^{-6} f_{max}$	668	1671	4.50
2	28.75 - 32.50	346,691	$3.80 \times 10^{-6} f$	$18.98 \times 10^{-6} f - 7.59 \times 10^{-6} f_{max}$	668	1671	3.75
1	32.50 - 34.50	891,790	$1.48 \times 10^{-6} f$	$7.38 \times 10^{-6} f - 2.95 \times 10^{-6} f_{max}$	668	1671	2.00

VERTICAL LOAD SETTLEMENT CURVE BY METHOD C
(Using High f_{max} Values)

Assuming A Point Bearing Pressure q_1 = 20,000 psf

Then $P_1 = \pi(1.5)^2 (20,000) = 141,372$ lb

W_1 = Point Settlement

$$= 0.306 \times 10^{-6} (20,000) = 0.61 \times 10^{-2} \text{ ft}$$

The Bell (Segment 1) is Assumed to be A 2.25 ft Dia. Cylinder

W_2 = Settlement of Top of Pile Segment 1

$$= 0.61 \times 10^{-2} + \frac{P_1 \Delta L_1}{A_1 E_{conc}} = 0.61 \times 10^{-2} + \frac{(141,372)(2.00)}{\pi(1.125)^2(4.5 \times 10^8)} =$$

$$0.63 \times 10^{-2} \text{ ft}$$

$$f_1 = \frac{W_1 + 2.95 \times 10^{-6} f_{max}}{7.38 \times 10^{-6}} = \frac{0.61 \times 10^{-2} + (2.95 \times 10^{-6})(1671)}{7.38 \times 10^{-6}} = 1495 \text{ psf}$$

$$\Delta\sigma_1 = \frac{2\pi R_1 \Delta L_1 f_1}{\pi R_2^2} = \frac{2(1.125)(2.00)(1495)}{(0.75)^2} = 11,960 \text{ psf}$$

$$\sigma_2 = \sigma_1 + \Delta\sigma_1 = \frac{141,372}{\pi(.75)^2} + 5316 = 91,960 \text{ psf}$$

W_3 = Settlement of Top of Pile Segment 2

$$= W_2 + \frac{\sigma_2 \Delta L_2}{E_{conc}} = 0.63 \times 10^{-2} + \frac{(91,960)(3.75)}{4.5 \times 10^8} = 0.71 \times 10^{-2} \text{ ft}$$

$$f_2 = \frac{W_2 + 7.59 \times 10^{-6} f_{max}}{18.98 \times 10^{-6}} = \frac{0.63 \times 10^{-2} + (7.59 \times 10^{-6})(1671)}{18.98 \times 10^{-6}} = 1000 \text{ psf}$$

$$\Delta\sigma_2 = \frac{2\Delta L_2 f_2}{R} = \frac{2(3.75)(1000)}{0.75} = 10,000 \text{ psf}$$

$$\sigma_3 = \sigma_2 + \Delta\sigma_2 = 91,960 + 10,000 = 101,960 \text{ psf}$$

W_4 = Settlement of Top of Pile Segment 3

$$= W_3 + \frac{\sigma_3 \Delta L_3}{E_{conc}} = 0.71 \times 10^{-2} + \frac{(101,960)(4.50)}{4.5 \times 10^8} = 0.81 \times 10^{-2} \text{ ft}$$

$$f_3 = \frac{W_3 + 11.15 \times 10^{-6} f_{max}}{27.88 \times 10^{-6}} = \frac{0.71 \times 10^{-2} + (11.15 \times 10^{-6})(1671)}{27.88 \times 10^{-6}} =$$

923 psf

$$\Delta\sigma_3 = \frac{2 \Delta L_3 f_3}{R} = \frac{2(4.50)(923)}{(0.75)} = 11,076 \text{ psf}$$

$$\sigma_4 = \sigma_3 + \Delta\sigma_3 = 101,960+11076 = 113036 \text{ psf}$$

W_5 = Settlement of Top of Pile Segment 4

$$= W_4 + \frac{\sigma_4 \Delta L_4}{E_{conc}} = 0.81 \times 10^{-2} + \frac{(113,036)(6.00)}{4.5 \times 10^8} = 0.96 \times 10^{-2} \text{ ft}$$

$$f_4 = \frac{W_4 + 6.46 \times 10^{-6} f_{max}}{16.15 \times 10^{-6}} = \frac{0.81 \times 10^{-2} + 6.46 \times 10^{-6} \ (1671)}{16.15 \times 10^{-6}} = 1170 \text{ psf}$$

$$\Delta\sigma_4 = \frac{2 \Delta L_4 f_4}{R} = \frac{2(6.00)(1170)}{0.75} = 18,720 \text{ psf}$$

$$\sigma_5 = \sigma_4 + \Delta\sigma_4 = 113,036 + 18720 = 131,756$$

W_6 = Settlement of Top of Pile Segment 5

$$= W_5 + \frac{\sigma_5 \Delta L_5}{E_{conc}} = 0.96 \times 10^{-2} + \frac{(131,756)(6.25)}{4.5 \times 10^8} = 1.14 \times 10^{-2} \text{ ft}$$

$$f_5 = \frac{W_5 + 20.66 \times 10^{-6} f_{max}}{51.64 \times 10^{-6}} = \frac{0.96 \times 10^{-2} + (20.66 \times 10^{-6})(1316)}{51.64 \times 10^{-6}} = 712 \text{ psf}$$

$$\Delta\sigma_5 = \frac{2 \Delta L_5 f_5}{R} = \frac{2(6.25)(712)}{0.75} = 11,867$$

$$\sigma_6 = \sigma_5 + \Delta\sigma_5 = 131,756 + 11,867 = 143,623$$

W_7 = Settlement of Top of Pile Segment 6

$$= W_6 + \frac{\sigma_6 \Delta L_6}{E_{conc}} = 1.14 \times 10^{-2} + \frac{(143,623)(5.25)}{4.5 \times 10^8} = 1.31 \times 10^{-2} \text{ ft}$$

$$f_6 = \frac{W_6 + 31.50 \times 10^{-6} f_{max}}{78.75 \times 10^{-6}} = \frac{1.14 \times 10^{-2} + 31.50 \times 10^{-6}(1253)}{78.75 \times 10^{-6}} = 646 \text{ psf}$$

$$\Delta\sigma_6 = \frac{2\Delta L_6 f_6}{R} = \frac{2(5.25)(646)}{0.75} = 9044$$

$$\sigma_7 = \sigma_6 + \Delta\sigma_6 + 143,623 + 9044 = 152,667$$

W_{8a} = Settlement of Top of Pile Segment 7

$$= W_7 + \frac{\sigma_7 \Delta L_7}{E_{conc}} = 1.31 \times 10^{-2} + \frac{(152,667)(3.25)}{4.5 \times 10^8} = 1.42 \times 10^{-2} \text{ ft}$$

$$f_7 = \frac{W_7 + (38.12 \times 10^{-6}) f_{max}}{95.46 \times 10^{-6}} = \frac{1.42 \times 10^{-2} + (38.12 \times 10^{-6})(940)}{95.46 \times 10^{-6}} = 524 \text{ psf}$$

$$\Delta\sigma_7 = \frac{2\Delta L_7 f_7}{R} = \frac{2(3.25)(524)}{0.75} = 4541 \text{ psf}$$

$$\sigma_{8a} = \sigma_7 + \Delta\sigma_7 = 152,667 + 4541 = 157,208 \text{ psf}$$

W_{8b} = Settlement of Top of Pile Segment 8a

$$= W_{8a} + \frac{\sigma_{8a} \Delta L_{8a}}{E_{conc}} = 1.42 \times 10^{-2} + \frac{(157,208)(1.75)}{4.5 \times 10^8} = 1.48 \times 10^{-2} \text{ ft}$$

$$f_{8a} = \frac{W_{8a} + (21.00 \times 10^{-6}) f_{max}}{52.50 \times 10^{-6}} = \frac{1.42 \times 10^{-2} + (21.00 \times 10^{-6})(1023)}{52.50 \times 10^{-6}} =$$

680 psf

$$\Delta\sigma_{8a} = \frac{2\Delta L_{8a} f_{8a}}{R} = \frac{2(1.75)(680)}{0.75} = 3173 \text{ psf}$$

$$\sigma_{8b} = \sigma_{8a} + \Delta\sigma_{8a} = 157,208 + 3173 = 160,381 \text{ psf}$$

W_T = Settlement of Top of Pile Segment 8b

$$= W_{8b} + \frac{\sigma_{8b}\Delta L_{8b}}{E_{conc}} = 1.48 \times 10^{-2} + \frac{(160,381)(1.75)}{4.5 \times 10^8} = 1.54 \times 10^{-2} \text{ ft}$$

$$f_{8b} = 0$$

$$\Delta\sigma_{8b} = 0$$

$$\sigma_T = 160,381 \text{ psf}$$

$$Q_T = \sigma_T \times \pi R^2$$

$$= (160,381) \times \pi(0.75)^2$$

$$= 283,416 \text{ lb} = 142 \text{ tons}$$

VERTICAL LOAD-SETTLEMENT CURVE BY METHOD C
(Using Low f_{max} Values)

Assuming A Point Bearing Pressure q_1 = 20,000 psf

Then $P_1 = \pi(1.5)^2 (20,000) = 141,372$ lb

W_1 = Point Settlement

$= 0.306 \times 10^{-6} (20,000) = 0.61 \times 10^{-2}$ ft

The Bell (Segment 1) is Assumed to be A 2.25 ft Dia. Cylinder

W_2 = Settlement of Top of Pile Segment 1

$$= W_1 + \frac{P_1 \Delta L_1}{A_1 E_{conc}} = 0.61 \times 10^{-2} + \frac{(141,372)(2.00)}{\pi(1.125)^2(4.5 \times 10^8)} = 0.63 \times 10^{-2} \text{ ft}$$

$$f_1 = \frac{W_1 + 2.95 \times 10^{-6} f_{max}}{7.38 \times 10^{-6}} = \frac{0.63 \times 10^{-2} + (2.95 \times 10^{-6})(668)}{7.38 \times 10^{-6}} =$$

1121 psf; use f_{max} = 668 psf

$$\Delta\sigma_1 = \frac{2\pi R_1 \Delta L_1 f_{max}}{\pi R_2^{\ 2}} = \frac{2(1.125)(2.00)(668)}{(0.75)^2} = 5344 \text{ psf}$$

$$\sigma_2 = \sigma_1 + \Delta\sigma_1 = \frac{141,372}{\pi(0.75)^2} + 5344 = 85344$$

W_3 = Settlement of Top of Pile Segment 2

$$= W_2 + \frac{\sigma_2 \Delta L_2}{E_{conc}} = 0.63 \times 10^{-2} + \frac{(85,344)(3.75)}{4.5 \times 10^8} = 0.70 \times 10^{-2} \text{ ft}$$

$$f_2 = \frac{W_2 + 7.59 \times 10^{-6} f_{max}}{18.98 \times 10^{-6}} = \frac{0.70 \times 10^{-2} + (7.59 \times 10^{-6})(668)}{18.98 \times 10^{-6}} = 639 \text{ psf}$$

$$\Delta\sigma_2 = \frac{2\Delta L_2 f_2}{R} = \frac{2(3.75)(639)}{0.75} = 6390 \text{ psf}$$

$$\sigma_3 = \sigma_2 + \Delta\sigma_2 = 85{,}344 + 6390 = 91{,}734 \text{ psf}$$

W_4 = Settlement of Top of Pile Segment 3

$$= W_3 + \frac{\sigma_3 \Delta L_3}{E_{conc}} = 0.70 \times 10^{-2} + \frac{(91{,}734)(4.50)}{4.5 \times 10^8} = 0.79 \times 10^{-2} \text{ ft}$$

$$f_3 = \frac{W_3 + 11.15 \times 10^{-6} f_{max}}{27.88 \times 10^{-6}} = \frac{0.70 \times 10^{-2} + (11.15 \times 10^{-6})(668)}{27.88 \times 10^{-6}} = 518 \text{ psf}$$

$$\Delta\sigma_3 = \frac{2\Delta L_3 f_3}{R} = \frac{2(4.50)(518)}{0.75} = 6216 \text{ psf}$$

$$\sigma_4 = \sigma_3 + \Delta\sigma_3 = 91{,}734 + 6216 = 97{,}950 \text{ psf}$$

W_5 = Settlement of Top of Pile Segment 4

$$= W_4 + \frac{\sigma_4 \Delta L_4}{E_{conc}} = 0.79 \times 10^{-2} + \frac{(97{,}950)(6.00)}{4.50 \times 10^8} = 0.92 \times 10^{-2} \text{ ft}$$

$$f_4 = \frac{W_4 + (6.46 \times 10^{-6}) f_{max}}{16.15 \times 10^{-6}} = \frac{0.79 \times 10^{-2} + (6.46 \times 10^{-6})(668)}{16.15 \times 10^{-6}} =$$

756 psf; use f_{max} = 668 psf

$$\Delta\sigma_4 = \frac{2\Delta L_4 f_{max}}{R} = \frac{2(6.00)(668)}{0.75} = 10{,}688 \text{ psf}$$

$$\sigma_5 = \sigma_4 + \Delta\sigma_4 = 97{,}950 + 10{,}688 = 108{,}638 \text{ psf}$$

W_6 = Settlement of Top of Pile Segment 5

$$= W_5 + \frac{\sigma_5 \Delta L_5}{E_{conc}} = 0.92 \times 10^{-2} + \frac{(108{,}638)(6.25)}{4.5 \times 10^8} = 1.07 \times 10^{-2} \text{ ft}$$

$$f_5 = \frac{W_5 + (20.66 \times 10^{-6}) f_{max}}{51.64 \times 10^{-6}} = \frac{0.92 \times 10^{-2} + (20.66 \times 10^{-6})(522)}{51.64 \times 10^{-6}} =$$

387 psf