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16. Abstract

In this report, a detailed description is made of the established procedures to design shallow foundations on the basis of preboring pressuremeter tests. Both the bearing capacity and 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 over 50 case histories.

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PRESSUREMETER DESIGN OF SHALLOW FOUNDATIONS

by

Jean-Louis Briaud and Gerald Jordan

Research Report 340-1

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
U. S. Department of Transportation, Federal Highway Administration

Texas Transportation Institute
The Texas A&M University System
College Station, Texas

April 1983

SUMMARY

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

The bearing capacity equation is:

$$q_p = k p_{Le}^* + q_o$$

where q_p is the bearing capacity of the foundation, k is the pressuremeter bearing capacity factor, p_{Le}^* is the equivalent net limit pressure obtained from preboring pressuremeter tests performed within the zone of influence of the foundation and q_o is the vertical total pressure at the foundation level prior to construction. The bearing capacity factor k depends on the relative depth of embedment of the foundation, the type of soil, and the shape of the foundation. Charts for k have been proposed by Menard and Gambin in 1963, Baquelin, Jezequel and Shields in 1978, and Bustamante and Gianeselli in 1982.

The three charts are presented and used to solve several example problems. The results of those examples show that generally the Bustamante-Gianeselli method gives the lowest bearing capacity values, that the Menard-Gambin method gives higher values and that the Baquelin-Jezequel-Shields method gives values which are slightly higher than the values obtained with the Menard-Gambin method.

The settlement equation is:

$$S = \frac{2}{9} \frac{1}{E_d} q B_o (\lambda_d \frac{B}{B_o})^{\alpha} + \frac{\alpha}{9} \frac{1}{E_c} q \lambda_c B$$

where S is the settlement of the foundation, E_d is the average modulus obtained from preboring pressuremeter tests performed within several foundation widths below the foundation level, q is the net bearing pressure, B_o is a reference width, B is the width of the foundation, λ_d and λ_c are shape factors, α is a rheologic factor, E_c is the average modulus obtained from preboring pressuremeter tests performed immediately below the foundation level.

The two terms of the settlement equations correspond to two distinct components: the settlement due to shearing stresses (deviatoric component) and the settlement due to hydrostatic compression (spherical component). When the width of the foundation is small compared to the thickness of the bearing stratum (common case of shallow foundation), the settlement due to shearing stresses is larger than the settlement due to the hydrostatic compression.

The above settlement equation applies when the ratio of the foundation width to the thickness of the bearing stratum is small. This equation is modified when the ratio is large and in this case the pressuremeter settlement analysis should be complemented by a consolidation test analysis. Example of settlement calculations are presented to illustrate the design procedures in various cases.

The above bearing capacity and settlement rules are evaluated by presenting the results of comparisons between predicted and measured behavior for over 50 case histories. It must be emphasized that one of the critical elements in the accuracy of the predictions is the performance of quality pressuremeter tests by trained professionals.

IMPLEMENTATION SETTLEMENT

This report gives the details of existing pressuremeter methods for the design of shallow foundations. 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.

ACKNOWLEDGMENTS

The authors are grateful for the continued support and encouragement of Mr. George Odom of the Texas State Department of Highways and Public Transportation.

DISCLAIMER

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GLOSSARY OF TERMS AND EQUATIONS

BEARING CAPACITY

k = Pressuremeter bearing capacity factor

$$= \frac{p_L \text{ (sphere)}}{p_L \text{ (cylinder)}}$$

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

p_{OH} = total horizontal stress in soil at rest

p_L = ultimate limit pressure

q_p = pressuremeter bearing capacity

$$q_p = k\, p_{Le}^* + q_o$$

$$q_{safe} = k\, p_{Le}^*/3 + q_o$$

$$q_{net} = k\, p_{Le}^*/3$$

where p_{Le}^* = equivalent net limit pressure

q_o = total stress overburden at foundation level

H_e = equivalent depth of embedment

$$H_e = \sum_{1}^{n} \Delta z_i \frac{p_{Li}^*}{p_{Le}^*}$$

q_p' = reduced bearing capacity for slopes and excavations

$q_p' = \mu\, q_p$ where: μ = reduction factor

q_p = normal bearing capacity

Q_v = vertical load on the foundation

f_s = friction on the side of the foundation

C_u = undrained soil shear strength

D = actual depth of embedment of the foundation

L = length of the foundation

B = width of the foundation

s_T = long-term, drained settlement

s_u = rapid, undrained settlement

s_c = consolidation settlement = $s_T - s_u$

Layer moduli by harmonic mean:

$$\frac{n}{E_k} = \sum_1^n \frac{1}{E_i}$$ where E_k = average PMT modulus within kth layer

E_i = moduli from PMT results in kth layer

Settlement with a thin, soft layer at depth:

$s = s' + s''$

s' = settlement without considering soft layer

s'' = settlement of soft layer alone

$$= \alpha\left(\frac{1}{E_{soft}} - \frac{1}{E_{hard}}\right)\Delta\sigma_v H$$

where $\Delta\sigma_v$ = change in vertical pressure between top and bottom of soft layer

H = thickness of soft layer

α = rheological factor

E = pressuremeter modulus

Settlement of a thin, soft layer at ground surface:

$$S = \sum_1^n \frac{\alpha_i\, p_i}{E_i}\, \Delta z_i$$

i = layer number,

where β = coefficient based on the safety factor, F

$$F = \frac{\text{ultimate bearing pressure}}{\text{actual bearing pressure}}$$

$$\beta = \frac{2}{3}\left(\frac{F}{F-1}\right)$$

Δ_{vi} = change in vertical pressure in the ith layer

αi = rheological factor of each layer

Δz_i = thickness of each layer

The pressuremeter settlement equation:

$$S = \frac{2}{9E_d} q \; B_o \; \lambda_d \; \frac{B}{B_o}^{\alpha} + \frac{\alpha}{9E_c} q \; \lambda_c \; B$$

S = total footing settlement

E_d = pressuremeter modulus within zone of deviatoric tensor influence

$$\frac{1}{E_d} = \frac{1}{4}\left(\frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3/5}} + \frac{1}{2.5E_{6/8}} + \frac{1}{2.5E_{9/16}}\right)$$

E_i = pressuremeter modulus within zone of spherical tensor influence

E_c = first layer average modulus

q = net footing bearing pressure (q_{net})

B_o = reference width = 2 ft or 60 cm

λ_d = deviatoric shape factor

λ_c = spherical shape factor

α = rheological factor

STRESS, STRAIN, MODULI

σ = total stress tensor

$= \sigma_s + \sigma_d$

where: σ_s = spherical stress component

σ_d = deviatoric stress component

likewise: ε_s = spherical strain component

ε_d = deviatoric strain component

E = Young's Modulus

γ = Poisson's Ratio

G = Shear Modulus

K = Bulk Modulus

CHAPTER 1. - INTRODUCTION

The established procedures to design shallow foundations on the basis of preboring pressuremeter tests are presented in detail in this report. In a first part the bearing capacity and settlement calculations are described in the form of step-by-step procedures. Then the accuracy of the methods presented are evaluated by comparing predicted and measured behavior of shallow foundations for over 50 case histories. Finally, design examples are solved to illustrate the design rules.

It must be emphasized that one of the critical elements for the successful prediction of shallow 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. - BEARING CAPACITY

2.1 Theoretical Background

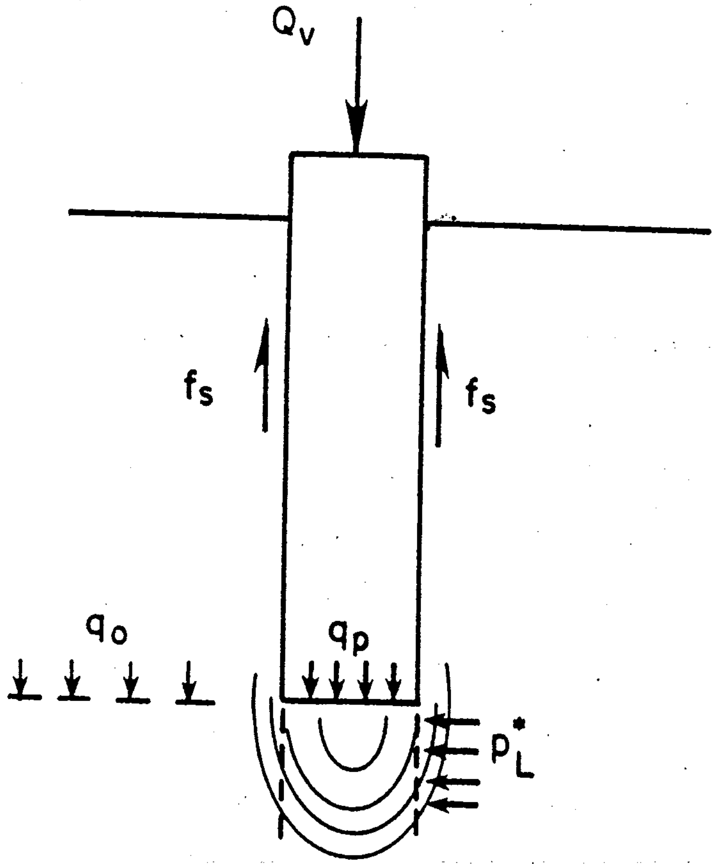
Figures 1 and 2 show the analogy between the pressuremeter limit pressure p_L and the ultimate bearing capacity q_p. If the penetration of a circular footing is associated with the expansion of a spherical cavity, then the ultimate bearing capacity of that footing is given by the limit pressure to the expansion of a spherical cavity (p_L sphere). The pressuremeter test on the other hand is associated with the expansion of a cylindrical cavity and leads to a limit pressure (p_L cylinder). The ratio between the pressuremeter limit pressure and the ultimate bearing capacity of a circular footing could therefore be expressed as the pressuremeter bearing capacity factor, k:

$$k = \frac{p_L \text{ (sphere)}}{p_L \text{(cylinder)}} \quad . \qquad (1).$$

This theoretical bearing capacity factor can be evaluated using plasticity theory; such values of k vary from 1.4 to 2.4 (6). However, the k values have been determined from full scale field tests.

2.2 Methods for Funding the Bearing Capacity Factor, k

At present there are three methods available to find the bearing capacity factor for shallow foundations. These are: the Menard chart

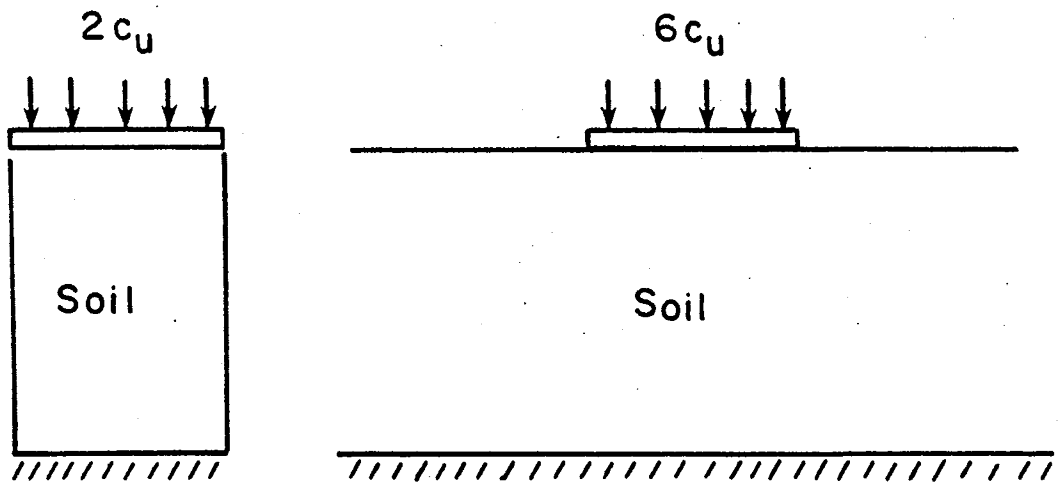


$$q_p = k\, p^*_{Le} + q_o$$

$$f_s = \alpha p^*_L$$

Theoretically $k = 1.4$ To 2.4
However k And α Obtained
From Load Test Data

FIGURE 1: Pressuremeter Bearing Capacity Method for Foundations



Bearing Capacity Of Plate = 6 c_u
Part Of Bearing Capacity
Due To Vertical Resistance Only = 2c_u
Part Of Bearing Capacity Due
To Lateral Resistance Only = 4c_u
Where $c_u \equiv$ Undrained Shear Strength

$$q^* = k\, p^*_{Le}$$

$$6\, c_u = k \times 4\, c_u \longrightarrow k = 1.5$$

FIGURE 2: Footing Capacity Due To Lateral Soil Support

(Ref. 10, Fig. 3), charts by Baguelin, Jezeguel, and Shields (B.J.S., Ref. 1, Fig. 4), and a chart developed after Bustamante and Gianselli (B.G., Ref. 3, Fig. 5).

Figure 5 was obtained from the early part of the B.G. chart for piles. It was assumed that circular footings have the same capacity factors as very shallow bored piles. This led to the design curves for circular footings. The curves for the strip footings were obtained by reducing consistently the k values of the circular footings.

The Menard, the B.J.S., and the B.G. charts relate the bearing capacity factor to a relavtive depth for various soil classifications. These charts can handle circular, square, and strip footings. Values of k must be interpolated for rectangular footings.

The Menard and B.G. charts use similar soil classification tables to distinguish between design curves (Figures 6 and 7). Both charts express k as a function of the ratio of the equivalent embedment depth of the foundation (H_e) to the radius of the foundation R. For non circular footings the radius of the foundation is considered to be half the width B of the foundation.

The B.J.S. charts express k as a function of the depth to width ratio $\frac{H_e}{B}$ (Figure 4). There are four charts; each one is used for a single soil classification and gives different curves for different soil strengths (p_L^*). This seems to allow for a more detailed determination of k. Anytime an interpolation is necessary to find the bearing capacity factor, a linear variation is assumed to exist between the design points on the chart; for rectangular footings the interpolation parameter is $\frac{B}{L}$ where L is the length of the foundation.

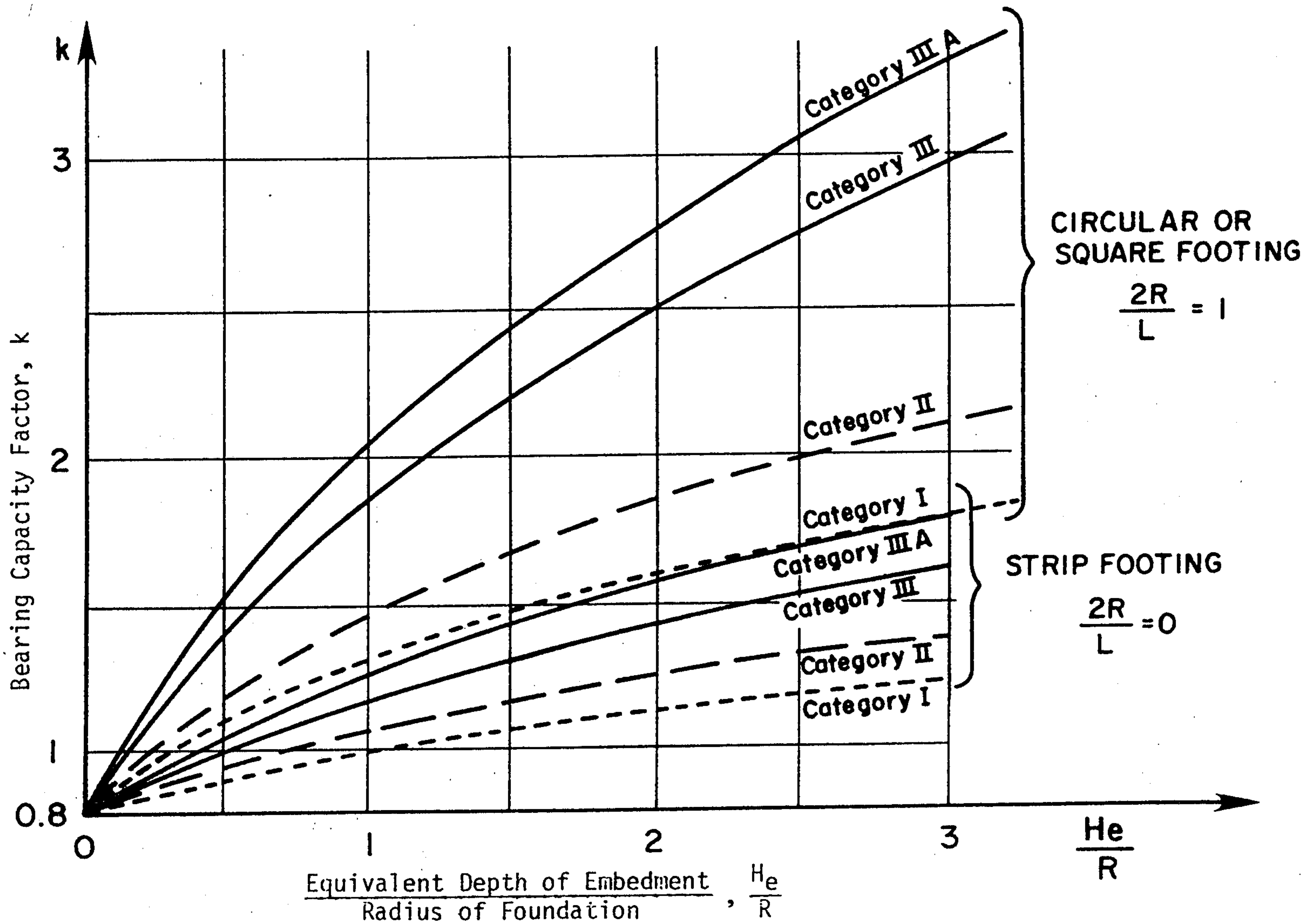


FIGURE 3: Bearing Capacity Factors For Menard (Reference 10)

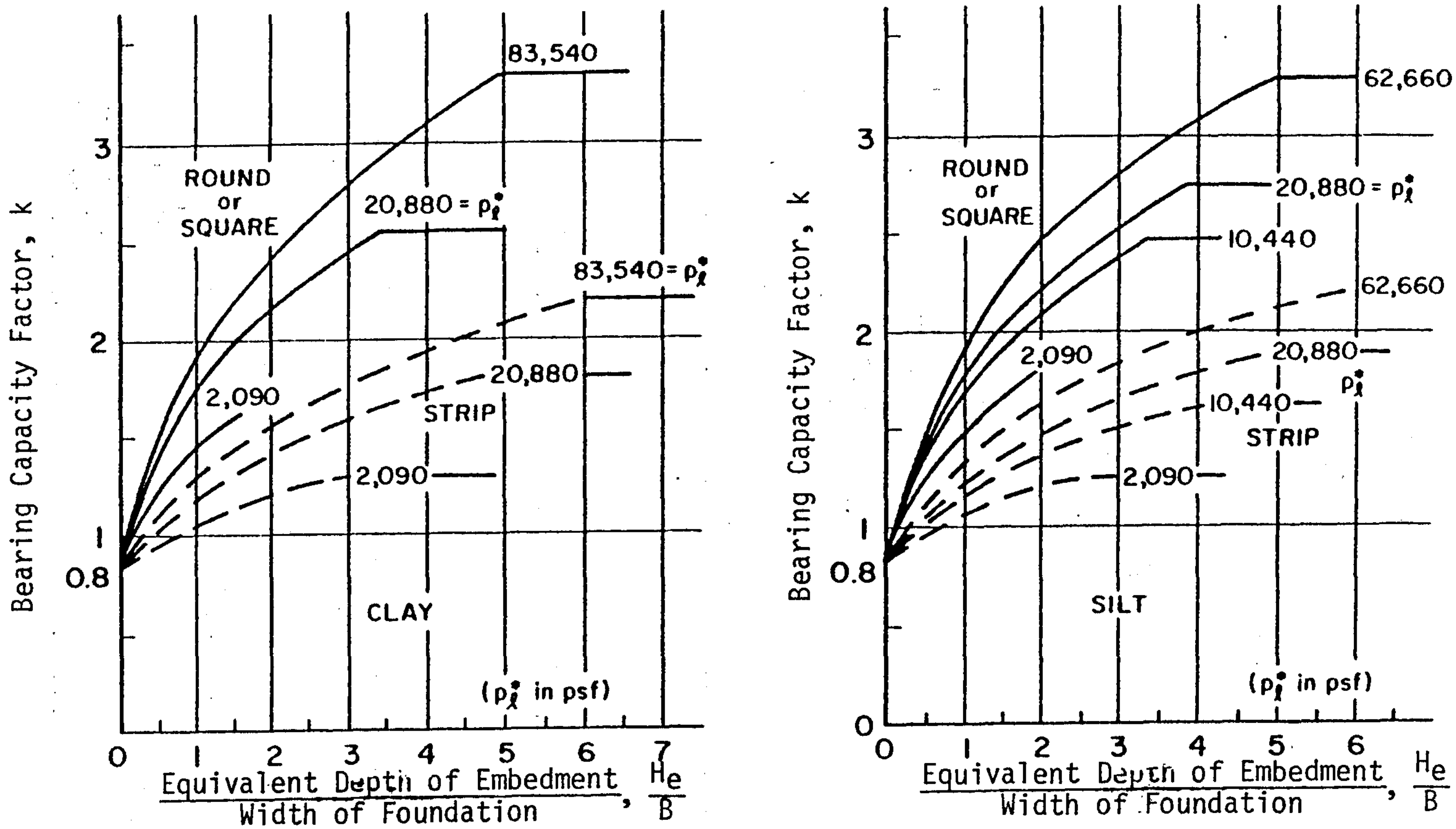
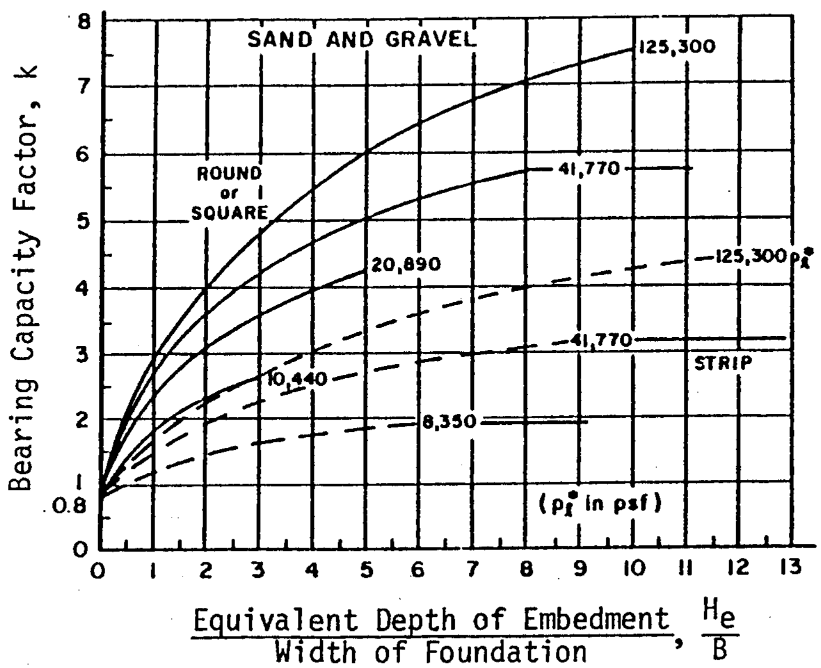


FIGURE 4: Bearing Capacity Factors For Baguelin, Jezeguel, Shields Method (Reference 1)



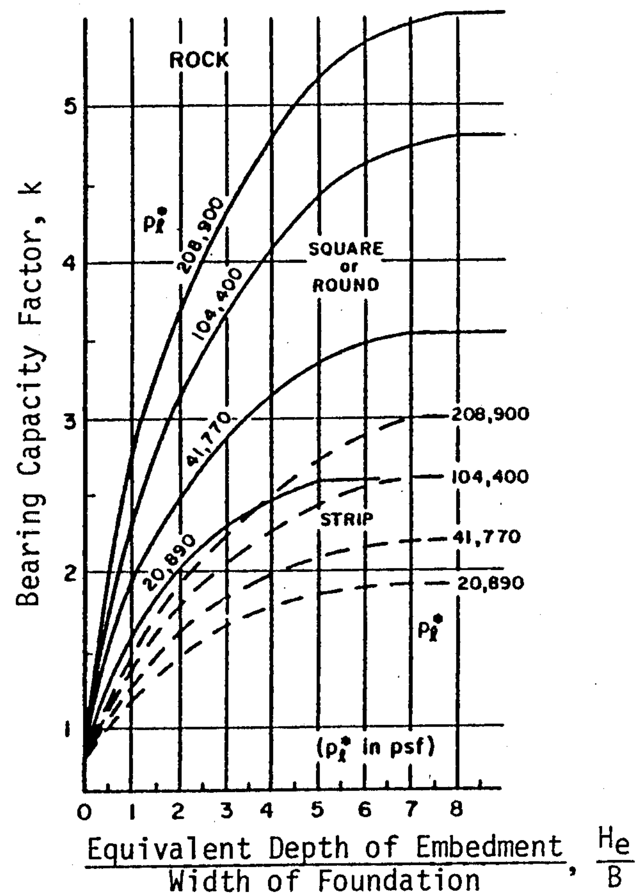


FIGURE 4: (Continued)

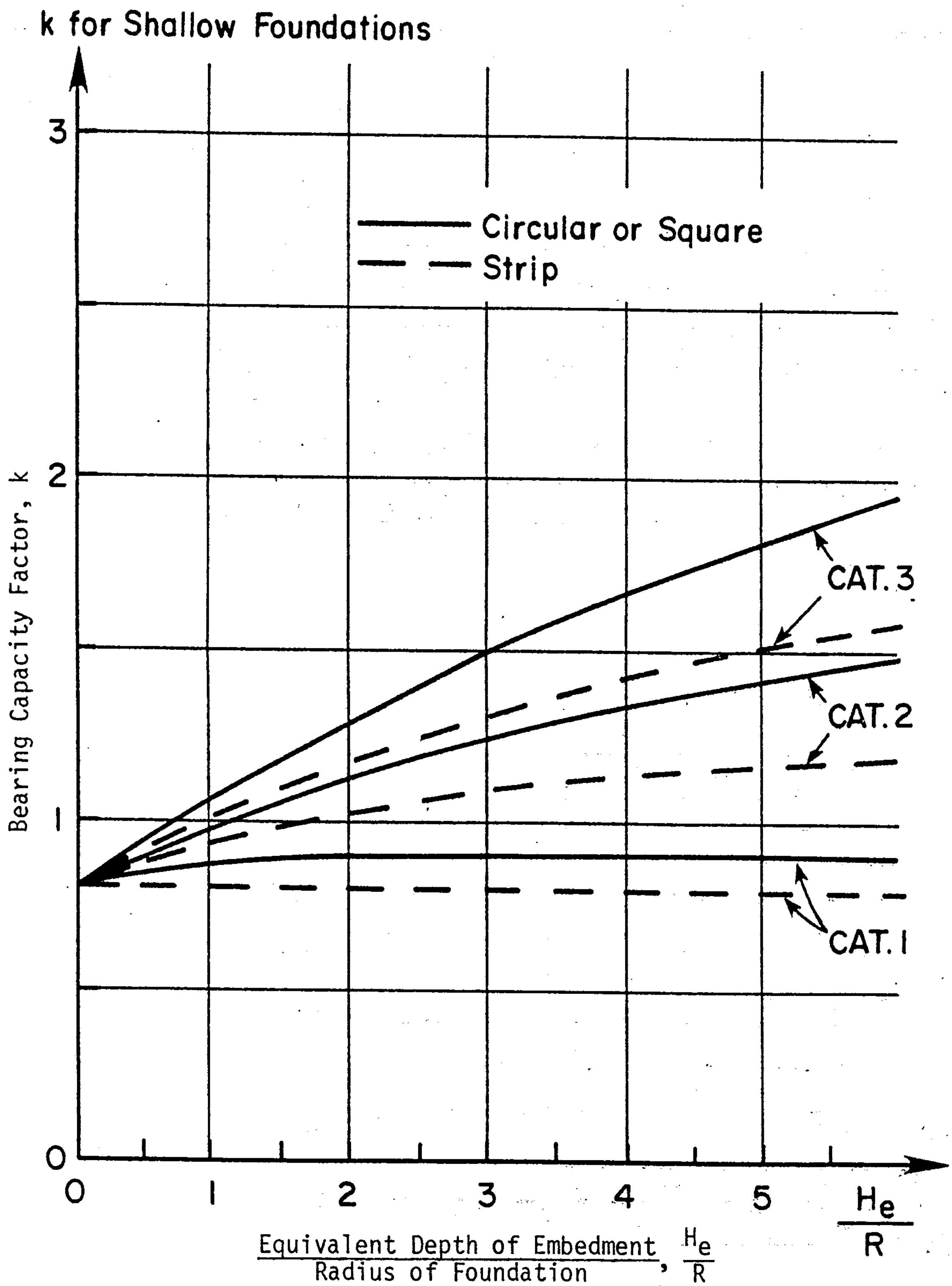


FIGURE 5: Bearing Capacity Factors
For Bustamante and Gianeselli
(Reference 3)

LIMIT PRESSURE (psf)	SOIL TYPE	CATEGORY
0 - 25000	Clay	I
0 - 15000	Silt	
37500 - 84000	Firm Clay or Marl	II
25000 - 63000	Compact Silt	
8000 - 17000	Compressible Sand	
21000 - 63000	Soft or Weathered Rock	
21000 - 42000	Sand and Gravel	III
83500 - 209000	Rock	
62500 - 125000	Very Compact Sand and Gravel	IIIA

Figure 6: Soil Categories for Use with Menard Bearing Capacity Chart of Figure 3 (Reference 10)

LIMIT PRESSURE (psf)	SOIL TYPE	CATEGORY
15000	Soft Clay	
17000	Silt and Soft Chalk	1
15000	Loose Clayey, Silty, or Muddy Soil	
21 - 42,000	Medium Dense Sand and Gravel	
25 - 63,000	Clay and Compact Silt	
31 - 84,000	Marl and Limestone-Marl	
21 - 52,000	Weathered Chalk	2
52 - 84,000	Weathered Rock	
63,000	Fragmented Chalk	
94,000	Very Compact Marl	
52,000	Dense to Very Dense Sand and Gravel	3
94,000	Fragmented Rock	

Figure 7. Soil Categories for Use with Bustamante and Gianeselli Bearing Capacity Chart of Figure 5 (Reference 3)

The B.J.S. and Menard charts give similar k values; the B.G. chart gives consistently lower values. The effects of these differences on bearing capacity can be seen in examples 1a,b,c and 2a,b,c.

2.3 Bearing Capacity Equation

The ultimate bearing capacity, q_p is:

$$q_p = k\, p^*_{Le} + q_o \qquad (2).$$

where

k = pressuremeter bearing capacity factor (Figs. 3, 4, 5),

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

p_{oh} = total horizontal stress at rest p_L = limit pressure (from test),

p^*_{Le} = equivalent net limit pressure near the foundation level, and

q_o = total stress overburden pressure at foundation level.

2.4 Calculating p^*_{Le}, the Equivalent Limit Pressure

$$p^*_{Le} = \sqrt[n]{p^*_{L1} \times p^*_{L2} \times \ldots \times p^*_{Ln}} \qquad (3).$$

where $p_{L1}, \ldots, p_{Ln}$ are the net limit pressures obtained from tests performed within the (+) 1.5B to (-) 1.5B zone near the foundation level. See examples 3 and 4.

2.5 Calculating H_e, the Equivalent Depth of Embedment

$$H_e = \sum_1^n \Delta z_i \, \frac{p^*_{Li}}{p^*_{Le}} \qquad (4).$$

where p^*_{Li} are the limit pressures obtained from tests between the ground surface and the foundation level, and Δz_i are the thicknesses of the elementary layers corresponding to the pressuremeter tests. See examples 3 and 4.

2.6 Obtaining k, the Pressuremeter Bearing Capacity Factor

The relative embedment depth is H_e/R for the Menard and the B.G. method, and H_e/B for the B.J.S. method. The parameter R is the radius of the footing or half the width and B is the diameter or the width of the footing.

The soil category is determined from Figures 6 and 7 for Menard and B.G. method, respectively; B.J.S. has separate charts for different soils. Then the bearing capacity factor is read on Figure 3, 4, or 5. If the footing is rectangular, linear interpolation is performed between the case of a square footing and the case of a strip footing; the interpolating parameter is B/L. See Examples 1, 2, and 3.

2.7 Calculating q_p, q_{safe}, and q_{net}

$$q_p = kp^*_{Le} + q_o, \quad (5).$$

$$q_{safe} = \frac{kp^*_{Le}}{3} + q_o, \text{ and} \quad (6).$$

$$q_{net} = q_{safe} - q_o = \frac{kp^*_{Le}}{3} . \quad (7).$$

See Examples 1, 2, and 3, Section 3.

2.8 Reduction of the Bearing Capacity Factor for Footings Near Excavations

It is sometimes necessary or practical to set footings near slopes or excavations. In this case the bearing capacity factor must be reduced to allow for the reduced lateral confinement in the soil below the footing.

Menard (10) proposes a reduction factor (μ) related to the tangent of the angle β between the near edge of the footing and the bottom of the excavation, or the angle of the slope on which the footing rests.

The definition of β and the value of μ can be found on Figure 8. The bearing capacity of the footing, q'_p is:

$$q'_p = \mu q$$

where q_p is the bearing capacity of the footing on flat ground and μ is the reduction factor. According to Menard, tan β should not exceed 0.67.

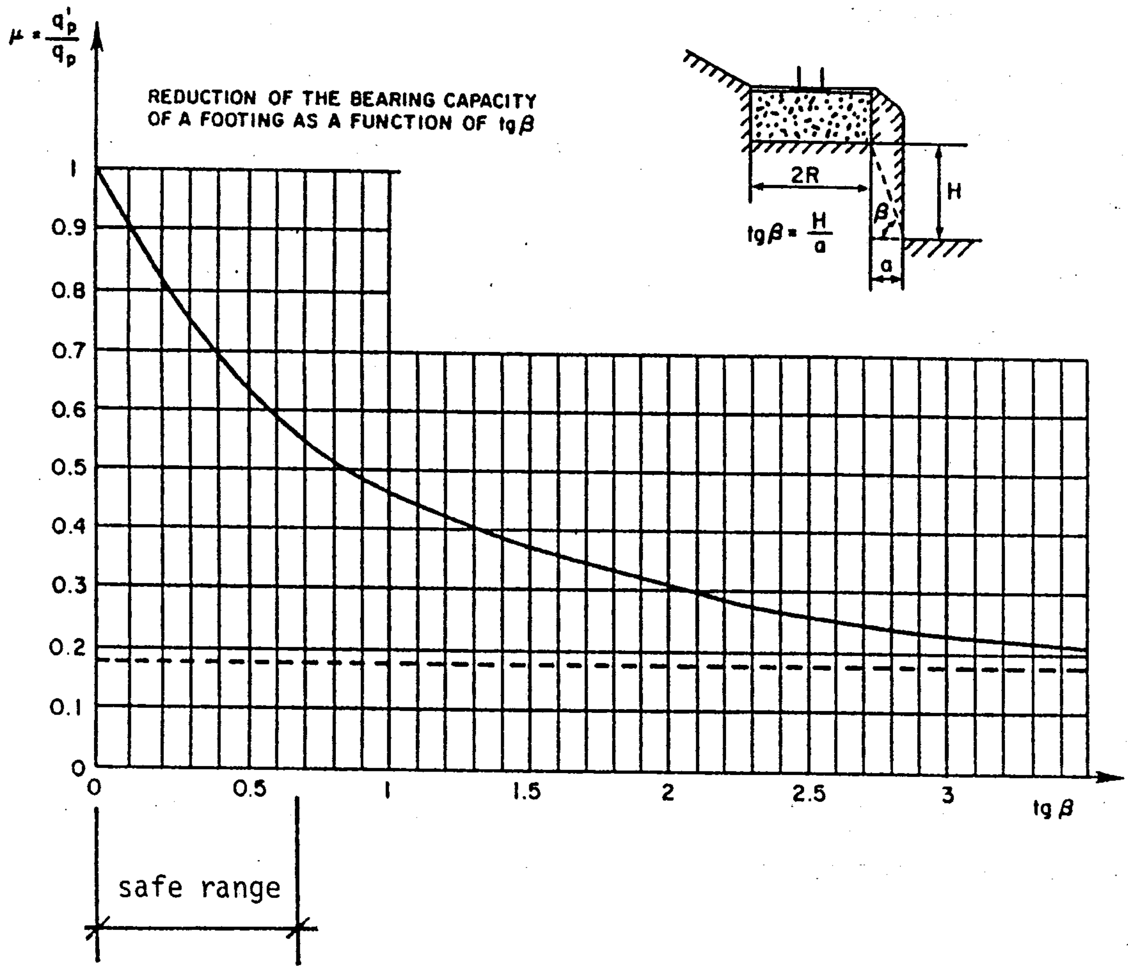


FIGURE 8: REDUCTION OF THE BEARING CAPACITY OF A FOOTING AS A FUNCTION OF TAN β (Reference 10)

CHAPTER 3. - SETTLEMENT

3.1 Menard Method

3.1.1 Theoretical Background

Two settlements can be considered: an undrained or no volume change settlement s_u which takes place rapidly and a drained or final settlement s_T. In elasticity s_u would be calculated by using undrained parameters (E_u, ν_u, G_u) and s_T by using drained-long term parameters (E', ν', G) where: E is Young's modulus, ν is Poisson's ratio, and G is the shear modulus.

The stress tensor (σ) at any point within the loaded mass of soil can be decomposed into its spherical component (σ_s), and deviatoric component (σ_d):

$$\sigma = \sigma_s + \sigma_d \qquad (8).$$

In elasticity the stress-strain relations can be written:

$$\sigma_s = 3K\varepsilon s = \frac{E}{3(1-2\nu)}\,\varepsilon_s \qquad (9).$$

$$\sigma_d = 2G\varepsilon_d = \frac{E}{1+\nu}\,\varepsilon_d \qquad (10).$$

where

K = Bulk Modulus,

ε_s = Spherical Strain Tensor, and

ε_d = Deviatoric Strain Tensor.

The deviatoric component of the stress tensor is the same whether it is expressed in effective stress or total stress. Therefore:

$$\sigma_{du} = \sigma_d \tag{11}$$

Since,

$$\sigma_{du} = 2G_u \, \varepsilon_d \tag{12}$$

and,

$$\sigma'_d = 2G' \, \varepsilon_d \tag{13}$$

then,

$$G_u = G' = G. \tag{14}$$

Let us consider the settlement of a rigid circular plate on an elastic half space:

$$s_T = \frac{\pi}{8} \frac{1-\nu'}{G} qB \tag{15}$$

and,

$$s_u = \frac{\pi}{8} \frac{1-0.5}{G} qB \tag{16}$$

The difference $s_T - s_u$ is the consolidation settlement s_c,

$$s_u = \frac{\pi}{16} \frac{qB}{G} \tag{17}$$

$$s_c = \frac{\pi}{16} (1-2\nu') \frac{qB}{G}. \tag{18}$$

For an average Poisson's ratio (ν') of 0.33, s_u is three times larger than s_c and therefore represent 75% of the total settlement s_T; this shows that when the width of the foundation is small compared to the depth of the compressible layer (most common case for shallow footings) the undrained settlement is the major portion of the final settlement.

The above discussion of the settlement problem is the backbone of the pressuremeter equation for settlement (3):

$$s = \underbrace{\frac{2}{9E_d} q B_o \left[\lambda_d \frac{B}{B_o}\right]^\alpha}_{\text{deviatoric settlement}} + \underbrace{\frac{\alpha}{9E_c} q \lambda_c B}_{\text{spherical settlement}} \qquad (19).$$

where

s = footing settlement,

E_d = pressuremeter modulus within the zone of influence of the deviatoric tensor,

q = footing net bearing pressure q_{net} ,

B_o = reference width of 2 ft. or 60 cm.,

B = footing width,

λ_d = shape factor for deviatoric term (Figure 10),

λ_c = shape factor for spherical term (Figure 10),

α = rheological factor (Figure 9), and

E_c = pressuremeter modulus within the zone of influence of the spherical tensor.

This equation is an elasticity equation which has been altered to take into account the real soil behavior, in particular the footing scale effect B^α and the magnitude of the pressuremeter modulus. This equation is applicable to pressuremeter results obtained in prebored holes.

3.1.2 Calculating the Layer Moduli

The soil below the foundation level is divided into a series of elementary layers B/2 thick (Fig. 11.) . In each layer the average pressuremeter modulus is calculated using the PMT results within that

Soil Type	Peat		Clay		Silt		Sand		Sand and Gravel	
	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α
Over-consolidated			>16	1	>14	2/3	>12	1/2	>10	1/3
Normally consolidated	for all values	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
	$\alpha = 1/3$	$\alpha = 1.2$	$\alpha = 2/3$

FIGURE 9: Rheological Factor α (Reference 1)

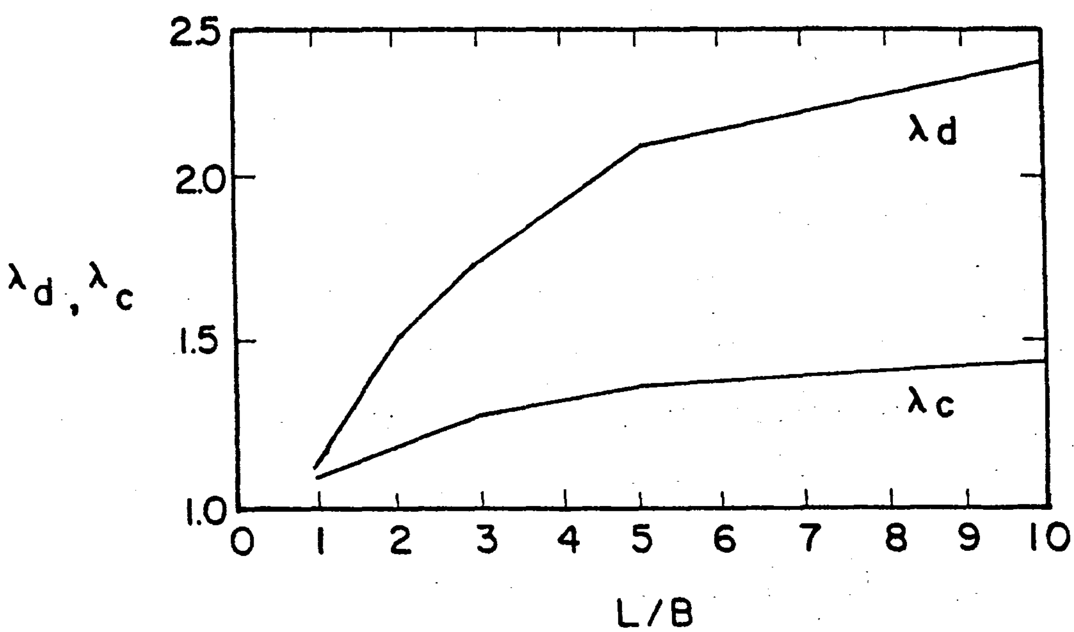


FIGURE 10: Shape Factors λ_c, λ_d. (Reference 1)

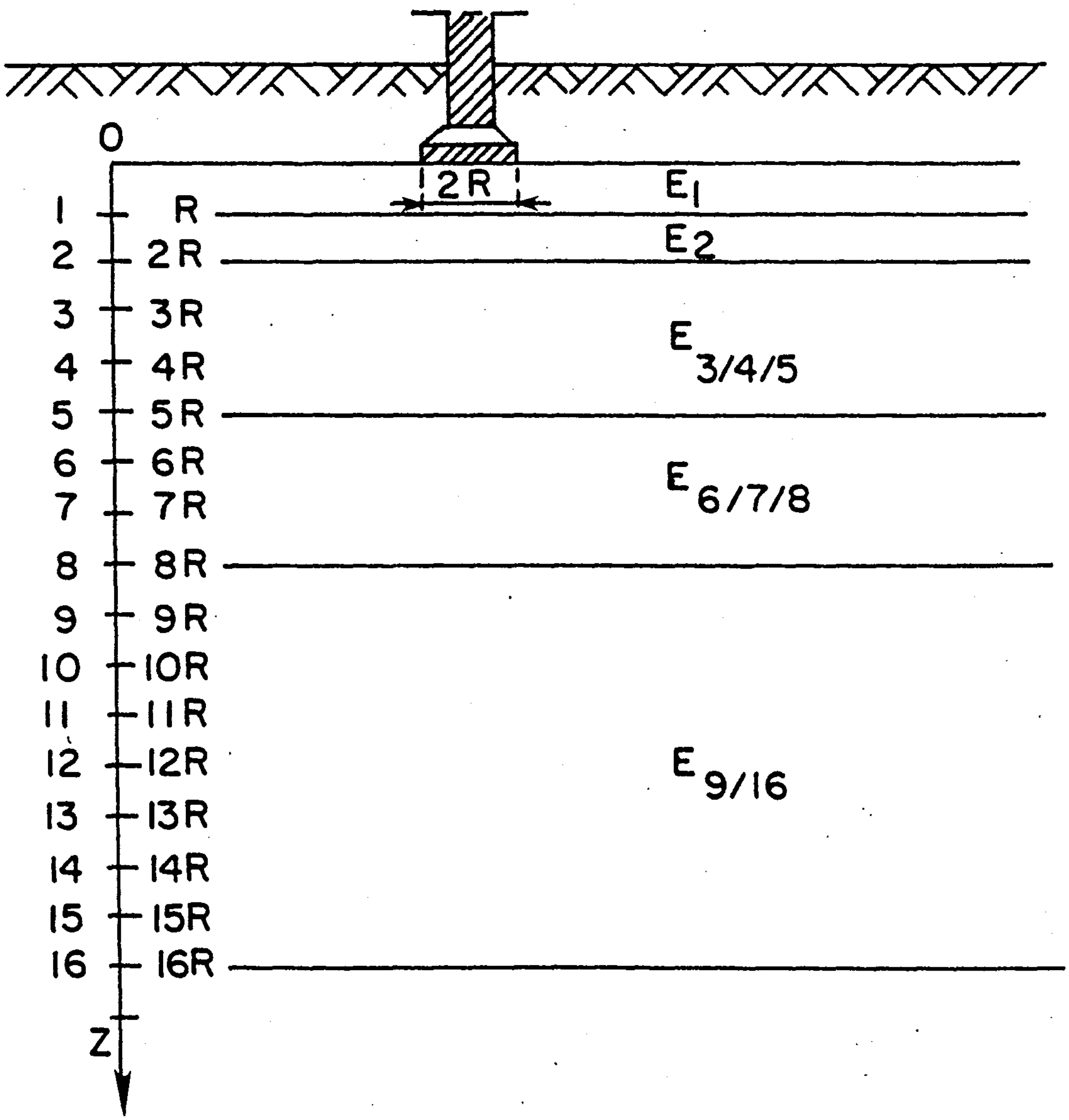


FIGURE 11: Layers to be considered in the Settlement Analysis

layer and the harmonic mean technique,

$$\frac{n}{E_k} = \sum_1^n \frac{1}{E_i}, \qquad (20).$$

where

E_i = PMT moduli within k^{th} layer and

E_k = average PMT modulus of k^{th} layer.

This process is repeated for all layers (1 through 16); if no PMT data is available beyond a certain depth z the moduli of the layers deeper than z are assumed to be equal to the deepest measured modulus. See Example 3.

3.1.3 Calculating E_c and E_d

According to the theory of elasticity the spherical part of the strain tensor (ε_s) decreases rapidly with depth (1); on the contrary, the magnitude of the deviatoric part of the strain tensor (E_d) is significant even at large depth. As a result, E_c is taken as the modulus of the first layer under the footing (see Example 3). On the other hand, E_d is taken as an equivalent modulus within 16 layers, B/2 thick, under the footing; the formula which gives the equivalent distortion modulus E_d is based on an assumed reasonable E_d strain distribution (2):

$$\frac{1}{E_d} = \frac{1}{4}\left(\frac{1}{E_1} + \frac{1}{0.85\ E_2} + \frac{1}{E_{3/4/5}} + \frac{1}{2.5\ E_{6/7/8}} + \frac{1}{2.5\ E_{9/16}}\right) \qquad (21)$$

where $E_{p/q}$ is the harmonic mean of the moduli of layers p to q. For example,

$$\frac{3}{E_{3/4/5}} = \frac{1}{E_3} + \frac{1}{E_4} + \frac{1}{E_5} .$$

See Examples 3 and 4 for examples of complete calculations.

3.1.4 Obtaining α and λ

The parameters α, λ_d, and λ_c are obtained from Figures 9 and 10. The determination of α is made by assessing the soil type and estimating the state of consolidation through the use of the ratio, E/p_L^*. The shape factors λ_c and λ_d depend on the length to width ratio, L/B. See Examples 1, 2, and 3.

3.1.5 Calculating the Settlement

The settlement is calculated using equation (19) mentioned above; the bearing pressure is taken to be the net safe pressure:

$$q_{net} = q_{safe} - q_o . \tag{7}$$

See Examples 1, 2, and 3.

3.1.6 The Special Case of a Thin Soft Layer at Depth

In this case the settlement is (1):

$$s = s' + s'' \tag{22}$$

where s' is the settlement of the footing when considering that the modulus of the soft layer (E_{soft}) is the same as the modulus of the soil immediately above the soft layer (E_{hard}) ; and s" is the compression of the soft layer alone.

$$s' = \frac{2}{9E_d} q B_o (\lambda_d \frac{B}{B_o})^\alpha + \frac{\alpha}{9E_c} q \lambda_c B \text{ and} \quad (19)$$

$$s'' = \alpha(\frac{1}{E_{soft}} - \frac{1}{E_{hard}}) \Delta\sigma_v H. \quad (23)$$

$\Delta\sigma_v$ is the average increase in vertical stress in the soft layer and H is the thickness of the soft layer. The settlement s" is calculated using an elasticity formula with a modulus equal to E/α. See Example 5.

3.1.7 The Special Case of a Thin Soft Layer Close to the Ground Surface

If the raft or the embankment rests on a soft layer which is thinner than B/2, the settlement of the soft layer is calculated as (1):

$$s = \sum_{1}^{n} \frac{\alpha i \; \beta \; \Delta\sigma i}{E_i} \Delta z_i, \quad (24)$$

where n is the number of layers constituting the soft layer and β is a function of the safety factor, F.

$$\beta = \frac{2}{3} \times \frac{F}{F-1}$$

F is the ratio of ultimate bearing capacity to the pressure applied by the foundation, $\Delta\sigma_{vi}$ is the average increase in vertical pressure in the i^{th} layer computed by elastic theory, αi is the rheological factor for the i^{th} layer, E_i is the pressuremeter modulus for the i^{th} layer, and Δ_{zi} is the thickness of the i^{th} layer. Equation (24) above is based on the theory of elasticity using a modulus $\frac{E}{\alpha}$.

The coefficient β tends to take into account the increase in compressibility beyond the preconsolidation pressure and is explained as follows:

1. s is a consolidation settlement,
2. if the factor of safety is 3, the bearing pressure is likely to be close to or smaller than the preconsolidation pressure, and β is 1 in this case.
3. if the factor of safety is less than 3, the bearing pressure is likely to exceed the preconsolidation pressure and β increases accordingly.

See Example 6.

3.2 Settlement : Schmertmann Method Using Pressuremeter Moduli

A method was developed by Schmertmann to calculate the settlement of shallow footings on sands (11, 12). The method is based on the theory of elasticity. A simplified strain distribution under the footing is assumed, a profile of moduli is obtained, and the compression of the layers within the depth of influence is calculated.

Schmertmann recommended that the soil modulus be obtained from the cone penetrometer test (CPT) by a correlation to the CPT point resistance. If no cone penetrometer data is available, the Standard Penetration Test blow count could also be used. Although Schmertmann made no mention of it, it appears to be logical to use the pressuremeter modulus profile in connection with this method. However, the pressuremeter modulus is usually a large strain modulus and may not be appropriate for Schmertmann's method. Further work is necessary to prove whether or not this alternate method is accurate. Encouraging results have already been obtained (4).

The Schmertmann-pressuremeter method is described in detail below:

$$S = C_1 C_2 \, \Delta p \sum_{0}^{n} \left(\frac{I_{zi}}{E_i} \Delta z_i \right) \tag{26}$$

where:

C_1 = Correction factor to take into account beneficial effect of embedment depth

$$C_1 = 1 - 0.5 \frac{p'_{ov\,(1)}}{p} \; ; \; 0.5 < C_1 < 1$$

$p'_{ov\,(1)}$ = effective vertical stress at foundation level after construction (See Figure 12)

C_2 = correction factor accounting for creep settlement

$$C_2 = 1 + 0.2 \log \left(\frac{t\ (yrs)}{0.1}\right);\ C_2 \leq 1$$

$$t \leq 1 \text{ yrs}$$

Δp = net bearing pressure = $p - p'_{ov}$

p = bearing pressure at foundation level

p'_{ov} = effective vertical stress at foundation level before construction

n = total number of layers

I_{zi} = average influence factor for the i^{th} layer

Figure 12 shows the simplified distribution of the strain influence factor proposed by Schmertmann. This distribution reaches a maximum I_{zmax}, expressed as:

$$I_{zmax} = 0.5 + 0.1 \sqrt{\frac{\Delta p}{\sigma'_{vp}}}$$

where:

σ'_{vp} = effective stress at the depth of I_{zmax} before construction

E_i = pressuremeter modulus of the i^{th} layer

Δz_i = thickness of i^{th} layer

The distribution of I_z for square and strip footings is shown on Figure 12. Interpolations must be made for rectangular footings.

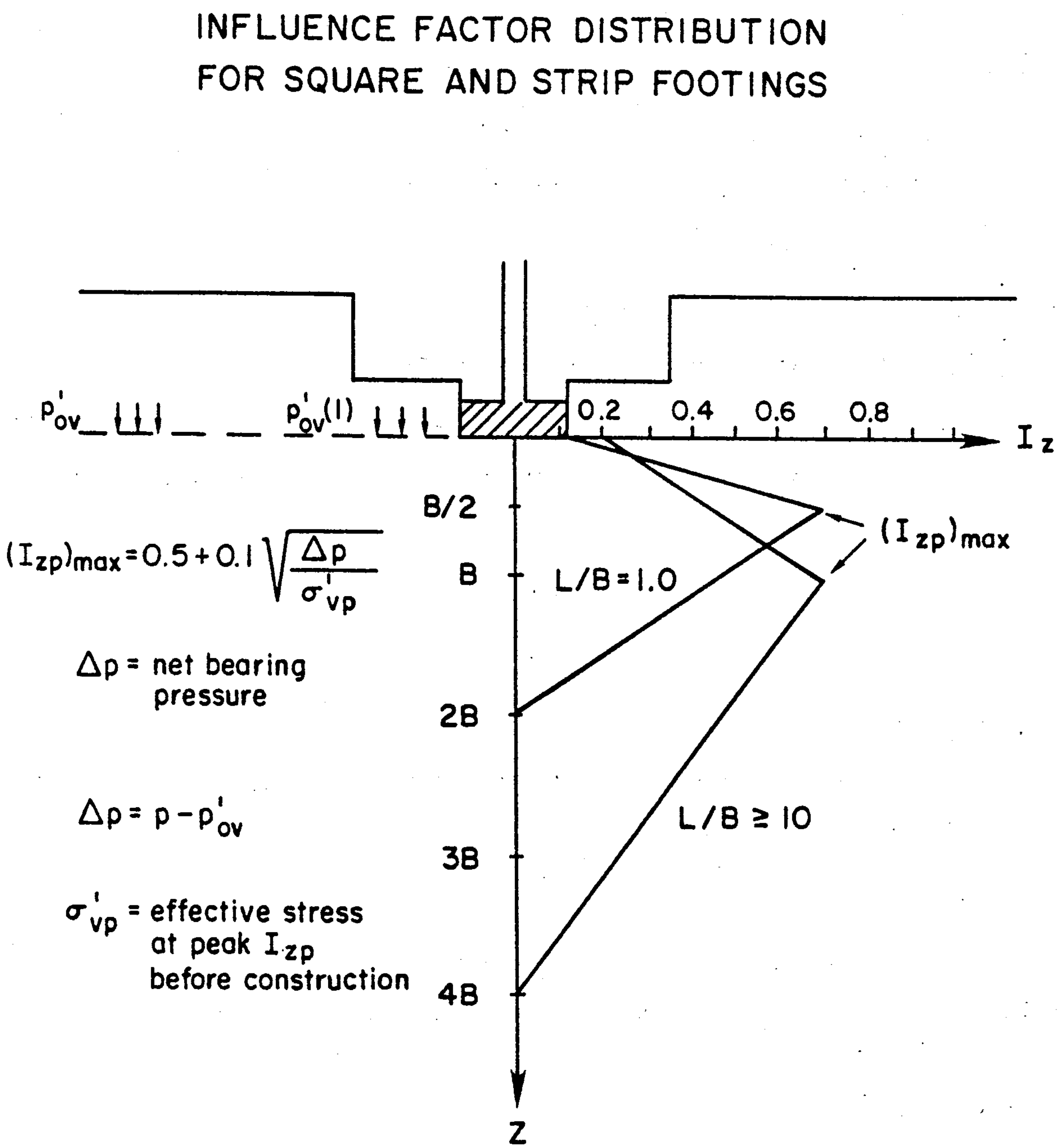


Figure 12: Schmertmann Settlement Concepts and the Influence Factor Distribution.

CHAPTER 4. - EXAMPLES OF DESIGN PROBLEMS AND THEIR SOLUTIONS

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

Example 1: Rectangular footing on a uniform deposit of clay. Menard, B.J.S., and B.G. methods demonstrated.

Example 2: Rectangular footing on a uniform deposit of sand. Menard, B.J.S., and B.G. methods demonstrated.

Example 3: Strip footing on a layered deposit of sand. Menard, B.J.S., and B.G. methods demonstrated.

Example 4: Rectangular footing on a layered deposit of clay. Menard method.

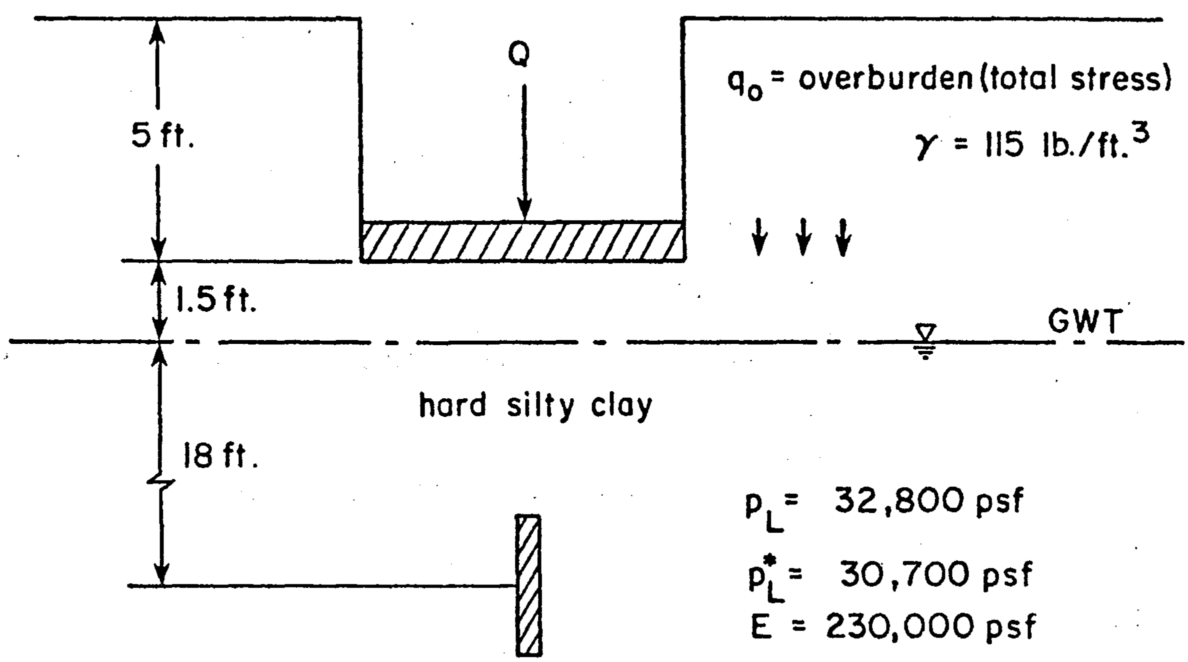
Example 5: Strip footing on a layered deposit of sand with a soft silt layer at depth.

Example 6. Mat foundation on a soft soil layer close to the ground surface.

EXAMPLE PROBLEM I:

RECTANGULAR FOOTING ON CLAY

L = 13 ft.   B = 6 ft.



EXAMPLE 1a. SHALLOW FOOTING ON A CLAY (MENARD METHOD)

Bearing Capacity

$q_L = k\ p_L^* + q_o$ $\qquad$ $q_{safe} = \frac{k}{3}\ p_L^* = q_o$

$p_{Le}^* = p_L^*$ $\qquad$ Ed = Ec (homogenous soil layer)

he = h = 5.0 ft $\qquad$ $R = \frac{B}{2} = 3.0$ ft

From Fig. 6 → Soil Category II $\qquad$ He/R = 1.67 and B/L = 0.46

From Fig. 3 → k(strip) = 1.20

k(square) = 1.76

Interpolating → $k\ (\frac{B}{L} = 0.46) = 1.46$

$q_{safe} = 1.46/3 \times 30700 \text{ psf} + (115 \frac{lb}{ft^3} \times 5 \text{ ft}) = 15516 \text{ psf}$

Settlement

$$s = \underbrace{\frac{2}{9E_d}\ q_{net}\ B_o \left[\lambda_d \frac{B}{B_o}\right]^{\alpha}}_{\text{deviatoric settlement}} + \underbrace{\frac{\alpha}{9E_c}\ q_{net}\ \lambda_c\ B}_{\text{spherical settlement}}$$

$q_{net} = q_{safe} - q_o = 15516 - 575 = 14941$ psf

$E/p_L \sim 7$; From Fig. 9 → $\alpha = 0.5$

L/B = 2.2; From Fig. 10 → $\lambda_d = 1.58$ and $\lambda_c = 1.22$

$$s = \frac{2}{9}\,\frac{1}{230{,}000} \times 14941 \times 2.0 \left[1.58\,\frac{6.0}{2.0}\right]^{0.5} + \frac{0.5}{9 \times 230{,}000} \times$$

$$14941 \times 1.22 \times 6.0$$

$$s = .063 + .026 = .089 \text{ ft} > s_{all} = .082 \text{ ft}$$

therefore, use $q_{all} = \frac{.082}{.089} \times 15516 = 14296$ psf

EXAMPLE 1b. SHALLOW FOOTING ON A CLAY (B.J.S. METHOD)

Bearing Capacity

$q_L = k\ p_L^* = q_o$

$p_{Le}^* = p_L^*$ $\quad\quad$ Ed = Ec

$He = h = 5.0$ ft $\quad\quad$ $R = \frac{B}{2} = 3.0$ ft

$D = He = 5.0$ ft $\quad\quad$ $D/B = 1.7$ $\quad\quad$ $p_L^* = 30700$ psf

From Fig. 4, k values for clay square footing:

for $p_L^* = 83540$ psf $\rightarrow k = 2.66$,

for $p_L^* = 20880$ psf $\rightarrow k = 2.36$, then

for $p_L^* = 30700$ psf $\rightarrow k = 2.41$

Similarly, for a strip footing: $k = 1.54$

One most interpolate between strip and square footings to yield:

$k\ (\frac{B}{2} = 0.46) = 1.94$

$$q_{safe} = \frac{1.94}{3} \times 30700 + (115 \times 5) = 20428 \text{ psf}$$

$$q_{net} = 19853 \text{ psf}$$

Settlement

$$s = \frac{2}{9E_d} q_{net}\ B_o \left[\lambda_d \frac{B}{B_o}\right]^{\alpha} + \frac{\alpha}{9E_c} q_{net}\ \lambda_c\ B$$

$E/p_L \sim 7$, then from Fig. 9 $\rightarrow \alpha = 0.5$

$L/B = 2.2$, then from Fig. 10 $\rightarrow \lambda_c = 1.22$ and $\lambda_d = 1.58$

$$s = \frac{2}{9} \frac{1}{230{,}000} \times 19853 \times 2.0 \times (1.58 \times \frac{6.0}{2.0})^{0.5} + \frac{0.5}{9} \frac{1}{230{,}000} \times 19853 \times 1.22 \times 6.0$$

$s = .082 + .035 = .117 \text{ ft} > s_{all} = .082 \text{ ft}$

therefore, use $p_{all} = \frac{.082}{.117} \times 20428 = 14317$ psf

EXAMPLE 1c. SHALLOW FOOTING ON A CLAY (B.G. METHOD)

Bearing Capacity

$$q_L = k\, p_L^* + q_o$$

$p_{Le}^* = p_L^*$ $\qquad$ Ed = Ec $\qquad$ h = He = 5.0 ft

From Fig. 7, hard silty clay → soil category 2

$R = \frac{B}{2} = 3.0$ ft $\qquad \frac{He}{R} = 1.7 \qquad \frac{B}{L} = 0.5$

From Fig. 5: k(strip) = 1.0

k(square) = 1.09

Interpolating: $k(\frac{B}{L} = 0.46) = 1.04$

$$q_{safe} = \frac{1.04}{3} \times 30700 + (115 \times 5) = 11218 \text{ psf}$$

$$q_{net} = 10643 \text{ psf}$$

Settlement

$$s = \frac{2}{9E_d}\, q_{net} \left[\lambda_d \frac{B}{B_o}\right]^{\alpha} + \frac{\alpha}{9E_c}\, q_{net}\, \lambda_c\, B$$

$E/p_L \sim 7$, then from Fig. 9 → $\alpha = 0.5$

L/B = 2.2, then from Fig. 10 → $\lambda_c = 1.22$ and $\lambda_d = 1.58$

$$s = \frac{2}{9}\frac{1}{230{,}000} \times 10643 \times 2.0 \times (1.58\,\frac{6.0}{2.0})^{0.5} + \frac{0.5}{9 \times 230{,}000} \times$$

$$10643 \times 1.22 \times 6.0$$

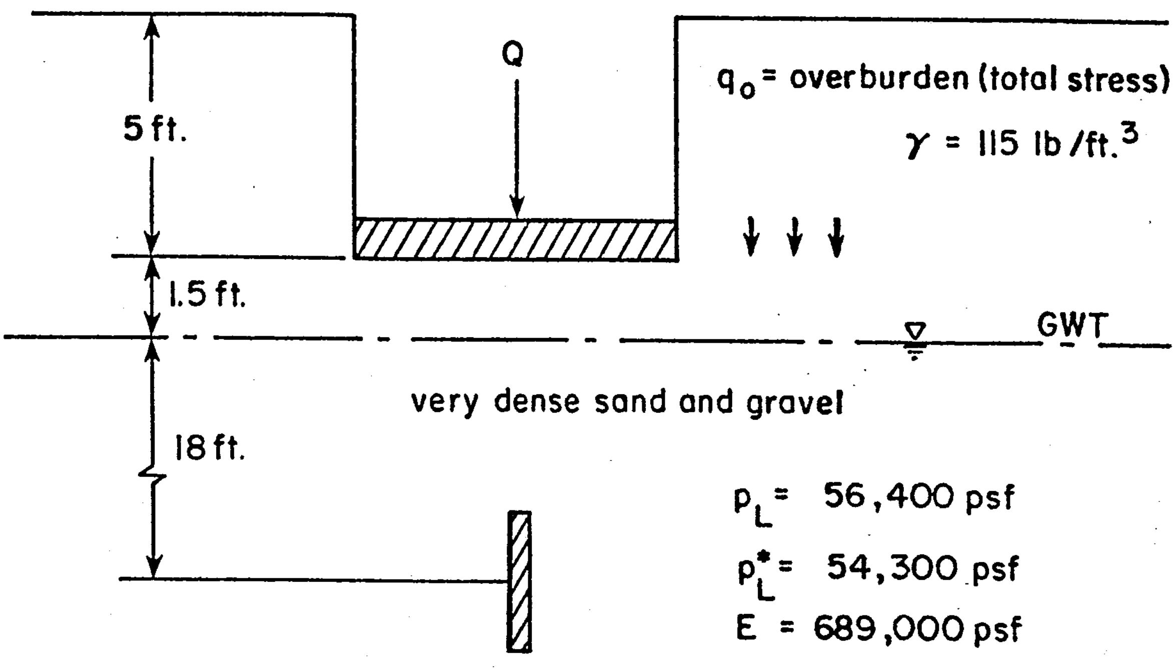
$s = .045 + .034 = .079$ ft $> s_{all} = .082$ ft

therefore, $p_{all} = q_{safe} = 11218$ psf

EXAMPLE PROBLEM 2:

RECTANGULAR FOOTING ON SAND

L = 13 ft.   B = 6 ft.



EXAMPLE 2a. SHALLOW FOOTING ON SAND (MENARD METHOD)

Bearing Capacity

$q_L = k\, p_L^* + q_o$

$p_{Le}^* = p_L^*$ Ed = Ec (homogenous soil)   He = h = 5.0 ft

From Fig. 6 → Soil Category III

$R = \frac{B}{2} = 3.0$ ft $\frac{He}{R} = 1.7$ $\frac{B}{L} = 0.46$

then from Fig. 3 k(strip) = 1.35

k(square) = 2.33

interpolating $k(\frac{B}{L} = 0.46) = 1.80$

$q_{safe} = \frac{1.80}{3} \times 54300 + (115 \times 5) = 33155$ psf

$q_{net} = q_{safe} - q_o = 32580$ psf

Settlement

$$s = \underbrace{\frac{2}{9E_d} q_{net}\, B_o \left[\lambda_d \frac{B}{B_o}\right]^{\alpha}}_{\text{deviatoric component}} + \underbrace{\frac{\alpha}{9E_c} q_{net}\, \lambda_c\, B}_{\text{spherical component}}$$

$\frac{E}{p_L} \sim 12$; then from Fig. 9: $\alpha = 0.33$

L/B = 2.2, then from Fig.10 : $\lambda_c = 1.22$ and $\lambda_d = 1.58$

$$s = \frac{2}{9}\ \frac{32580}{689,000} \times 2.0 \times \left(1.58\ \frac{6.0}{2.0}\right)^{0.33} + \frac{0.33}{9} \times \frac{32580}{689,000} \times$$

$$1.22 \times 6.0$$

$s = .035 + .013 = .048$ ft $< s_{all} = .082$ ft

therefore, use $p_{all} = q_{safe} = 33155$ psf

EXAMPLE 2b. SHALLOW FOOTING ON SAND (B.J.S. METHOD)

Bearing Capacity

$q_L = k\ p_L^* + q_o$

$p_{Le}^* = p_L$ $Ed = Ec$ $D = He = 5.0$ ft

From Fig. 4: Use chart for sand and gravel

$\frac{D}{B} = \frac{5.0}{6.0} = 0.83$ $p_L^* = 54300$

For a square footing: $p_L^* = 125300$ psf $\rightarrow k = 2.64$

$p_L^* = 41770$ psf $\rightarrow k = 2.55$

then for $p_L^* = 54300$ psf $\rightarrow k = 2.56$

Similarly, for a strip footing: $k = 1.43$

Interpolating with $\frac{B}{L} = 0.46 \rightarrow k = 1.95$

$q_{safe} = \frac{1.95}{3} \times 54300 + (115 \times 5) = 35870$ psf

$q_{net} = 35295$ psf

Settlement

$$s = \frac{2}{9E_d} q_{net} B_o \left(\lambda_d \frac{B}{B_o}\right)^{\alpha} + \frac{\alpha}{9E_c} q_{net} \lambda_c B$$

$E/p_L \sim 12$, From Fig. 9 $\rightarrow \alpha = 0.33$

$L/B = 2.2$, From Fig. 10 $\rightarrow \lambda_d = 1.58$ and $\lambda_c = 1.22$

$$s = \frac{2}{9} \frac{35295}{689,000} \times 2.0 \times \left(1.58 \frac{6.0}{2.0}\right)^{0.33} + \frac{0.33}{9} \frac{35295}{689,000} \times 1.22 \times 6.0$$

$s = .038 + .014 = .052$ ft

$s < s_{all}$, therefore, use $p_{all} = q_{safe} = 35870$ psf

EXAMPLE 2c. SHALLOW FOOTING ON SAND (B.G. METHOD)

Bearing Capacity

$$q_L = k\, p_L^* + q_o$$

$p_{Le}^* = p_L^*$ He = h = 5.0 ft   Ed = Ec

From Fig. 7 → Soil Category 3

$\frac{He}{R} = 1.7$ $\frac{B}{L} = 0.46$

From Fig. 5: k(square) = 1.21

k(strip) = 1.11

Interpolating: $k\ (\frac{B}{L} = 0.46) = 1.16$

$$q_{safe} = \frac{1.16}{3} \times 54300 + (115 \times 5) = 21571 \text{ psf}$$

$$q_{net} = 20996 \text{ psf}$$

Settlement

$$s = \frac{2}{9E_d} q_{net} B_o \left(\lambda_d \frac{B}{B_o}\right)^{\alpha} + \frac{\alpha}{9E_c} q_{net} \lambda_c B$$

$E/p_L \sim 12$, then from Fig. 9 → $\alpha = 0.33$

L/B = 2.2, then from Fig. 10 → $\lambda_d = 1.58$ and $\lambda_c = 1.22$

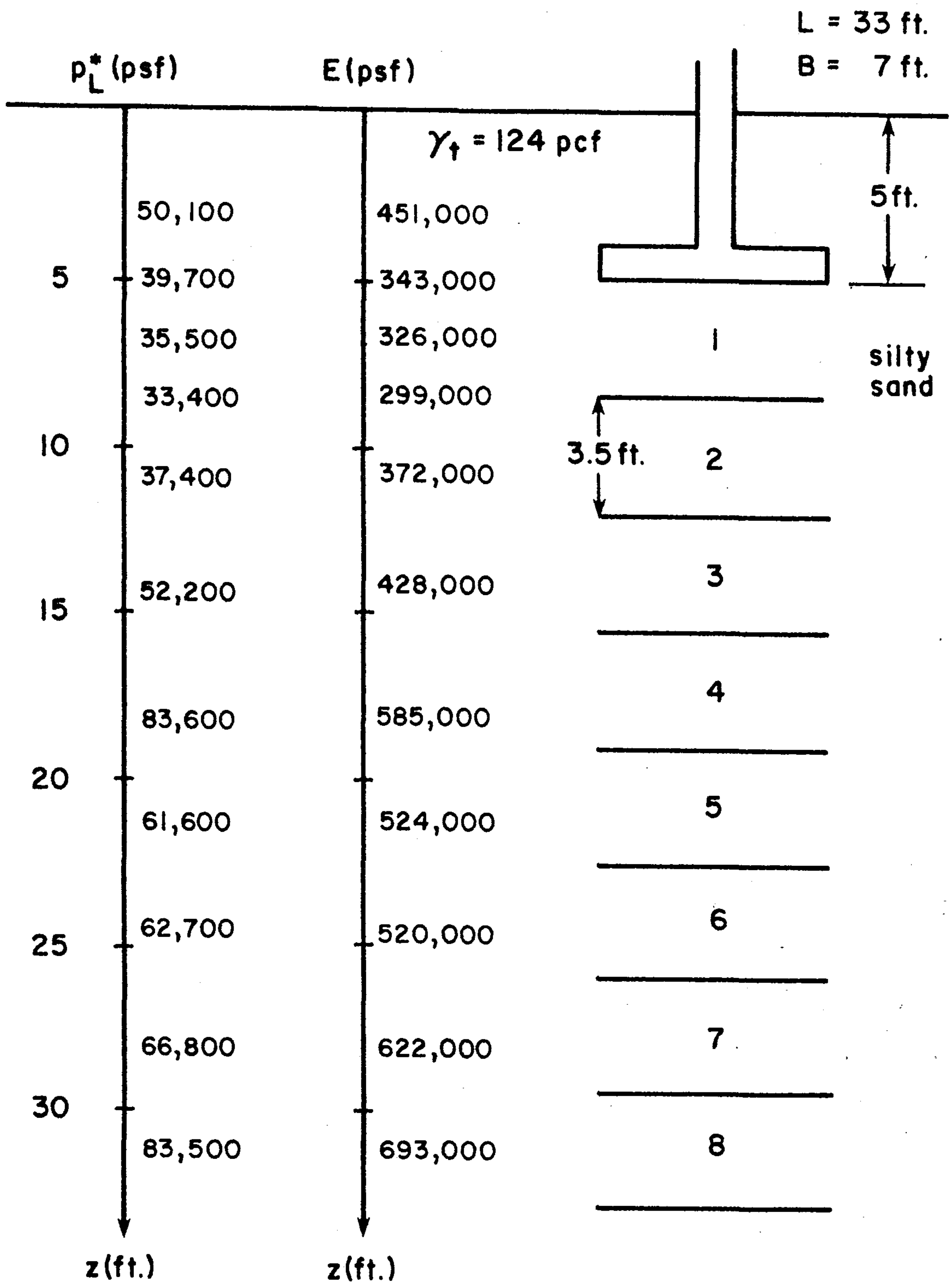
$$s = \frac{2}{9} \frac{20996}{689,000} \times 2.0 \times \left(1.58\, \frac{6.0}{2.0}\right)^{0.33} + \frac{0.33}{9} \frac{20996}{689,000} \times 1.22 \times 6.0$$

$s = .023 + .008 = .031$ ft

$s < s_{all}$, therefore, use $p_{all} = q_{safe} = 21571$ psf

EXAMPLE PROBLEM 3:

STRIP FOOTING ON SAND



EXAMPLE 3 STRIP FOOTING ON SAND

Bearing Capacity

$$q_L = k\ p^*_{Le} + q_o$$

where $p^*_{Li} = \left[\prod_{o}^{n} p_{Li}\right]^{1/n}$ ≡ equivalent net limit pressure

$$p^*_{L1} = (50100 \times 39700)^{1/2} = 44600$$

$$p^*_{L2} = (39700 \times 35500 \times 33400)^{1/3} = 35700$$

$$p^*_{L3} = 37400$$

$$p^*_{Le} = (44600 \times 35700 \times 37400)^{1/3} = 38600$$

$He = \frac{1}{p^*_{Le}} \sum_{1}^{n} (p^*_{Li} \times z_i)$ ≡ equivalent embedment depth where 1 - n are layers within the actual depth of embedment

$$He = \frac{1}{38600} (50100 \times 5.0) = 6.5 \text{ ft}$$

Determination of k, (Menard)

From Fig. 6, silty sand → soil category 1

$\frac{B}{L} = \frac{7.0}{33.0} = 0.21$ $\frac{he}{R} = \frac{6.5}{3.5} = 1.9$

From Fig. 3: k(strip) = 1.11

k(square) = 1.58

$k(\frac{B}{L} = 0.21) = 1.30$

$$q_{safe} = \frac{1.30}{3} \times 38600 + (124 \times 5) +17347 \text{ psf}$$

$q_{net} = 16727$ psf

Determination of k, (B.J.S.)

$\frac{D}{B} = \frac{5.0}{7.0} = 0.71$ $\frac{B}{L} = 0.21$

From Fig. 4, sand and gravel:

square footing → for $p_L^* = 41770$; $k = 2.4$

for $p_L^* = 20890$; $k = 2.0$

then, for $p_{Le}^* = 38600$; $k = 2.34$

Similarly, for a strip footing:

for $p_L^* = 41770$; $k = 1.35$

for $p_L^* = 8350$; $k = 1.10$

then, for $p_L^* = 38600$; $k = 1.31$

Interpolating with $\frac{B}{L} = 0.21$ → $k = 1.53$

$$q_{safe} = \frac{1.53}{3} \times 38600 + (124 \times 5) = 20306 \text{ psf}$$

$q_{net} = 19686$ psf

Determination of k, (B.G.)

He = 6.5 ft $\frac{He}{R} = \frac{6.5}{3.5} = 1.9$ $\frac{B}{L} = 0.21$

From Fig. 7, Soil Category 2

From Fig. 5: k(square) = 1.11

k(strip) = 1.0

$k(\frac{B}{L} = 0.21) = 1.02$

$q_{safe} = \frac{1.02}{3} \times 38600 + (124 \times 5) = 13744$ psf

$q_{net} = 13124$ psf

Settlement

$$s = \frac{2}{9E_d} q_n B_o \left(\lambda_d \frac{B}{B_o}\right)^{\alpha} + \frac{\alpha}{9E_c} q_n \lambda_c B$$

where $E_c \equiv$ harmonic mean of E's within layer 1

$E_d \equiv$ weighted average of E's from layers 1 - 16

$$E_c \rightarrow \frac{3}{E_c} = \frac{1}{343{,}000} + \frac{1}{326{,}000} + \frac{1}{299{,}000}$$

$E_c = 321{,}632$ psf

$E_d \rightarrow E_1 = E_c = 321{,}632$ psf

$$\frac{2}{E_2} = \frac{1}{299{,}000} + \frac{1}{372{,}000}$$

$E_2 = 331{,}529$ psf

$$\frac{3}{E_{3/4/5}} = \frac{1}{428{,}000} + \frac{1}{585{,}000} + \frac{1}{524{,}000}$$

$$\frac{3}{E_{3/4/5}} = 503{,}842 \text{ psf}$$

$$\frac{3}{E_{6/7/8}} = \frac{1}{520{,}000} + \frac{1}{622{,}000} + \frac{1}{693{,}000}$$

$$E_{6/7/8} = 603{,}161 \text{ psf}$$

$E_{9/16}$ is taken as equal to $E_{6/7/8}$. This is conservative since modulus increases with depth.

$$\frac{4}{E_d} = \frac{1}{321{,}632} + \frac{1}{(0.85)(331{,}529)} + \frac{1}{503{,}842} + \frac{1}{(2.5)(603{,}161)} + \frac{1}{(2.5)(603{,}161}$$

$$E_d = 401{,}250 \text{ psf}$$

Note: (0.85) and (2.5) are weighing coefficients used to indicate the relative importance of the depth to the soil layers in question.

$$\frac{E_d}{p^*_{Le}} = \frac{401{,}250}{38600} \sim 10$$

From Fig. 9 → $\alpha_d = 0.33$

$$\frac{E_c}{p^*_{Le}} = \frac{321{,}632}{38600} \sim 8$$

From Fig. 9 → $\alpha_c = 0.33$

$\frac{L}{B} = \frac{33}{7} = 4.7$ From Fig. 10 → $\lambda_d = 2.09$ and $\lambda_c = 1.38$

Settlement with Menard q_{net}:

$$s = \frac{2}{9} = \frac{16727}{401{,}250} (2.0) \left(2.09 \frac{7.0}{2.0}\right)^{0.33} + \frac{0.33}{9} \frac{16727}{321{,}632} (1.38) (7.0)$$

$$s = .036 + .018 = .054 \text{ ft}$$

Settlement with B.J.S. q_{net}:

$$s = \frac{2}{9} \frac{19686}{401,250} (2.0) \left(2.09 \frac{7.0}{2.0}\right)^{0.33} + \frac{0.33}{9} \frac{19686}{321,632} (1.38) (7.0)$$

$$s = .042 + .021 = .063 \text{ ft}$$

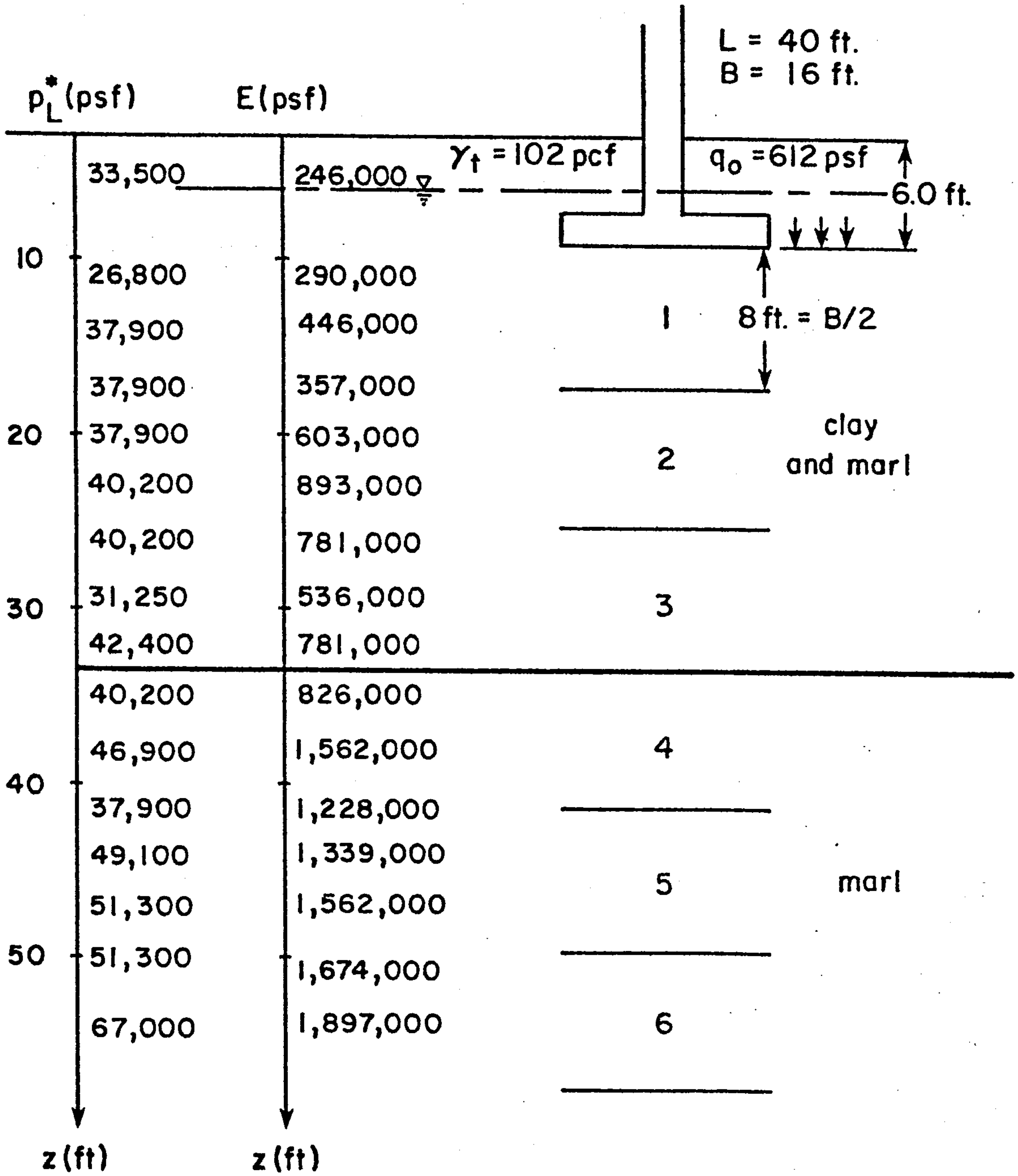
Settlement with B.G. q_{net}:

$$s = \frac{2}{9} \frac{13124}{401,250} (2.0) \left(2.09 \frac{7.0}{2.0}\right)^{0.33} + \frac{0.33}{9} \frac{13124}{321,632} (1.38) (7.0)$$

$$s = .028 + .014 = .042 \text{ ft}$$

EXAMPLE PROBLEM 4:

RECTANGULAR FOOTING ON LAYERED SOIL



EXAMPLE 4. RECTANGULAR FOOTING ON A LAYERED SOIL (MENARD METHOD)

Bearing Capacity

$$q_L = k\, p^*_{Le} + q_o$$

where: $p^*_{Le} = (p^*_{L1} \times p^*_{L2} \times p^*_{L3})^{\frac{1}{3}}$

$p^*_{L1} = 33500$ psf

$p^*_{L2} = (26800 \times 37900 \times 37900)^{\frac{1}{3}}$

$p^*_{L2} = 33415$ psf

$p^*_{L3} = (37900 \times 40200)^{\frac{1}{2}}$

$p^*_{L3} = 39033$ psf

$p^*_{Le} = (33500 \times 33415 \times 39033)^{\frac{1}{3}}$

so that: $p^*_{Le} = 34855$ psf

$$He = \frac{1}{p^*_{Le}} \sum_{1}^{n} (p^*_{L1} \times \Delta z_i) = \frac{1}{34855}\ 33500 \times 6.0$$

$He = 5.77$ ft

From Fig. 6 → Soil Category II

$\frac{B}{L} = \frac{16}{40} = 0.40$ $\qquad$ $\frac{He}{R} = \frac{5.77}{8.00} = 0.72$

From Fig 3 → k(square) = 1.32

k(strip) = 1.00

From Fig. 3 → $k(\frac{B}{L} = 0.40) = 1.13$

$$q_{safe} = \frac{1.13}{3} \times 34855 + (6.0 \times 102) = 13741 \text{ psf}$$

$$q_{net} = 13129 \text{ psf}$$

Settlement

$$s = \frac{2}{9E_d} q_{net} B_o \left(\lambda_d \frac{B}{B_o} \right)^{\alpha} + \frac{\alpha}{9E_c} q_{net} \lambda_c B$$

E_c = harmonic mean of E's within layer 1

$$\frac{3}{E_c} = \frac{1}{290{,}000} + \frac{1}{446{,}000} + \frac{1}{357{,}000}$$

$$E_c = 353{,}292 \text{ psf}$$

E_d = weighted harmonic mean of E's from layers 1 - 16

$$E_1 = E_c = 353{,}292 \text{ psf}$$

$$\frac{2}{E_2} = \frac{1}{603{,}000} + \frac{1}{893{,}000}$$

$$E_2 = 719{,}892 \text{ psf}$$

$$\frac{8}{E_{3/4/5}} = \frac{1}{781{,}000} + \frac{1}{536{,}000} + \frac{1}{781{,}000} + \frac{1}{826{,}000} + \frac{1}{1{,}562{,}000} + \frac{1}{1{,}228{,}000} + \frac{1}{1{,}339{,}000} + \frac{1}{1{,}562{,}000}$$

$$E_{3/4/5} = 943{,}539 \text{ psf}$$

$$\frac{2}{E_6} = \frac{1}{1{,}674{,}000} + \frac{1}{1{,}897{,}000}$$

$$E_6 = 1{,}778{,}537 \text{ psf}$$

$E_{6/8}$ and $E_{9/16}$ are taken equal to E_6 since no deeper modulus data is available. This assumption is conservative if the modulus continues to increase with depth.

$$\frac{4}{E_d} = \frac{1}{353{,}292} + \frac{1}{(0.85)(719{,}892)} + \frac{1}{943{,}539} + \frac{1}{(2.5)(1{,}778{,}537)} + \frac{1}{(2.5)(1{,}778{,}537)}$$

E_d = 669,523 psf

$$\frac{Ed}{p^*_{Le}} = \frac{669{,}523}{34855} \sim 19; \text{ From Fig. 9} \rightarrow \alpha_d = 1.0$$

$$\frac{Ec}{p^*_{Le}} = \frac{353{,}292}{34855} \sim 10; \text{ From Fig. 9} \rightarrow \alpha_c = 0.67$$

$$\frac{L}{B} = \frac{40}{16} = 2.5; \text{ From Fig. 10} \rightarrow \lambda_d = 1.82 \text{ and } \lambda_c = 1.25$$

$$s = \frac{2}{9}\,\frac{34855}{669{,}523}\,(2.0)\,\left(1.82\,\frac{16}{2.0}\right)^{1.0} + \frac{0.67}{9}\,\frac{34855}{353{,}292}\,(1.25)(16)$$

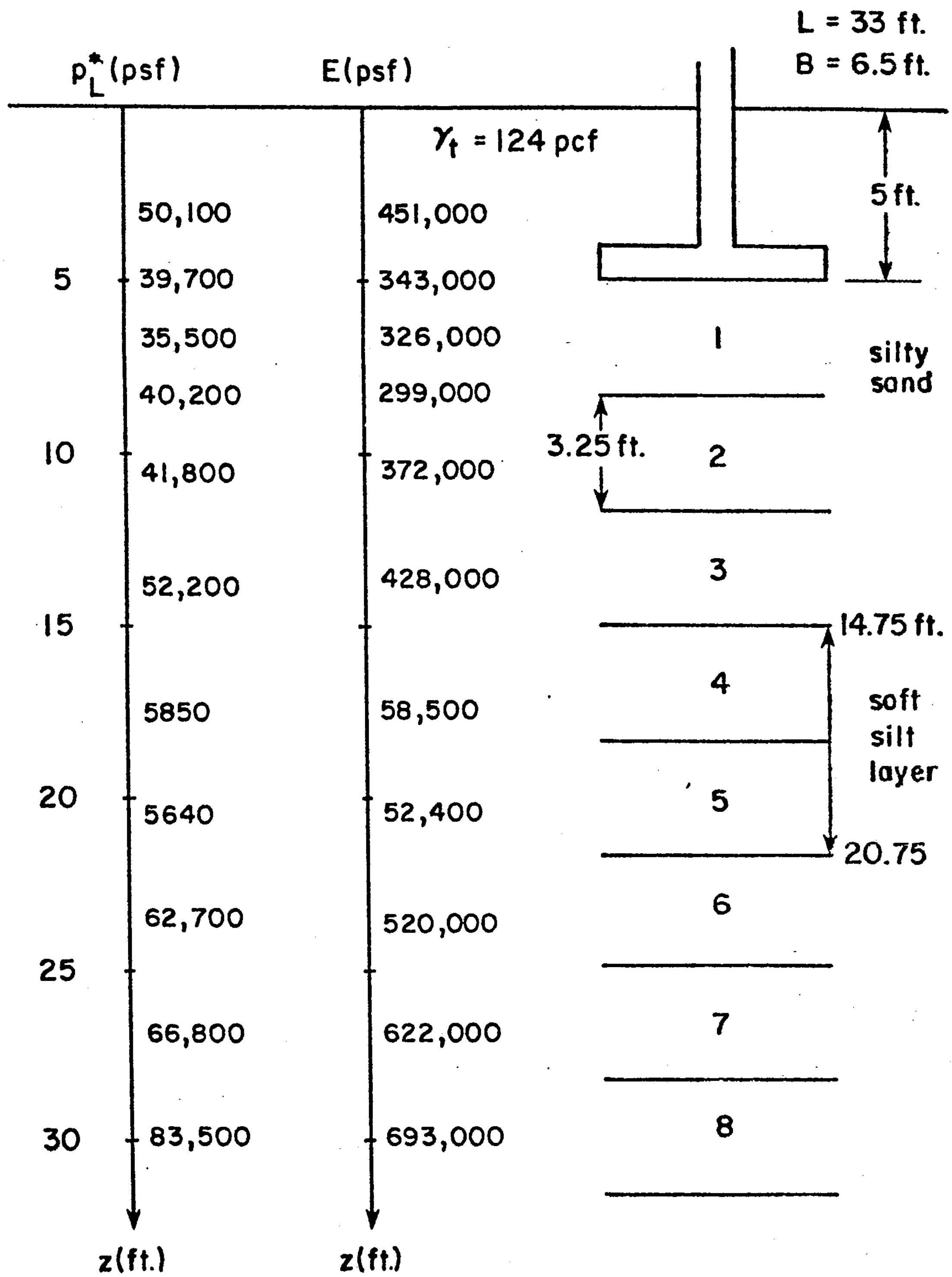
s = .337 + .147 = .484 ft

Recommend bearing pressure: $p_{all} = \frac{.082}{.484} \times 13741$

p_{all} = 2328 psf

EXAMPLE PROBLEM 5:

STRIP FOOTING WITH SOFT LAYER AT DEPTH



EXAMPLE 5. STRIP FOOTING ON A SOFT LAYER AT DEPTH

Bearing Capacity

Estimate: $q_{safe} = \frac{1}{3} k\, p^*_{Le} + q_o$

$$q_{net} = q_{safe} - q_o = \frac{1}{3} k\, p^*_{Le}$$

Silty sand, from Fig. 7 → Soil Category 2

$\frac{B}{L} = \frac{6.5}{33.0} = 0.2$ $\frac{He}{R} = \frac{5.0}{3.25} = 1.54$

Now, from Fig. 5 → k(square) = 1.06

k(strip) = 1.02

$k(\frac{B}{L} = 0.2) = 1.03$

Assume that p^*_{Le} is probably controlled by weak layer. (This is conservatively false.)

$$p^*_{Le} = (5850 \times 5640)^{1/2} = 5744 \text{ psf}$$

$$q_{safe} = \frac{1}{3} (1.03)\ (5744) + (5 \times 124) = 2592 \text{ psf}$$

$$q_{net} = 1972 \text{ psf}$$

Settlement

Here: $s_T = s' + s''$

Where: $s' = \frac{2}{9E_d} q_n B_o \left(\lambda_d \frac{B}{B_o}\right)^{\alpha} + \frac{\alpha}{9E_c} q_n \lambda_c B$

$$s'' = \alpha \left(\frac{1}{E_c} - \frac{1}{E_m}\right) \Delta p_c \; H$$

From Example 3, Menard method yields s' = .054 ft

Now consider the softness of the silt layer:

E_c = pressuremeter modulus of soft layer

$$\frac{2}{E_c} = \frac{1}{58500} + \frac{1}{52400}$$

E_c = 55282 psf

$E_m = E_d$ = 401,250 psf (from Example 3)

Note: $\frac{1}{E_c} - \frac{1}{E_m}$ is a measure of the different hardnesses of the soil layers in question.

From Boussinesq theory and Newmark's chart, the vertical stresses at the upper and lower surfaces of the soft layer have changed by:

$z = 14.75 \text{ ft} \rightarrow \Delta\sigma_v = 0.24\ q_{net}$

$z = 20.75 \text{ ft} \rightarrow \Delta\sigma_v = 0.17\ q_{net}$

q_{net} = 1972 psf $(\Delta\sigma_v)$ at 14.75 ft = 473 psf

$(\Delta\sigma_v)$ at 20.75 ft = 335 psf

Average Δp = (473 + 335)/2 = 404 pst

$$s'' = \alpha \left(\frac{1}{E_c} - \frac{1}{E_m}\right) \Delta p \; H$$

$\frac{E_c}{p_L^*} = \frac{55282}{5744} \sim 10$, From Fig. 9, silt $\rightarrow \alpha = 0.5$

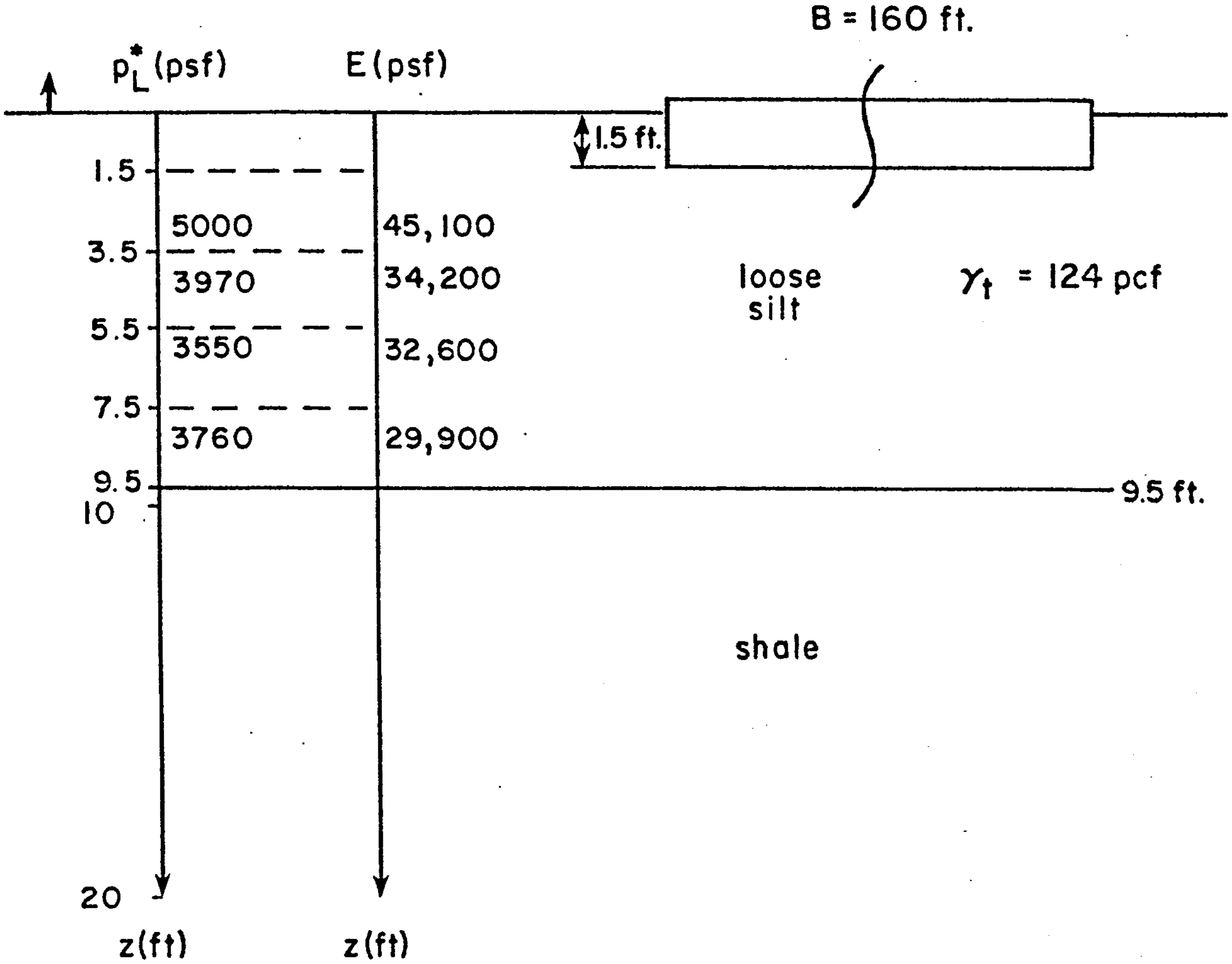
$$s'' = 0.5 \left(\frac{1}{55282} - \frac{1}{401,250}\right)(404)(20.75 - 14.75)$$

s" = 0.018'

thus, s_T = 0.054' + 0.018' = 0.072'

EXAMPLE PROBLEM 6:

MAT FOUNDATION ON A SOFT LAYER CLOSE TO GROUND SURFACE



EXAMPLE 6. MAT FOUNDATION ON A SOFT LAYER

Bearing Capacity

Estimate $q_{safe} = \frac{1}{3} k\, p^*_{Le} + q_o$

$$q_{net} = \frac{1}{3} k\, p^*_{Le}$$

Since $\frac{He}{R} \sim 0 \rightarrow k = 0.8$

Assume that the silt layer controls bearing capacity; then let:

$p^*_{Le} \equiv$ average of the compressible layer

$$p^*_{Le} = (5000 \times 3970 \times 3550 \times 3760)^{\frac{1}{4}} = 4035 \text{ psf}$$

$$q_{safe} = (\tfrac{1}{3} \times 0.8 \times 4035) + (124 \times 1.5) = 1262 \text{ psf}$$

$$q_{net} = 1076 \text{ psf}$$

Settlement

For a wide foundation underlain by a soft layer (i.e. relatively thin, soft layer)

$$s = \int_o^h \frac{\alpha(z)\ B(F)\ p(z)}{E(z)}\, dz = \sum_1^n \frac{\alpha_i\ B_i\ p_i}{E_i}\, \Delta z_i$$

For silt layer:

$\frac{E}{p^*_L} \sim 9$, From Fig. 9 $\rightarrow \alpha = 0.5$

$F \equiv$ safety factor $= \frac{k\, p^*_{Le}}{q_{net}}$, where $k = 0.8$

$$F = (0.8)\,\frac{4035}{1076} = 3.0$$

$$\text{thus, } B(F) = \frac{2}{3}\left(\frac{F}{F-1}\right) = \frac{2}{3}\left(\frac{3.0}{3.0-1.0}\right) = 1.0$$

Assume that Δp_v due to foundation loading is equal to actual foundation pressure since layers of silt are thin compared to foundation.

$$\text{Then:} \quad s = \Sigma\,\frac{\alpha B q_n}{E}\,\Delta z = \alpha B q_n\,\Sigma\left(\frac{\Delta z}{E}\right)$$

$$s = (0.5)(1.0)(1.076\ \text{psf})\left(\frac{(3.5-1.5)}{45100} + \frac{2.0}{34200} + \frac{2.0}{32600} + \frac{2.0}{29900}\right)$$

$$s = 0.124\ \text{ft}$$

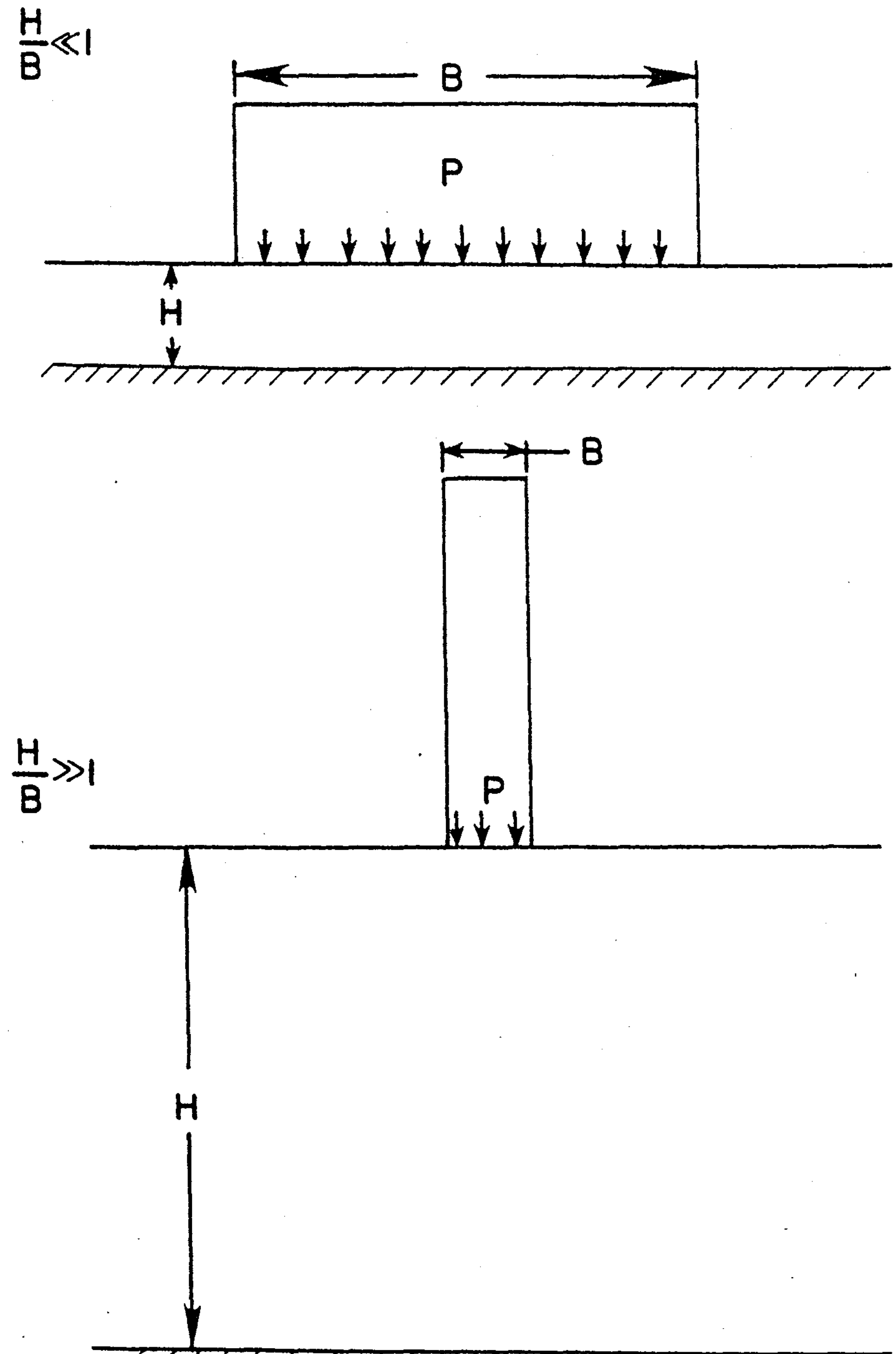
CHAPTER 5. - COMPARISON BETWEEN PREDICTED AND MEASURED BEHAVIOR

It has been shown in 3.1.1 that when the width of the footing (B) is small compared to the depth of the deposit (H), the major part of the settlement is induced by the deviatoric tensor (very little consolidation settlement). In section 3.1.6 and 3.1.7 special steps were taken to deal with the cases where the width of the foundation (B) is large compared to the thickness of the compressible layer (H); in this case the major part of the settlement is due to consolidation (Figure 13). As a result, the pressuremeter approach to settlement of shallow foundations is recommended when H/B is large (2 or more); otherwise the pressuremeter approach must be complemented by conventional consolidation tests.

Numerous comparisons of predicted versus measured settlement have been made with the pressuremeter approach (1); they are presented in Figures 14 through 16.

Experimental evidence for the bearing capacity factor k (shallow foundation) can be found in references 6, 7, 8, and 1. The experimental results are presented in Figure 17.

Figure 17 shows the design bearing capacity curves for the B.G. method and the actual data points found through experiment by Menard (1). In the investigation by Menard, the ultimate bearing capacity was considered to be the pressure at a footing penetration of 1.6 inches. The design curves shown on Figure 17 are the design curves of Bustamante and Gianeselli (Figure 7).



$$S = \frac{2}{9}\,\frac{1}{E_d}\,q\,B_o\left(\lambda_d\,\frac{B}{B_o}\right)^{\alpha} + \frac{\alpha}{9E_c}\,q\,\lambda_c$$

FIGURE 13: Pressuremeter Settlement Concepts

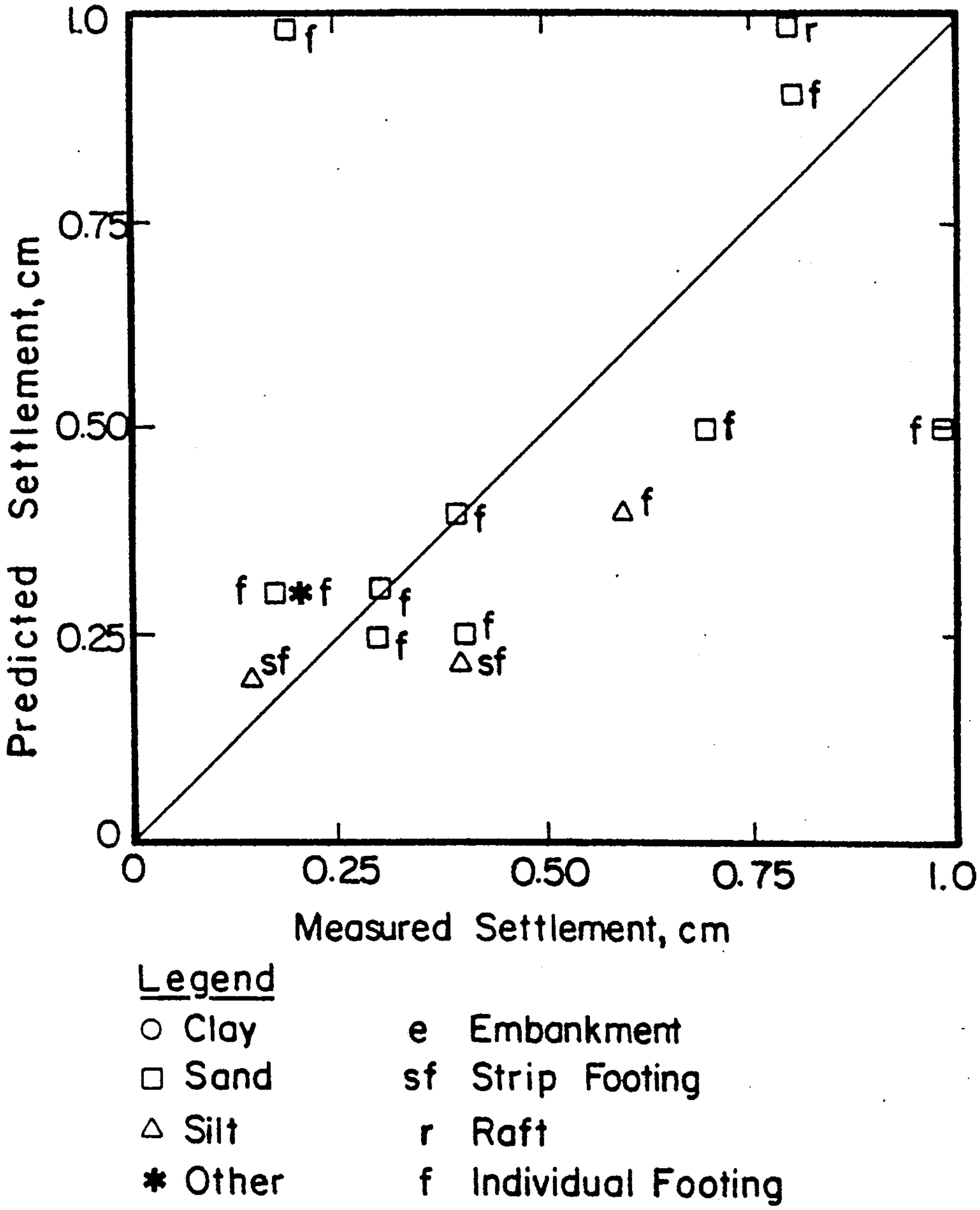


FIGURE 14: PREDICTED VERSUS MEASURED SETTLEMENT (Very Small Settlement)

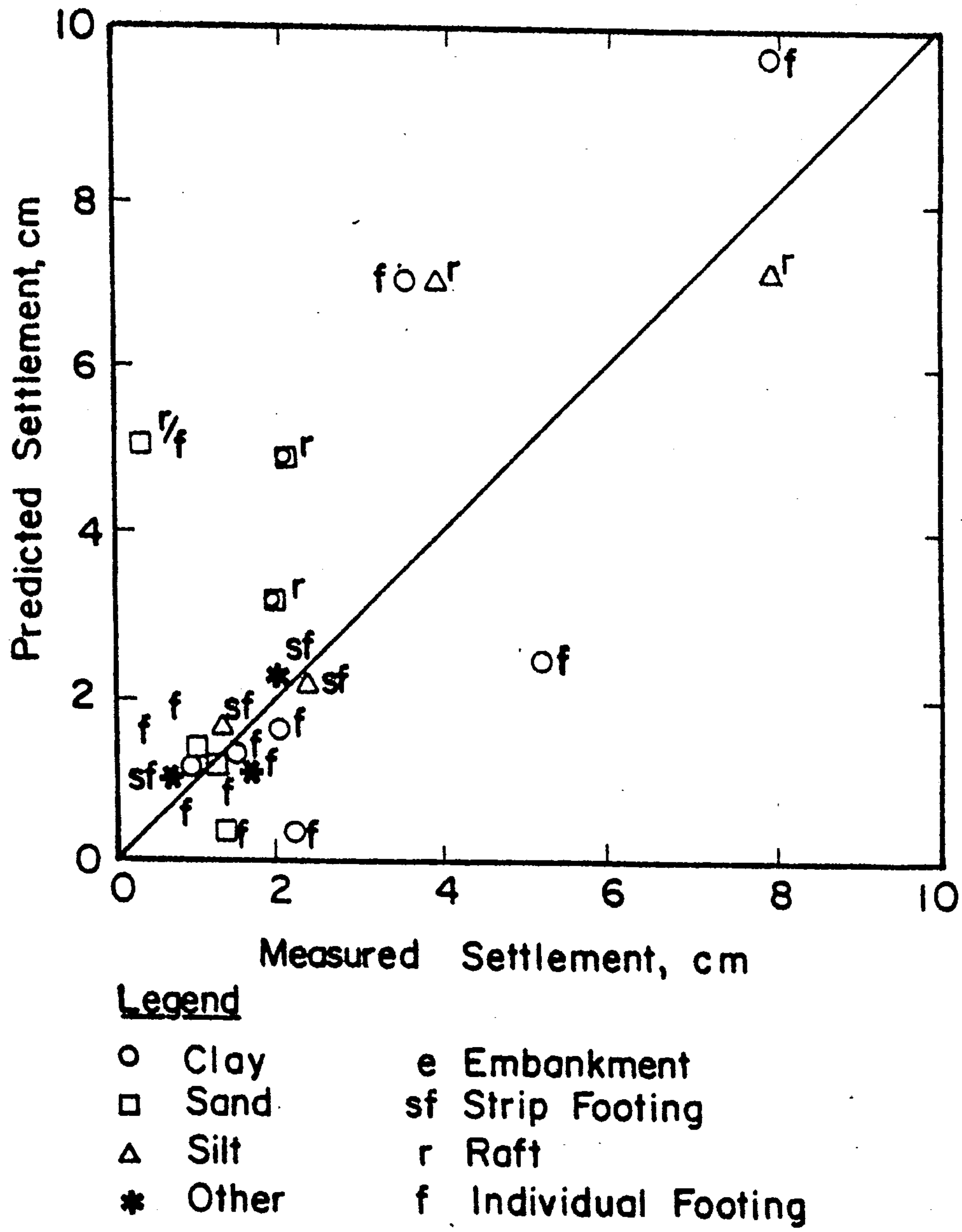


FIGURE 15 : PREDICTED VERSUS MEASURED SETTLEMENT (Moderate Settlement)

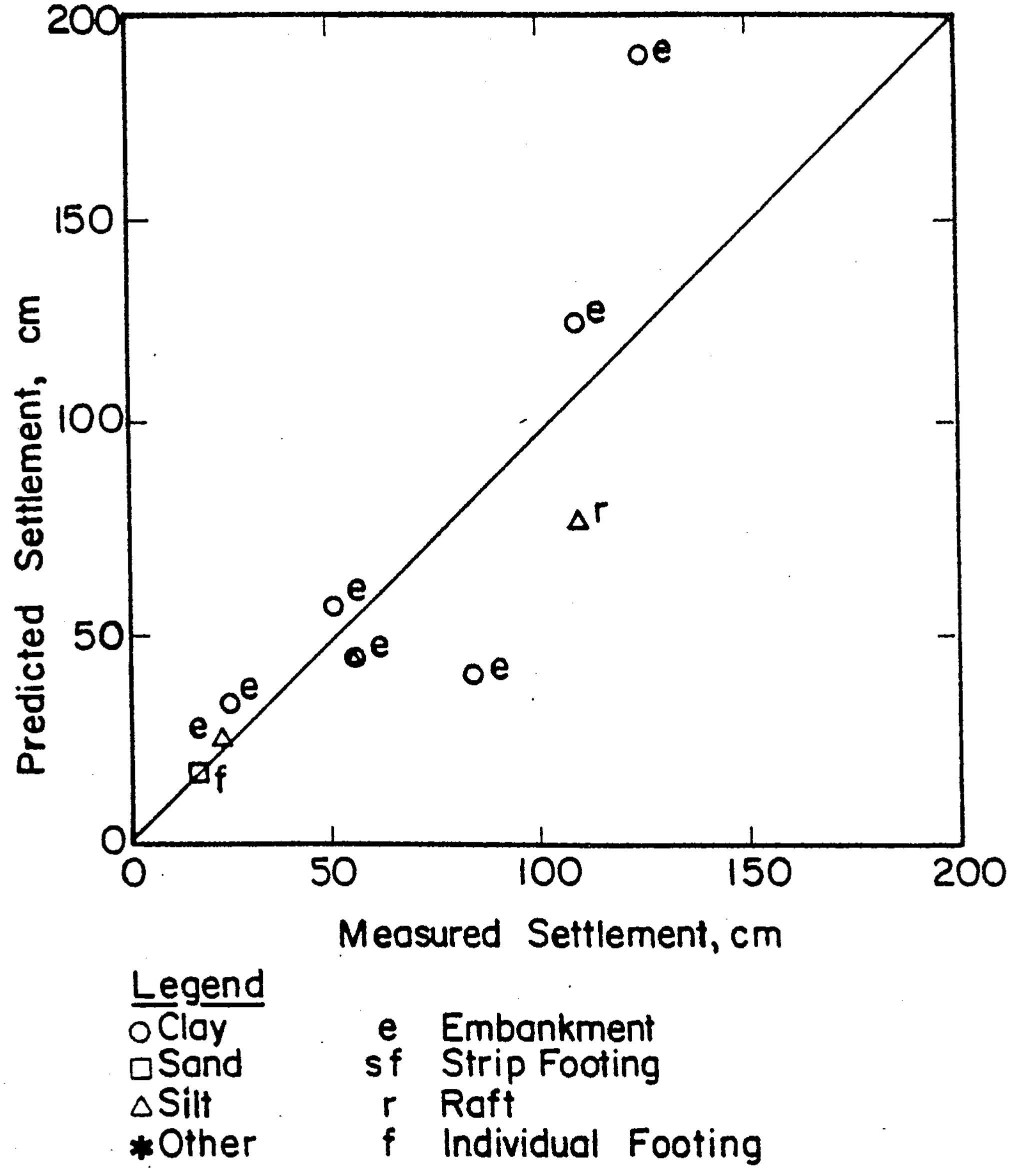


FIGURE 16 : PREDICTED VERSUS MEASURED SETTLEMENT
(Large Settlement)

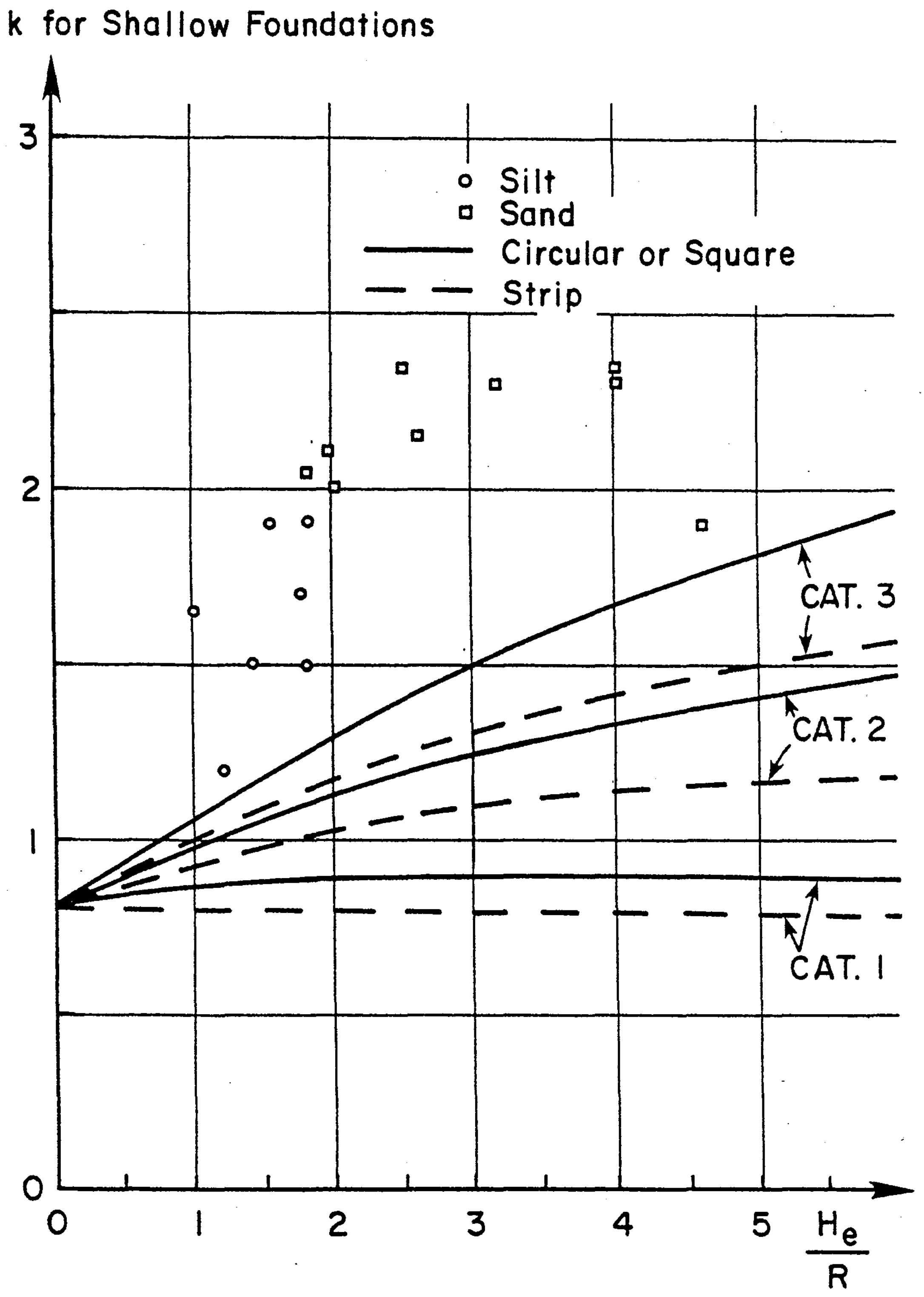


FIGURE 17 : Comparison of the Bearing Capacity Factors Predicted by the B.G. Method and Measured by Menard (References 1, 6, 7, 8).

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