AN EXAMINATION OF THE INFLUENCE OF FOUNDATION SIZE ON ULTIMATE BEARING CAPACITY AND SETTLEMENT

by 1264

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I. INTRODUCTION

1. Statement of the Problem:

As is well known every foundation problem necessitates two different studies; one concerning the ultimate bearing capacity with regard to shear of the soil under the considered foundation, the second concerning the limit of settlements. There are many factors which will influence the ultimate bearing capacity and allowable settlement. Among these factors, the foundation size is extremely important. As a consequence, in 1948, Taylor (1) Observed that "The size of footing is a variable that might, from intuition alone, be expected to have important effect on the bearing capacity," he also requires "a true understanding of the effects of size on both the ultimate bearing capacity and the coefficient of settlement."

In 1957, Hough(2) pointed out that "Settlement due to soil compression under a given loading intensity is greater for large footings than for small footings.

In 1962, Balla(3) also said that "The bearing capacity factors do not depend solely on the angle of shearing resistance, but also on other propertles of soil (cohesion,density) and characteristic dimensions of the foundation (depth, width). The width of the foundation has a greater influence on the value of the bearing capacity."

In view of the above considerations, an examination of the influence of foundation size on bearing capacity and settlement is one of the most significant and, from the view-point of practical application, very important questions in the field of foundation engineering.

2. Purpose of the Study:

Foundation problems have grown rapidly during the last several years. The practical applications have become more numerous and the amount of existing soil mechanics literature has increased rapidly, but it is probable that no single book has served for this specific problem. The purpose of this study is to show the size effect of shallow foundations on the bearing capacity and settlement.

3. Scope of the Study:

The heart of the report is part II and part III. Part II consists of careful and adequate review of the important literature on ultimate bearing capacity and settlement and will especially include reviews of experience, theory and test results. Part III deals with the writer's conclusions based on the literature review.

II. REVIEW OF THE LITERATURE

A. Ultimate Bearing Capacity:

The conventional method of foundation design is based on the concept of bearing capacity, or allowable bearing pressure of the soil. In order to be able to provide an adequate factor of safety against foundation collapse, the so-called Ultimate Bearing Capacity must be known.

Review of Bearing Capacity Analysis:

(a). The Terzaghi Solution for Bearing Capacity:

Terzaghi (4) using his shallow foundation definition derived a general solution for ultimate bearing capacity. This method contains various assumptions. Figure (1) shows "If the soil is fairly dense or stiff, the settlement curve is similar to curve C_1," which was designated by Terzaghi as the General Shear curve, the abscissa q is representing the bearing capacity of the soil. "If the soil is loose or fairly soft, the settlement curve may be similar to C_2," which was designated by Terzaghi as the local Shear curve, in this case, the bearing capacity is not always well defined. Terzaghi states "We specify arbitrarily, but in accordance with current conceptions, that the earth support has failed as soon as the curve passes into a steep and fairly straight tangent". Therefore, the bearing capacity is assumed to be the abscissa q' of the point at which the settlement curve becomes steep and straight.

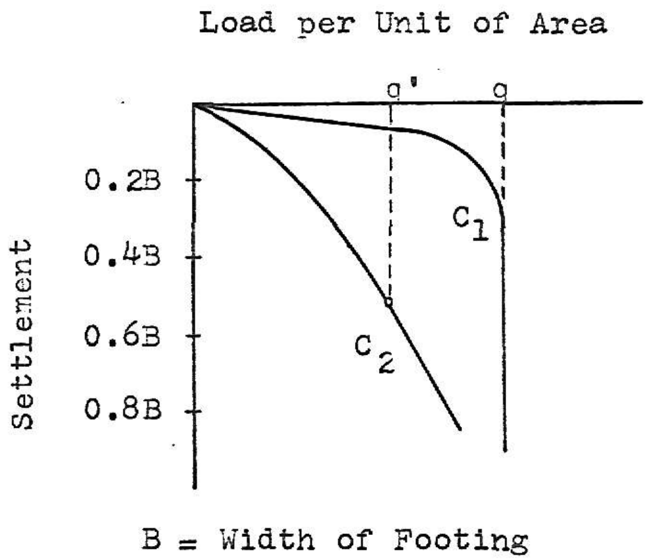
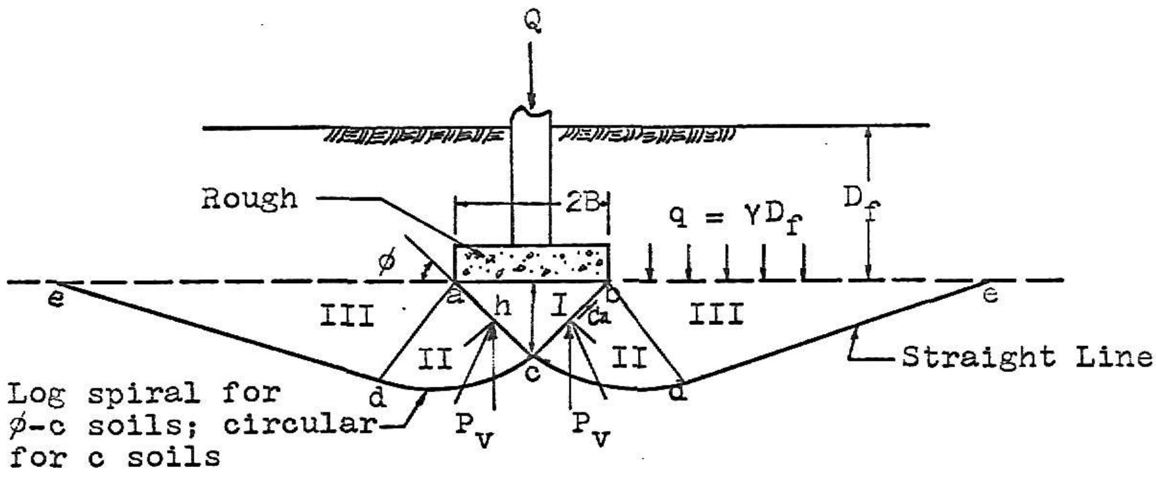


Fig. 1. Relation between intensity of Load and settlement of a footing



(a)

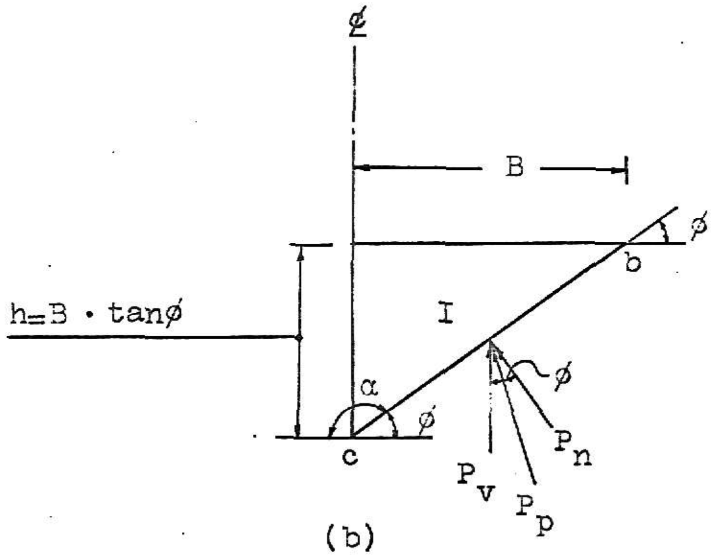


Fig. 2. Assumptions for the Terzaghi Bearing Capacity

Figure (2) illustrates the basic principle of the Terzaghi bearing capacity theory. When a load Q is applied centrally on the footing, the footing undergoes a certain amount of elastic and plastic settlement. An increase in the load Q tends to push the footing down, and a mass of soil within the triangle abc also moves down with the footing. This downward movement is resisted by the weight of the soil in the sliding wedges acde and bcde and by the shear resistance of the soil along the slip surface cde. For each set of assumed slip surface, we can compute the corresponding load Q that is necessary to cause the failure.

To compute the ultimate bearing capacity of a shallow continuous footing, Terzaghi neglected the shearing resistance of the soil above the horizontal plane through the base of the footing and replaced this with the surcharge, ($q = \gamma D_f$), and assumed that the shear strength can be described by Coulomb's equation. The shearing resistance of the soil given by this equation is;

$$s = c + \sigma \tan \phi \text{ (Coulomb Mohr's theory of rupture)}$$

where

c = cohesion

σ = the normal stress on shear plane

ϕ = the angle of internal friction

In applying this approach the shearing stresses at the contact face ac are

$$p_t = c + P_n \tan\phi$$

p_n is the normal component of the passive earth pressure per unit of area of the contact face.

The passive earth pressure on each one of these contact face(ac or bc) consists of two components, P_v and the adhesion C_a;

$$C_a = \frac{B}{\cos \phi} \cdot c$$

The equilibrium within the zone abc of the elastic equilibrium requires that the sum of the vertical forces including the weight ($W = \frac{1}{2} \cdot 2B \cdot B\tan\phi \cdot \gamma = \gamma B^2 \tan\phi$) of the earth in the zone should be equal to zero, such that

$$\Sigma Fv = 0$$

$$Q + W - 2Pv - 2Ca \sin\phi = 0$$

$$Q + \gamma B^2 \tan\phi - 2Pv - 2Bc\tan\phi = 0$$

$$Q = 2Pv + 2Bc\tan\phi - \gamma B^2 \tan\phi \qquad (1)$$

Where

$$Pv = \frac{Pn}{\cos\phi} = \text{total passive earth pressure on the contact face} \qquad (2)$$

and,

$$Pn^* = \frac{h}{\sin\alpha}(cKpc + qKpq) + 1/2\gamma h^2 \frac{Kp\gamma}{\sin\alpha} \qquad (3)$$

Where

Pn is the normal component of the passive pressure.

α is the slope angle of the contact face.

Kpc, Kpq, $Kp\gamma$ are coefficients.

If ac in Figure (2a) represents the contact face the values h, α, and δ contained in equation (3) are equal to

$$h = B\tan\phi, \quad \alpha = 180 - \phi, \quad \delta = \phi$$

Then, from equation (2) and (3) we get

$$Pv = \frac{Pn}{\cos\phi} = \frac{B\tan\phi}{\cos\phi \sin\phi} \cdot (cKpc + qKpq) + 1/2 \cdot \frac{\gamma B^2 \tan^2\phi}{\cos\phi} \cdot \frac{Kp\gamma}{\sin\phi}$$

*; For derivation see Appendix A1.

$$= \frac{B}{\cos^2\phi}(cK_{pc} + qK_{pq}) + 1/2 \cdot \gamma \cdot B \cdot \frac{\tan\phi}{\cos\phi} \cdot K_{p\gamma} \tag{4}$$

Combining equation (4) and (1) we get

$$Q = 2P_v + 2Bc\tan\phi - \gamma B^2 \tan\phi$$

$$Q = \frac{2B}{\cos^2\phi}(cK_{pc} + qk_{pq}) + rB^2 \frac{\tan\phi}{\cos^2\phi} K_{p\gamma} + 2Bc\tan\phi - \gamma B^2 \tan\phi$$

$$Q = 2Bc(\frac{K_{pc}}{\cos^2\phi} + \tan\phi) + 2Bq\frac{K_{pq}}{\cos^2\phi} + \gamma B^2 \tan\phi(\frac{K_{p\gamma}}{\cos^2\phi} - 1)$$

$$Q = 2BcNc + 2BqNq + 2B \cdot \gamma \cdot B\ N_\gamma$$

Sunstituting $q = \gamma D_f$, Thus

$$Q = 2B\ (cNc + \gamma D_f Nq + \gamma BN_\gamma) \tag{5}$$

Equation (5) is called Terzaghi general ultimate bearing capacity formula. The equation is valid on the condition that the soil support fails by general shear.

In local shear failure the shear resistance of soil is not mobilised along the full length of the failure surface of Fig. (2a). No analytical solution for this condition has been obtained. Terzaghi has suggested emperical reductions to the actual cohesion and angle of shearing resistance in case of local shear failure as follows:

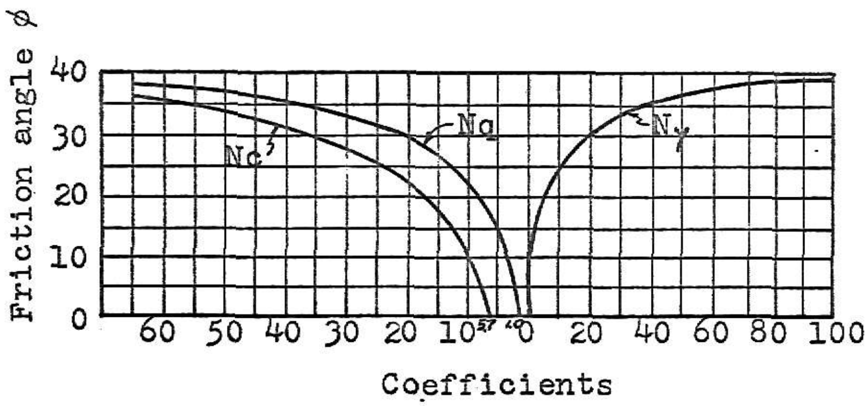
$$c' = 2/3c$$

$\tan\phi' = 2/3 \tan\phi$

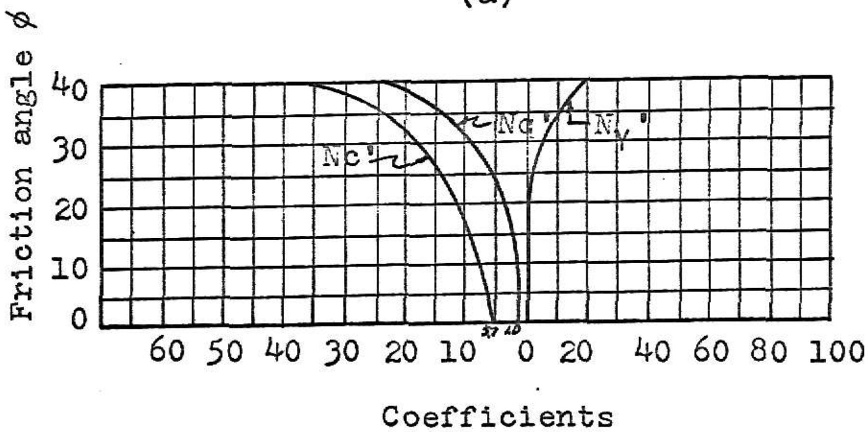
Therefore, the critical load Q' is equal to the sum

$$Q' = 2B\,(2/3\ cN_c' + \gamma D_f N_q' + \gamma B N_\gamma') \qquad (6)$$

The coefficients Nc, Nq, N_γ, and N'c, N'q, N'_γ are called the bearing capacity factors, their values are given in the curves of Figure (3a) and Figure (3b) respectively.



(a)

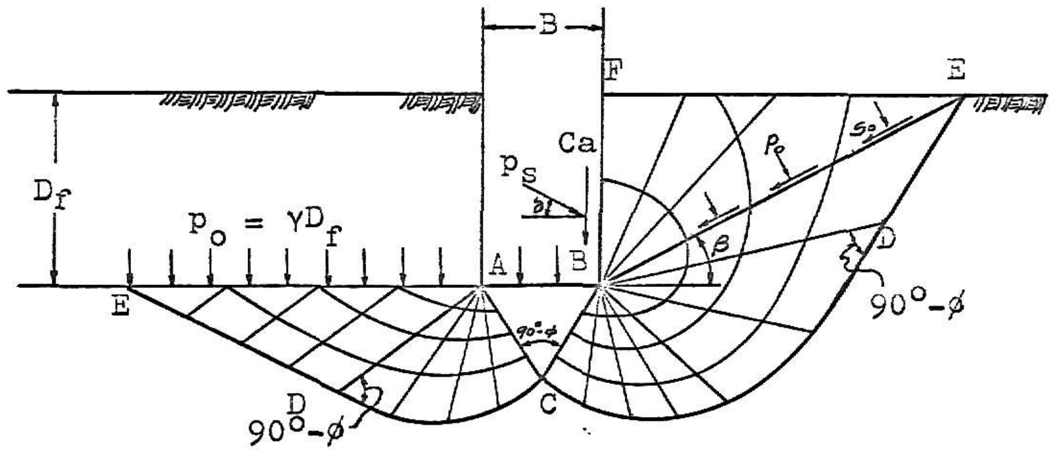


(b)

Fig. 3. Coefficients of the Terzaghi expressions for ultimate bearing capacity

(b). The Meyerhof Solution for Bearing Capacity;

The above analysis of Terzaghi's solution is based on plastic theory and the corresponding zones of plastic equilibrium in the material are shown in Figure (4a) for the case of a rough foundation. Below the base is a central zone ABC which remains in an elastic state of equilibrium,on



(a) Previous theory

(b) Present theory

Fig. 4. Plastic Zones of Shallow Foundation.

each side of this zone there are two plastic zones, i. e. a zone of radial shear ACD, and a zone of plane shear ADE. In the case of Terzaghi's solution, the shearing strength of the overburden is ignored and only its weight is taken into account as an equivalent surcharge p_o equal to γDf. This method has been found to be conservative, and "The assumed mechanism of failure usually not in accordance with the observed ground movement."

In an attempt to overcome these limitations, Meyerhof (5), in 1951 has extended the previous analysis of the plastic equilibrium to shallow and deep foundations. According to this theory, it is assumed to be divided into two main zones on each side of the central zone ABC as shown in Figure (4b), namely a radial shear zone BCD and a mixed shear zone BDEF in which the shear varies between the limits of radial and plane shear, depending largely on the depth and roughness of the foundation.

For the convenience of illustration, Meyerhof made the zones of plastic equilibrium corresponding to the general case as shown in Figure (5a), where the surface AE is inclined at B and subjected to the stresses p_o and s_o, normally and tangentially, respectively.

In the plane shear zone ADE, with angle η at A, the plastic equilibrium requires that along AD and DE the shearing strength s_1 under the normal pressure p_1 is fully mobilized and is equal to $c + p_1 \tan\phi$.

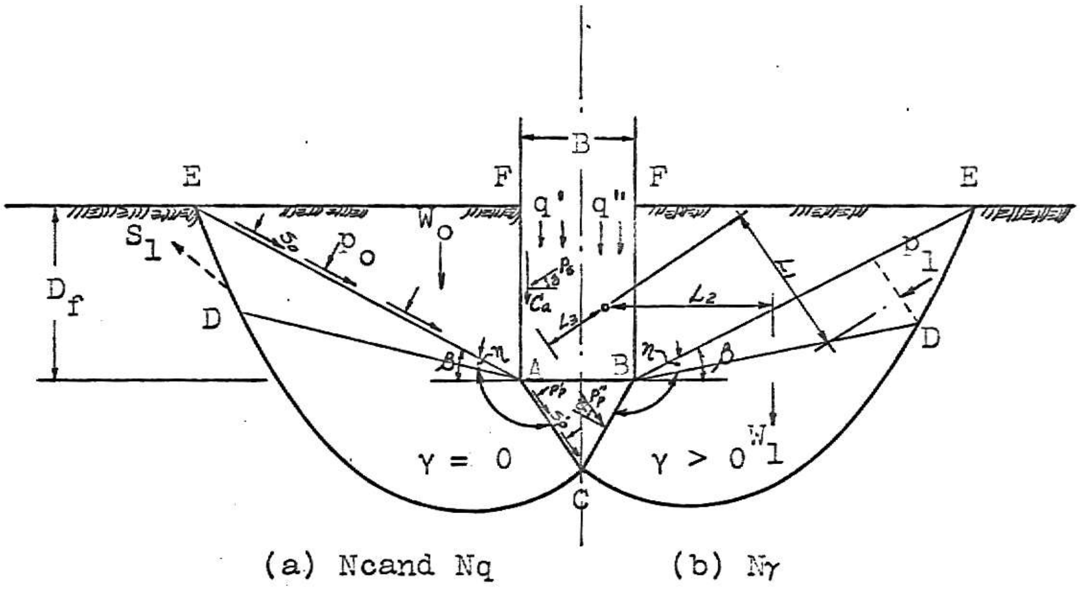


Fig. 5. Determination of general bearing capacity factors for strip foundation with rough base.

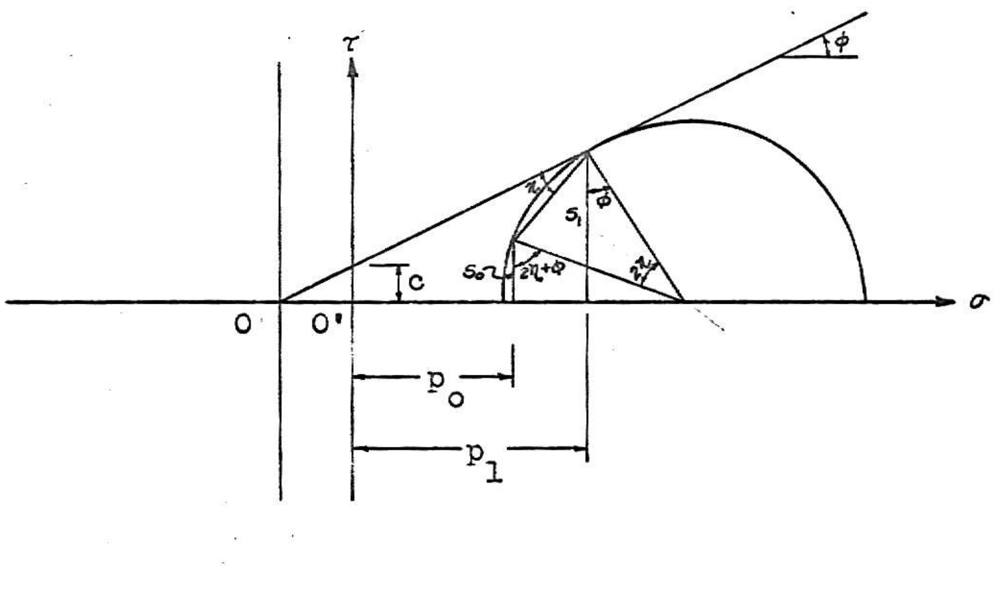


Fig. 6. Mohr's Diagram

Hence from Mohr's diagram (Fig. 6),

$$\cos(2\eta + \phi) = \frac{s_o \cos\phi}{c + p_1 \tan\phi}$$

$$= \frac{(c + p_1 \tan\phi)\cos\phi}{c + p_1 \tan\phi} \qquad (7)$$

and

$$p_1 = \frac{c + p_1 \tan\phi}{\cos\phi}\left[\sin(2\eta + \phi) - \sin\phi\right] + p_o \qquad (8)$$

In the radial shear zone ACD with angle θ, CD is a logarithmic spiral surface, and along this surface and along radial sections the shearing strength is fully mobilized. Along AC the normal and tangential components of the passive earth pressure are, respectively,

$$p'_p = (s'_p - c)\cot\phi \qquad (9)$$

$$s'_p = (c + p_1 \tan\phi)\, e^{2\theta\tan\phi} \qquad (10)$$

from which the bearing capacity is

$$q' = p'_p + s'_p \cot(45^o - \phi/2) \qquad (11)$$

Substituting equations (8) to (10) into (11), we get

$$* \; q' = c\left[\cot\phi\left\{\frac{(1 + \sin\phi)\, e^{2\theta\tan\phi}}{1 - \sin\phi\sin(2\eta + \phi)}\right\} - 1\right]$$

$$+ p_o\left[\frac{(1 + \sin\phi)\, e^{2\theta\tan\phi}}{1 - \sin\phi\sin(2\eta + \phi)}\right] \qquad (12)$$

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*; The computation see Appendix A2.

or $q' = cNc + p\ Nq$

Where Nc and Nq have the values given in the square brackets above.

Considering forces to the right of the foundation centre line of Figure (5b), in order to obtain the minimum factor N_γ, Meyerhof used the logarithmic spiral method (Terzaghi 1943) for locating the worst centre of the spiral. The plastic equilibrium is found by balancing the moments about any point 0. Thus,

$$L_3\ Pp'' = P_1\ L_1 + W_1\ L_2$$

$$Pp'' = \frac{P_1\ L_1 + W_1\ L_2}{L_3}$$

This analysis is repeated for different centres 0 until the minimum value of Pp" is found which represents the total passive earth pressure. This procedure is rather laborious in practice since at least a dozen trials have to be made in any given case to determine the minimum resistence from which Meyerhof provided

$$q'' = \frac{\gamma B}{2}\left[\frac{4\ Pp''\ \sin(45^o + \phi/2)}{\gamma B^2} - 1/2\ \tan(45^o + \phi/2)\right] \quad (13)$$

or $q'' = \gamma B/2\ N_\gamma$.

Where N_γ has the value given in the square brackets above.

The total bearing capacity is

$$q = q' + q''$$

$$q = cNc + p_oNq + \gamma B/2 \; N_\gamma \qquad (14)$$

Where Nc, Nq and N_γ are coefficients, their values also can be given in the curves of Figure (7).

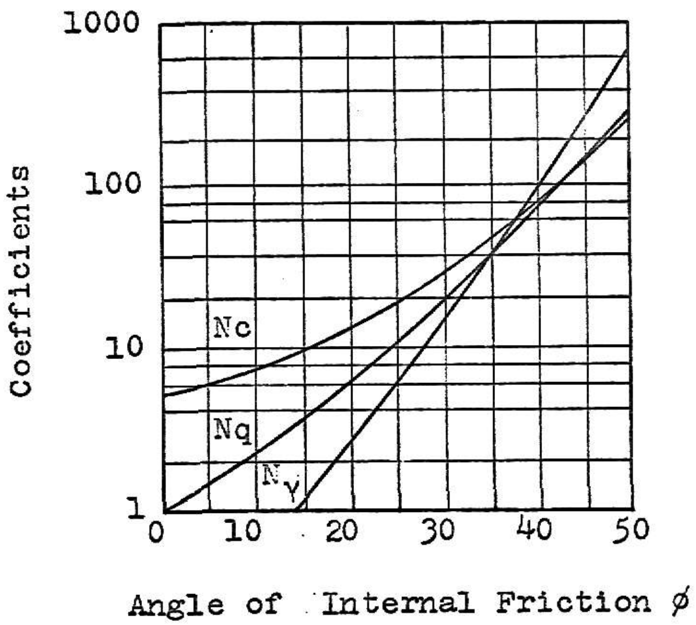

Fig. 7. Meyerhof's Bearing Capacity Factors for Shallow Strip Foundation with Rough Base.

(c). General Bearing Capacity Equation--Terzaghi-Meyerhof;

A simple and conservative analysis was extended by Terzaghi and further modified by Meyerhof, is called Terzaghi-Meyerhof method. The equation for bearing capacity can be rewritten as:

$$q_{ult} = \frac{\gamma B}{2} N_\gamma + cNc + qNq \qquad (15)$$

Where

q_{ult} = ultimate bearing capacity

γ = the effective soil unit weight

B = the foundation width

c = cohesion

q = surcharge = γDf

Nc, Nq, and N_γ are bearing capacity factors that are functions of the angle of internal friction. The values of these factors for different values of ϕ are given in Figure (8).

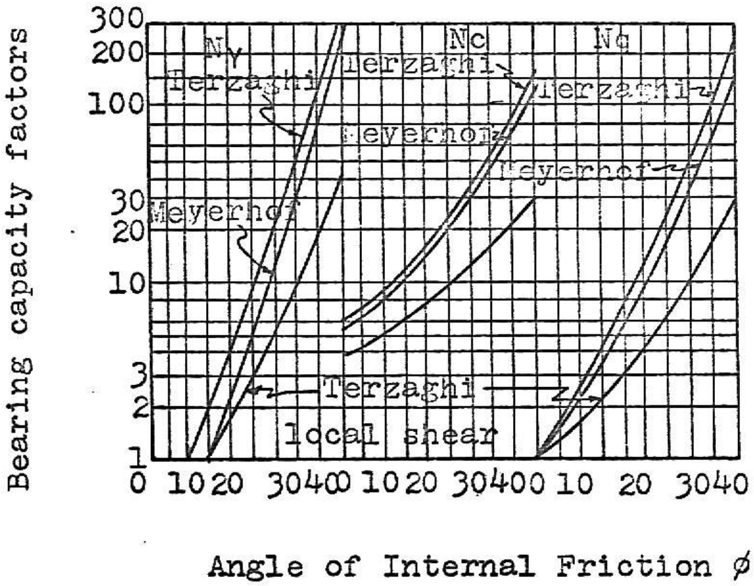

Fig. 8. Bearing Capacity Factors for the General Equation.

As can be seen by an examination of the general bearing capacity equation (Equation 15) and the curve of Figure(8), the soil bearing capacity for both Terzaghi and Meyerhof analyses depends on the angle of internal friction ϕ, the soil unit weight γ, the foundation width B, the cohesion c, and the surcharge q. The angle of internal friction has by far the greatest influence on all three bearing capacity terms. All increase at a rapidly increasing rate when the angle of internal friction becomes larger.

(a). Cohesionless Soils:

$c = 0$, then equation (15) becomes

$$q_{ult} = \gamma B/2 N_\gamma + q Nq$$

The first term of equation varies in direct proportion to the foundation width, in cohesionless soils such as sands, the above equation shows that the bearing capacity is greatly influenced by the footing size and the surcharge. This means that the bearing capacity of small foundations is low and that of large foundation is high. This relationship may be expressed by the statement that for a footing on the surface of sand, the bearing capacity is directly proportional to the size of the footing. Further, the bearing capacity goes up significantly as the depth of the footing below the surface increases. The importance of these two variables(footing size and depth below the surface) is illustrated in the numerical example (1) which will be shown later.

(b). Clay Soils:

$\phi=0$, then the values of the N_γ and Nq are very small, the first term and third term of the equation (15) approaches zero, and the soil cohesion becomes the major point of the bearing capacity, then, the equation (15) becomes,

$$q_{ult} = cNc$$

This means that the contribution of the foundation and surcharge to bearing capacity is small and that may be negligible for soils with a small angle of internal friction such as saturated clays.

For Plate-Load Test:

According to Sowers's test (6) results;

(a). Cohesionless Soils

Sowers stated that "In sands and gravels the bearing capacity increases in direct proportion to the width of the loaded area," such that

$$q_{ult}\text{ (foundation)} = q_{ult}\text{(load test)}\cdot\left[\frac{\text{width foundation}}{\text{width test plate}}\right]$$

(b). Clay Soils

Sowers stated that "The bearing capacity of footings on a clay soil is independent to the width of the loaded area, the critical pressure determined by the load test is the same for all footing size," such that

$$q_{ult}\text{ (foundation)} = q_{ult}\text{ (load test)}$$

B. Settlement:

Settlement of the soil producted by loading comes from two soures; (1). The Immediate Settlement (ρ_i) [Skempton,(7)] or The Contact Settlement [Sowers,(6)] takes place during application of the loading as a result of elastic deformation of the soil without change in water content. (2). The Consolidation Settlement (ρc) [Skempton,(7)] or The Compression Settlement [Sowers,(6)] takes place as a result reduction of the soil caused by extrusion of some of porewater from the soil. The Final Settlement (ρf) is the sum of ρi and ρc.

(1). The Immediate Settlement or Contact Settlement:

This method assumes that the soil is perfectly elastic and isotropic, and is in current use for the prediction of immediate settlement in clays.

(a). Cohesive Soils:

According to theory of elasticity, Terzaghi(4) and Skempton(7) provided the expression;

$$\rho_i = qB \cdot \frac{1 - \mu^2}{E} \cdot Ip \qquad (16)$$

Where

q = net foundation pressure

B = width of the foundation or diameter of the loaded area

μ = Poisson's ratio

E = modulus of elasticity

I_p= influence value, depending on the shape of the loaded area and the depth of the clay bed

The typical range of values for Poisson's ratio μ is given in Table (1). And the influence values for some conditions of leading are given in Table (2).

Type of soil	μ
Clay, saturated	0.4-0.5
Clay, unsaturated	0.1-0.3
Sandy clay	0.2-0.3
Silt	0.3-0.35
Sand (dense)	0.2-0.4
Coarse (void ratio=0.4-0.7)	0.15
Fine-grained(void ratio=0.4-0.7)	0.25
Rock	0.1-0.4

Table (1). Typical range of values for Poisson's ratio μ

Shape	Flexible			Rigid
	Center	Corner	Average	
Circle	1.00	0.64	0.85	0.88
Square	1.12	0.56	0.95	0.82
Rectangle				
L/B=1.5	1.36	0.68	1.20	1.06
2	1.53	0.77	1.31	1.20
5	2.10	1.05	1.83	1.70
10	2.52	1.26	2.25	2.10
100	3.38	1.69	2.96	3.40

Table (2). Influence values for various-shaped members (I_p).

For saturated clays, there is no volume change so long as there is no dissipation of pore pressure. Consequently in calculation of immediate settlement $\mu = 0.5$, thus

$$\rho_i = qB \frac{0.75}{E} I_p \tag{17}$$

Sowers (6) stated that "The distortion settlement (or immediate settlement) occurs because of a change in shape of the soil mass rather than because of a change in void ratio." He divided this into two parts to express the immediate settlement;

1. Flexible Foundation: For a loaded square area of width B, the settlement ρ_i of a corner and of the center are given by the formulas;

$$(\rho_i)_{cor} = \frac{0.42qB}{E} \tag{18a}$$

$$(\rho_i)_{cen} = \frac{0.84qB}{E} \tag{18b}$$

2. Rigid Foundation:

$$(\rho_i) = \frac{0.6qB}{E} \tag{18c}$$

Sowers also according to the load test to determine the immediate settlement (or distortion settlement) for saturated clays as following;

$$\rho_i(\text{foundation}) = \rho(\text{test plate})\cdot\left[\frac{B(\text{foundation})}{B(\text{plate})}\right] \tag{19}$$

Looking at the formulas above which given by Skempton and Sowers. Although there exist some differences, in general, they all indicate that the immediate settlement (or distortion settlement) of the saturated clays are directly proportional to the width of the foundation.

Skempton and Sowers are assuming that the clay is perfectly elastic, it is only applicable for low loads. For this reason, a semi-empirical relationship was provided by

Housel (8) based on simplifying assumptions that "If foundations having different sizes are supported by the same purely cohesive soil, the foundation pressure P can within limits be compared with foundation size for the same magnitude of settlements in each case by the equation" ;

$$P = P_c + s \cdot \frac{P_r}{A} \qquad (20)$$

Where

P = the foundation contact pressure within limits

A = area of the foundation

P_r = the perimeter of the loaded area A

P_c and s are empirical constants which vary with the type of soil and with the magnitude of the settlement. Their values can be found from loading tests using plates having different sizes.

This formula is valid within limits for circular, square, and rectangular foundations on clays. This method has been used successfully on cohesive subgrades for roads and aerodrome runways. An example is shown in Figure (9).

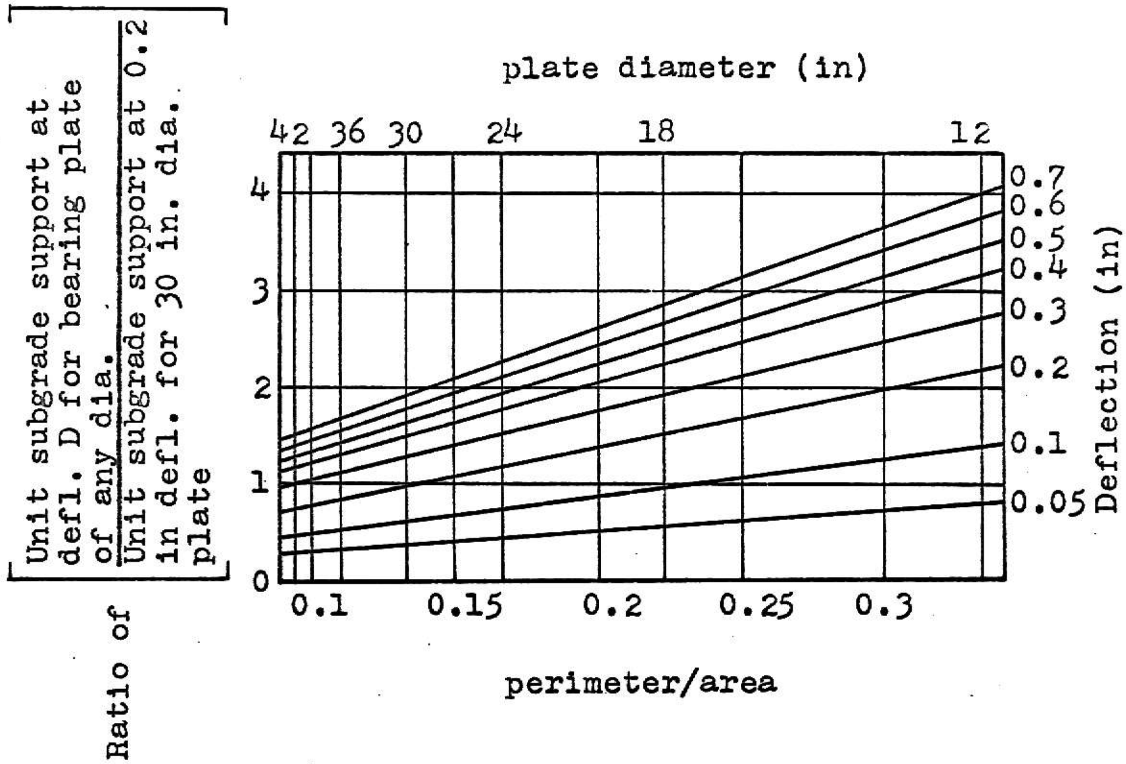

Fig. 9. Unit subgrade support for any diameter of plate compared with that of a 30-in.-dia. plate for 0.2-in. deflection.

From Figure (9) we can see that within the same contact pressure, the wider the foundation is, the greater settlement will be.

(b). Cohesionless Soils:

The settlements of foundations in granular soils such as sands. Bond (9) introduced a statistical correlation of limited data provided the relationship of Figure (10) to separate that portion of settlement due to compressibility from that due to lateral deformation. Although this was not of significant practical use, it did serve to show that "For a given shape of foundation on cohesionless soil and for any particular value of load per unit area, there is one size of foundation for which the value of settlement is a minimum."

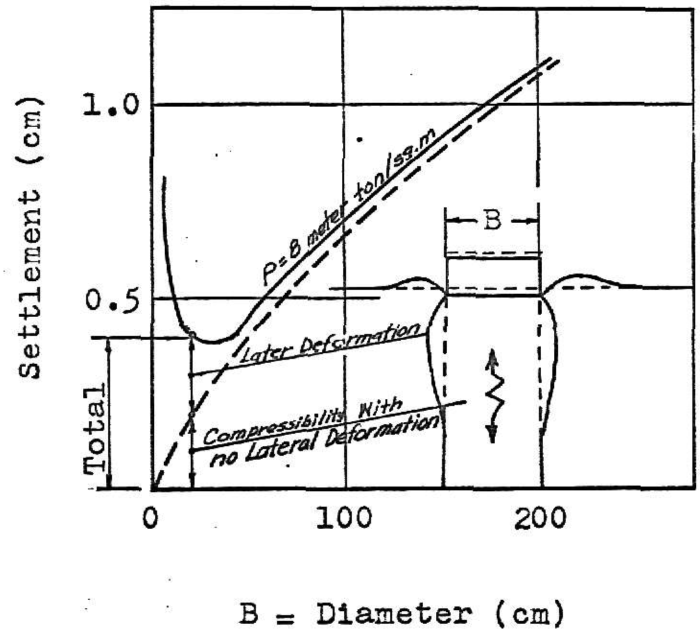

Fig. 10. The Influence of Lateral Deformation on Settlement in Cohesionless Soils.

Using the settlement observations of the above expression, Terzaghi and Peck (10) recommended an empirical formula:

$$\frac{\rho}{\rho_1} = \left[\frac{2B}{B+1}\right]^2 \qquad (21)$$

Where

ρ = the settlement of a 1 ft square test plate under a given load q per unit of area (in)

ρ_1 = the settlement at the same load per unit of area of a footing with a width B (in)

B = the width of the foundation (ft)

Bond (9) indicated that "This method is the only practical method which exists at present for computing settlements in sand with foundation size, its application is restricted." It is based on a statistical correlation of limited data. It is not applicable to plate sizes of less than 12 in. and for sizes above 12 in. it possibly gives the upper limits of values of settlement in sand, as shown in Figure (11).

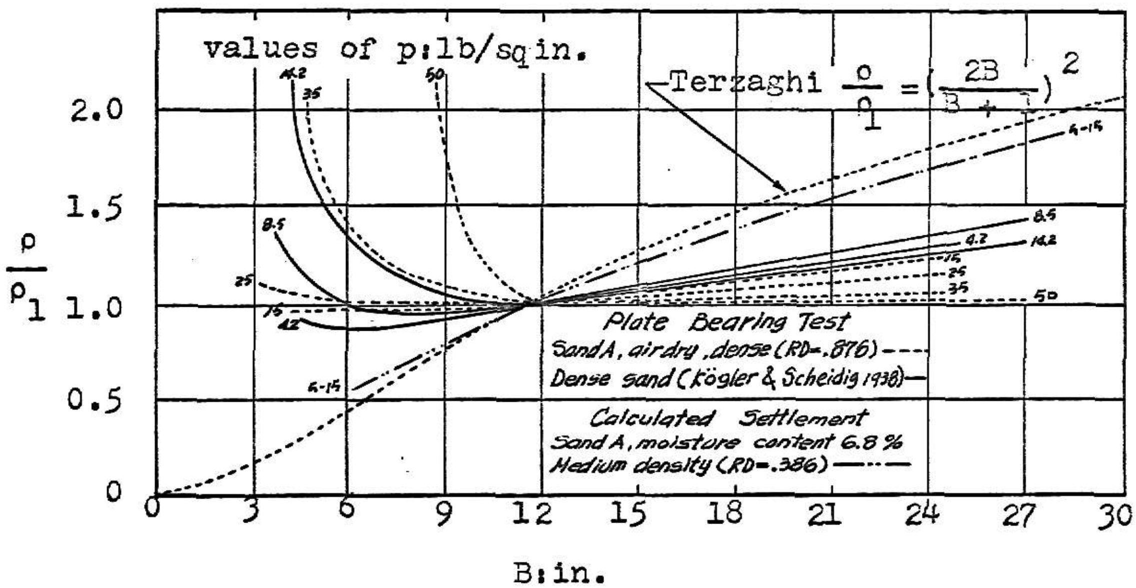

Fig. 11. Values of ρ/ρ_1 compared with B for sand for particular values of p.

Sowers extended Terzaghi and Peck's work and developed the following expression;

$$\rho_i \text{ (foundation)} = \rho \text{ (test plate)} \cdot \left[\frac{B_f(B_p+1)}{B_p(B_f+1)} \right]^2 \qquad (22)$$

Where

B_f = the width of the foundation (ft)

B_p = the width of the plate (ft)

If we let B_f = 1 ft, the result is same as Terzaghi and Peck's formula.

Terzaghi & Peck and Sowers' equations (equ. 21 and 22) indicate that if the foundation is not too small, the immediate settlement is approximately directly proportional to the width of the foundation; if the foundation is too small, it may occur great amount of lateral deformation cause for settlement.

(2). The Consolidation Settlement or The Compression Settlement:

To compute the compression settlement of the clay layer below each of the selected points. According to Terzaghi and Peck's one-dimensional consolidation theory (Ref. 10, pp 209),

$$d\rho_{oed} = m_v \Delta\sigma_1 \, dz$$

Then

$$\rho_{oed} = \int_0^z m_v \Delta\sigma_1 \, dz \qquad (23)$$

where

m_v = the coefficient of volume compressibility

$\Delta\sigma_1$ = vertical pressure at any depth Z below the point at which the settlement is to be computed

dz = the thickness of the element

Skempton (7) expressed by an analogous equation

$$d\rho_c = m_v \cdot u \cdot dz$$

$$\rho_c = \int_o^Z m_v \cdot u \cdot dz \qquad (24)$$

where

ρ_c = the consolidation settlement

*m_v = the coefficient of volume compressibility

dz = the thickness of the element

$u = B\left[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right]$ = pore pressure

Skempton, A. W. (11) provided that B and A are the pore pressure coefficients. And according to his test results in the saturated soils where B=1;

$$u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$$

$$= \Delta\sigma_1 \cdot \left[A + \frac{\Delta\sigma_3}{\Delta\sigma_1}(1 - A)\right]$$

where

$\Delta\sigma_1$ = vertical stress at any depth below the point at which the settlement is to be computed

$\Delta\sigma_3$ = the confining pressure at that point

*; Typical values of the coefficient m_v see Appendix B.

Hence, equation (24) becomes

$$\rho_c = \int_0^z m_v \cdot \Delta\sigma_1 \left[A + \frac{\Delta\sigma_3}{\Delta\sigma_1}(1 - A)\right]dz \qquad (25)$$

Upon comparing equations (23) and (25), Skempton postulated that the two can be related by a factor U, such that

$$\rho_c = U \cdot \rho_{oed}$$

Where

$$U = \frac{\int_0^z m_v \cdot \Delta\sigma_1 \cdot \left[A + \frac{\Delta\sigma_3}{\Delta\sigma_1}(1-A)\right] dz}{\int_0^z m_v \cdot \Delta\sigma_1 \, dz}$$

$$= A + \alpha(1 - A)$$

$$\alpha = \frac{\int_0^z \Delta\sigma_3 \, dz}{\int_0^z \Delta\sigma_1 \, dz}$$

The values of α has been provided by Skempton (7) as shown in Table (3).

Z/B	Circular footing	Strip footing
0	1.00	1.00
0.25	0.67	0.74
0.5	0.50	0.53
1	0.38	0.37
2	0.30	0.26
4	0.28	0.20
10	0.26	0.14
∞	0.25	0.00

Table (3). Values of α in the equation $U = A + \alpha(1-A)$

The values of U can be determined by the values of A and α, Figure (12) is also sufficient to enable an approximate values of U to be chosen.

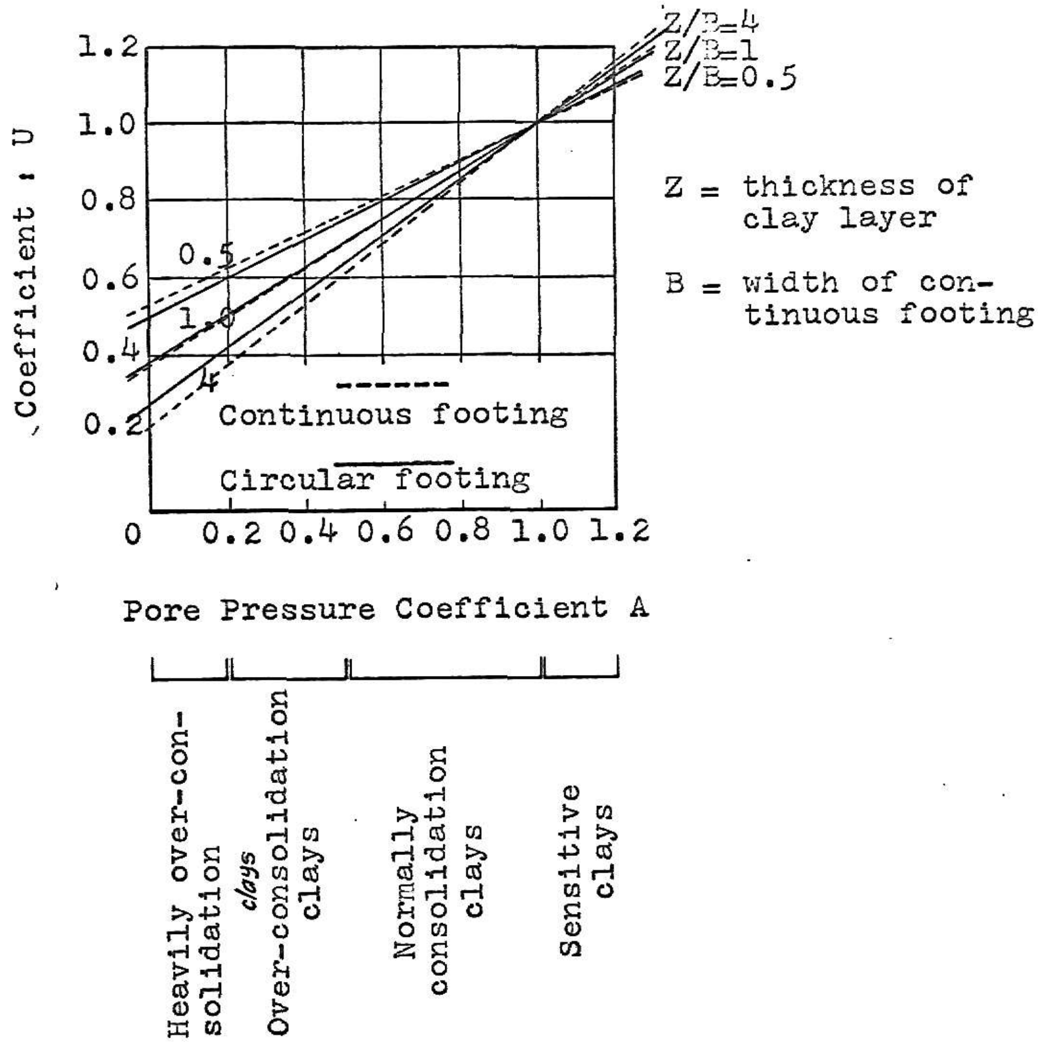

Fig. 12. Values of the Factor U.

Therefore, the consolidation settlement is

$$\rho_c = U \int_o^z m_v \cdot \Delta \sigma_1 \cdot dz \qquad (26)$$

Values of $\Delta\sigma_1$ have been tabulated by many different formulas. The followings are recommended:

(a). The Boussinesq Equation; (Ref.1, pp. 252)

$$\sigma_1 = \frac{Q}{Z^2} \cdot \frac{3/2\pi}{[1 + (r/Z)^2]^{5/2}} = \frac{Q}{Z^2} N_B \qquad (27)$$

where N_B = Boussinesq index

(b). The Westergaard Equation; (Ref.1, pp. 258)

$$\sigma_1 = \frac{Q}{Z^2} \cdot \frac{\frac{1}{2\pi}\sqrt{\frac{1 - 2u}{2 - 2u}}}{\left[\frac{1 - 2u}{2 - 2u} + (\frac{r}{Z})^2\right]^{3/2}} \qquad (28)$$

When $\mu = 0$ then

$$\sigma_1 = \frac{Q}{Z^2} \cdot \frac{1/\pi}{[1 + 2(r/Z)^2]^{3/2}} = \frac{Q}{Z^2} N_W \qquad (29)$$

In Figure (13). a plot of N_B and N_W as a function of r/Z is given.

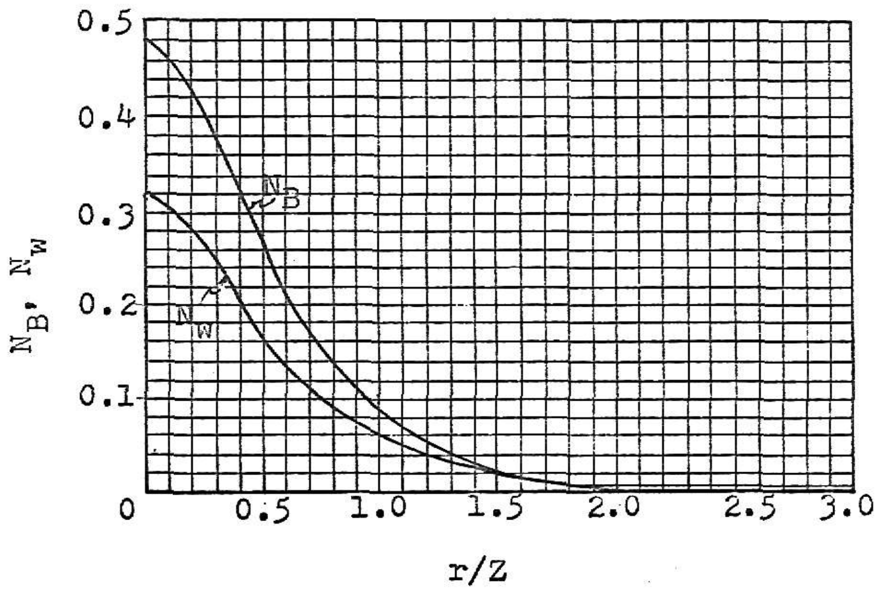

Fig. 13. Chart for determing vertical stresses caused by surface point loads.

(c). The Love Equation; (12)

For stresses below a uniformly loaded area, Love based on Boussinesq's equation, dividing the loaded area into small parts, and then equation (27) becomes

$$d\Delta\sigma_1 = \frac{3qdA}{2\pi Z^2} \cdot \left[\frac{1}{1 + (r/Z)^2}\right]^{5/2} \qquad (30)$$

The vertical pressure $\Delta\sigma_1$ due to the distributed load on the entire area of radius R is obtained by integrating equation (30), replacing dA with $2\pi rdr$, then

$$* \quad \Delta\sigma_1 = \int_0^R \frac{3q(2\pi r dr)}{2\pi Z^2} \left[\frac{1}{(1 + (r/Z)^2}\right]^{5/2}$$

$$= q\left\{1 - \left[\frac{1}{1 + (R/Z)^2}\right]^{3/2}\right\} \qquad (31)$$

For square foundation, $R= \sqrt{\frac{A}{\pi}}$

(d). Hough's Approximate Equation;

The analysis of stress beneath a footing can not always be made by direct application of the Boussinesq equation. This is particularly true for conditions when Z is very small, since as Z approaches zero, the stress increment approaches an infinite value. Because of this and other difficulties encountered in practice, Hough's (13) approximate method of analyzing the stresses was developed.

It is assumed that the stress increment at successive depths beneath the footing is also distributed uniformly over a finite area, as shown in Figure (14a). The intensity of the stress increment at any depth is assumed to be equal to the total load P, on the footing divided by the area A_2. In applying this approximation, Hough assumed the inclination of the bounding area is $63\frac{1}{2}^{\circ}$ rather than 60° as shown in Figure (14b). On this basis, if one dimension of the footing

---

*; The computation see Appendix A3 (a).

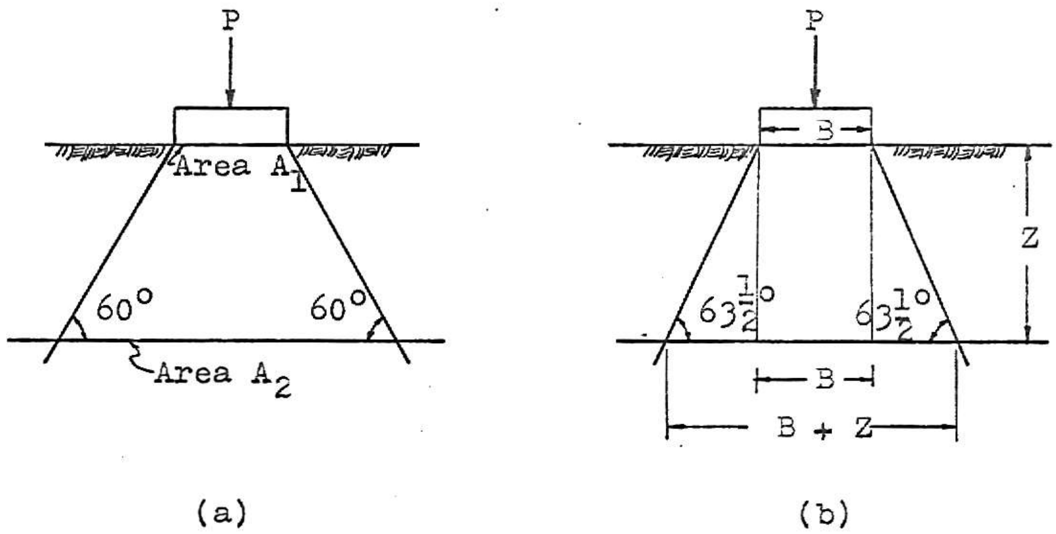

Fig. (14). Hough's Approximation for Stress due to Footing.

is B, the corresponding dimension of the area A_2 becomes $(B + Z)$. For a square footing the area A_2 is $(B + Z)^2$, and the stress intensity at depth Z is expressed as

$$\Delta\sigma_1 = \frac{B^2}{(B + Z)^2} \cdot q \tag{32}$$

If substitute equations (31) and (32) into (26), we get;

Skempton-Love Equation

$$\rho_c = m_v \ U \ q \int_0^Z \left\{ 1 - \left[\frac{1}{1 + (R/Z)^2} \right]^{3/2} \right\} dz \tag{33}$$

and

Skempton-Hough Equation

$$\rho_c = m_v \ U \ q \int_0^Z \frac{B^2}{(B + Z)^2} \, dz \tag{34}$$

By integrating equations (33) and (34), then Skempton-Love Equation

$$* \quad \rho_c = m_v \, U \, q \left\{ \cdot Z - \left[\frac{Z^2 + 2R^2}{(Z^2 + R^2)^{\frac{1}{2}}} - 2R \right] \right\} \qquad (35)$$

Skempton-Hough Equation

$$* \quad \rho_c = m_v \, U \, q \, \frac{Z}{1 + Z/B} \qquad (36)$$

From examination of equation (35) and (36), it is seen that settlement is directly proportional to foundation size.

(e). The Newmark Influence Chart:

Using the influence chart for computing vertical stress is a simple, rapid, and accurate method for calculation, and is sufficiently accurate for all practical purposes. This concept and the influence charts were presented by Newmark (14). The use of the chart is based on a factor termed the "Influence value," determined from the number of units into which the chart is subdivided. (For example, if the series of rings are subdivided so that there are 400 units, often made approximate squares, the influence value is 1/400 = 0.0025).

The following equation is provided for computing the stress at the depth Z

$$\Delta\sigma_1 = q_o \cdot M \cdot I \qquad (37)$$

---

*; The computation see Appendix A3 (b), (c).

Where

q_o = foundation contact pressure

M = number of units counted (partial units are estimated)

I = influence factor of a particular chart used

It is also indicated that the vertical stresses $\Delta\sigma_1$ and the settlements under the footing at the same depth are directly proportional to the foundation size.

(3) The Final Settlement:

The final settlement is the sum of the corrected values of the immediate settlement and the consolidation settlement, i.e.

$$\rho_f = \rho_i + \rho_c \tag{38}$$

C . Numerical Examples:

For the convenience of the illustration and a clearer understanding, the following examples are taking from Lambe (15) in an attempt to illustrate the influence of footing size on bearing capacity and settlement.

Example 1.

Given: A round footing resting on sand with $\phi = 34\frac{1}{2}^{o}$ and $\gamma = 100$ lb/ft

Find: The bearing capacity for

(a). B = 3 ft, Df = 0

(b). B = 3 ft, Df = 2 ft

(c). B = 6 ft, Df = 0

(d). B = 6 ft, Df = 2 ft

Where B = diameter of the round footing

Df = depth of the footing below surface

Solution:

For round footing on sand, c = 0 and equation (15) becomes

$$q_{ult} = 0.3\,\gamma B N_\gamma + qN_q$$

From Figure (3a) get $N_\gamma = N_q = 35$, then

(a). Df = 0, q = 0, B = 3 ft

$$q_{ult} = 0.3(100)(3)(35) = 3.15 \text{ kips/ft}^2$$

(b). Df = 2 ft, q = (100)(2), B = 3 ft

$$q_{ult} = 0.3(100)(3)(35) + (100)(2)(35) = 10.15 \text{ kips/ft}^2$$

(c). $Df = o$, $q = 0$, $B = 6ft$

$$q_{ult} = 0.3(100)(6)(35) = 6.3 \text{ kips/ft}^2$$

(d). $Df = 2$ ft, $q = (100)(2)$, $B = 6$ ft

$$q_{ult} = 0.3(100)(6)(35) = (100)(2)(35)$$

$$= 13.3 \text{ kips/ft}$$

The illustration of example 1 may be presented by the statement that "For footings on the surface of sand, the bearing capacity is directly proportional to the size of the footing. Further, the bearing capacity goes up significantly as the depth of the footing below the surface increses."

Example 2.

Given: A 48-ft-high tank is built on sand with $\gamma = 129$pcf and $\mu = 0.45$.

Find: The settlement of the center of the tank when filled with water for the following conditions:

(a). $B = 100$ ft, E constant and equal to 4000 kips/ft^2

(b). $B = 200$ ft, E constant and equal to 4000 kips/ft^2

(c). $B = 100$ ft, E varies as σ_1 and equal to 4000 kips/ft^2 at $Df = 75$ ft.

(d). $B = 200$ ft, E varies as σ_1 and equal to 4000 kips/ft^2 at $Df = 75$ ft.

Solution:

For settlement at center of the tank, the equation (16) becomes

$$\rho = q\frac{R}{E}\, 2\ (1 - u^2)$$

q = average stress over the loaded area

R = radius of the loaded area

$q = 48 \text{ ft} \times 62.4 \text{ lb/ft}^3 = 3.0 \text{ kips/ft}^2$

$2(1 - \mu^2) = 2(1 - 0.45^2) = 1.6$

(a). $\rho = 3 \times \dfrac{(50)(1.6)}{4000} = 0.06 \text{ ft}$

(b). $\rho = \dfrac{(3)(100)(1.6)}{4000} = 0.12 \text{ ft}$

(c). Since E varies as σ_1 and σ_1 varies as depth, so E varies as depth, i.e. $E_{\frac{3}{4}Df} = E_{75ft} = 4000 \text{ kips/ft}^2$

$\rho = \dfrac{(3)(50)(1.6)}{4000} = 0.06 \text{ ft}$

(d). Now $E_{\frac{3}{4}Df} = E_{150ft} = 2 \times 4000 \text{ kips/ft}^2$

$\rho = \dfrac{(3)(100)(1.6)}{2 \times 4000} = 0.06 \text{ ft}$

The illustration of example 2 may be presented by the statement that "For footings on the surface of sand, if E is constant with depth, settlement is directly proportional to foundation size. If the modulus E varies directly with the vertical confining stress, the settlement is independent of foundation size."

In view of the above expressions, it is showing that reasonably good agreement is seen to exist between these expressions and those of which presented in the preceding sections.

III. CONCLUSIONS:

Previous theories of the ultimate bearing capacity and settlement analysis have been reviewed. The analysis indicates that in general the bearing capacity increases with size, depth and roughness of the base, and depends on the shape of the foundation. Settlement increases somewhat with increasing footing size.

The above considerations lead to the conclusion that

For Cohesionless Soils:

In a cohesionless soil, the ultimate bearing capacity is proportional to the width of the footing. That is, the ultimate bearing capacity increases as the footing size increase. Under the same contact pressure(E is constant), settlement is directly proportional to foundation size, as shown in Figure (15b).

For Cohesive Soils:

In a cohesive soil, the ultimate bearing capacity is a constant and is independent of the width of the footing. That is, foundation size has no effect on the ultimate bearing capacity. If the contact pressure is constant(E is constant), settlement is directly proportional to foundation size, as shown in Figure (15a).

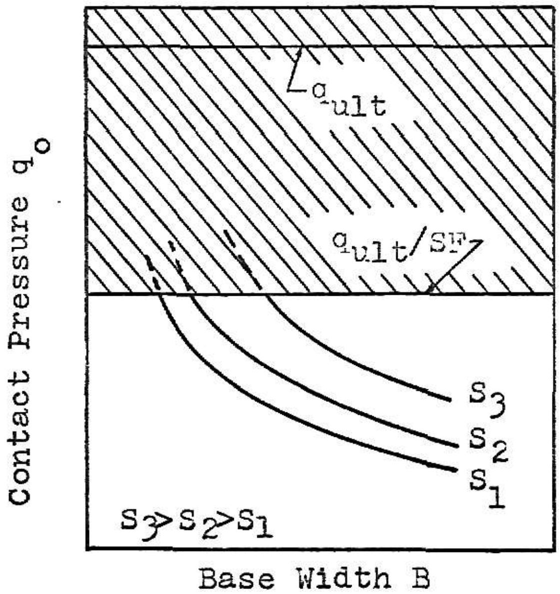

Fig. 15a. Cohesive Soil

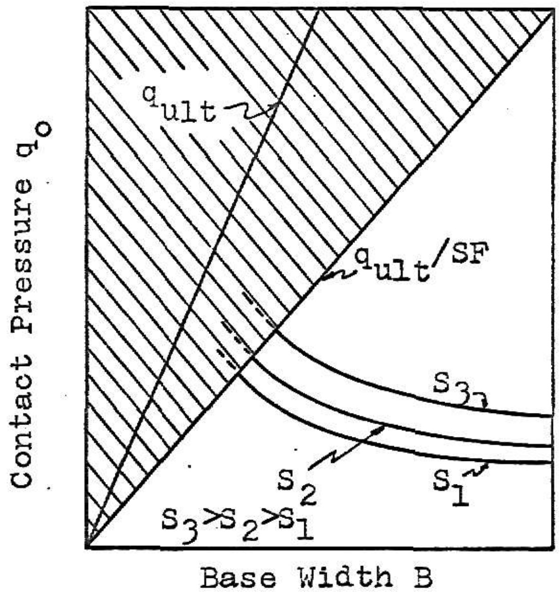

Fig. 15b. Cohesionless Soil

APPENDIX A1

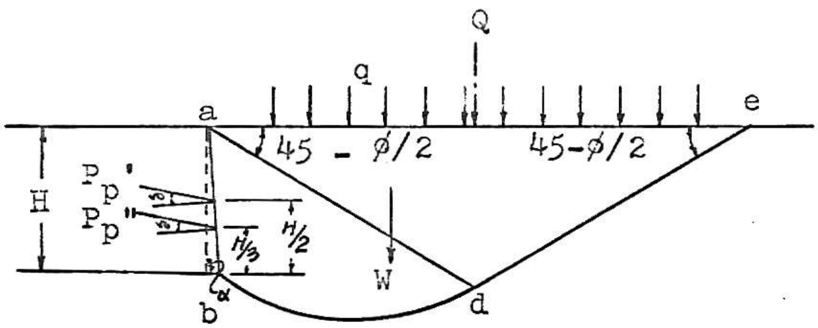

Fig. A.

The normal component p_n of the passive earth pressure per unit of area of a plane contact face (ab, Fig. A) at depth z below a can be expressed approximately by the linear equation

$$
\begin{aligned}
p_n &= cK_{pc} + qK_{pq} + \gamma z K_{p\gamma} \\
&= p_n' + p_n''
\end{aligned}
$$

$$p_n' = cK_{pc} + qK_{pq}$$

$$P_n' = \frac{1}{\sin\alpha}\int_o^H p_n' \, dz = \frac{H}{\sin\alpha}\,(cK_{pc} + qK_{pq})$$

$$P_n'' = \gamma z K_p$$

$$P_n'' = \frac{1}{\sin\alpha}\int_o^H p_n'' \, dz = 1/2\,\gamma H^2 \frac{K_{p\gamma}}{\sin\alpha}$$

$$
\begin{aligned}
P_n = P_n' + P_n'' &= \frac{H}{\sin\alpha}(cK_{pc} + qK_{pq}) \\
&\quad + 1/2\,\gamma H^2 \frac{K_{p\gamma}}{\sin\alpha} \qquad (3)
\end{aligned}
$$

APPENDIX A2

$$p_1 = \frac{c + p_1\tan\phi}{\cos\phi}\left[\sin(2\eta+\phi)\right] - \sin\phi + p_o \qquad (8)$$

$$p_p' = (s_p' - c).\ \cot\phi \qquad (9)$$

$$s_p' = (c + p_1\tan\phi)\ e^{2\theta\tan\phi} \qquad (10)$$

$$q' = p_p' + s_p'\ \cot(45^o - \phi/2) \qquad (11)$$

Substituting equations (8) to (10) into (11), then

From (8),

$$p_1\cos\phi = (c + p_1\tan\phi)\ [\sin(2\eta+\phi) - \sin\phi] + p_o\cos\phi$$

$$p_1\cos\phi - p_1\tan\phi\ [\sin(2\eta+\phi)-\sin\phi] = c[\sin(2\eta+\phi)-\sin\phi] + p_o\cos\phi$$

$$p_1 = \frac{c\ [\sin(2\eta+\phi) - \sin\phi] + p_o\cos\phi}{\cos\phi - \tan\phi\ [\sin(2\eta+\phi) - \sin\phi]}$$

$$p_1\tan\phi = \frac{\tan\phi\ \{c\ [\sin(2\eta+\phi) - \sin\phi] + p_o\cos\phi\}}{\cos\phi - \tan\phi\ [\sin(2\eta+\phi) - \sin\phi]}$$

$$p_1\tan\phi = \frac{\sin\phi\ \{c\ [\sin(2\eta+\phi) - \sin\phi]\ p_o\cos\phi\}}{[1 - \sin\phi\ \sin(2\eta+\phi)]}$$

From (10),

$$s_p' = (c + p_1\tan\phi)\ e^{2\theta\tan\phi}$$

$$= ce^{2\theta\tan\phi} + p_1\tan\phi\ e^{2\theta\tan\phi}$$

$$= ce^{2\theta\tan\phi} + \frac{\sin\phi\{c\,[\sin(2\eta+\phi) - \sin\phi] + p_o\cos\phi\}e^{2\theta\tan\phi}}{[1 - \sin\phi\ \sin(2\eta+\phi)]}$$

$$s_p' = \frac{ce^{2\theta\tan\phi} - ce^{2\theta\tan\phi}\sin\phi\sin(2\eta+\phi) + ce^{2\theta\tan\phi}\sin\phi\sin(2\eta+\phi)}{[1 - \sin\phi\ \sin(2\eta+\phi)]}$$

$$\underline{-\ e^{2\theta\tan\phi}c\ \sin^2\phi + e^{2\theta\tan\phi}p_o\sin\phi\ \cos\phi}$$

$$s_p' = \frac{ce^{2\theta\tan\phi} - ce^{2\theta\tan\phi}\sin^2\phi + e^{2\theta\tan\phi}\; p_o\sin\phi\,\cos\phi}{1 - \sin\phi\,\sin(2\eta+\phi)}$$

$$s_p' = \frac{ce^{2\theta\tan\phi}(1 - \sin^2\phi)}{[1 - \sin\phi\,\sin(2\eta+\phi)]} + \frac{p_o\sin\phi\,\cos\phi\; e^{2\theta\tan\phi}}{[1 - \sin\phi\,\sin(2\;+\phi)]}$$

$$s_p' = \frac{ce^{2\theta\tan\phi}\;\cos^2\phi}{[1 - \sin\phi\,\sin(2\eta+\phi)]} + \frac{p_o\sin\phi\,\cos\phi\; e^{2\theta\tan\phi}}{[1 - \sin\phi\,\sin(2\eta+\phi)]}$$

Then equation (11) becomes,

$$\begin{aligned}
q' &= p_p' + s_p'\cot(45^\circ - \phi/2) \\
&= s_p'\cot\phi - c\cot\phi + s_p'\cot(45^\circ - \phi/2) \\
&= s_p'\cot\phi - c\cot\phi + s_p'\left[\frac{1+\sin\phi}{\cos\phi}\right] \\
&= s_p'\left[\frac{\cot\phi\,\cos\phi + 1 + \sin\phi}{\cos\phi}\right] - c\cot\phi \\
&= \left[\frac{ce^{2\theta\tan\phi}\cos^2\phi}{1 - \sin\phi\sin(2\eta+\phi)}\right]\left[\frac{1+\sin\phi}{\cos\phi\sin\phi}\right] - c\cot\phi \\
&\quad + \left[\frac{p_o\sin\phi\,\cos\phi\; e^{2\theta\tan\phi}}{1 - \sin\phi\,\sin(2\eta+\phi)}\right]\left[\frac{1+\sin\phi}{\cos\phi\,\sin\phi}\right] \\
&= \frac{c\cot\phi\; e^{2\theta\tan\phi}(1+\sin\phi) - c\cot\phi + c\cot\phi\sin\phi\sin(2\eta+\phi)}{[1 - \sin\phi\,\sin(2\eta+\phi)]} \\
&\quad + p_o\left[\frac{e^{2\theta\tan\phi}\;(1+\sin\phi)}{1 - \sin\phi\,\sin(2\eta+\phi)}\right] \\
&= c\cot\phi\left\{\frac{e^{2\theta\tan\phi}(1+\sin\phi) - [1 - \sin\phi\sin(2\eta+\phi)]}{[1 - \sin\phi\,\sin(2\eta+\phi)]}\right\} \\
&\quad + p_o\left[\frac{(1+\sin\phi)\; e^{2\theta\tan\phi}}{1 - \sin\phi\,\sin(2\eta+\phi)}\right]
\end{aligned}$$

$$q' = c\left[\cot\phi\left\{\frac{(1+\sin\phi)\,e^{2\theta\tan\phi}}{1-\sin\phi\,\sin(2\eta+\phi)}\right\}-1\right]$$

$$+\;p_o\left[\frac{(1+\sin\phi)\,e^{2\theta\tan\phi}}{1-\sin\phi\,\sin(2\eta+\phi)}\right] \qquad (12)$$

APPENDIX A3

(a). The computation of equation (31):

$$\Delta\sigma_1 = \int_o^R \frac{3q(2\pi\ rdr)}{2\pi Z^2}\left[\frac{1}{1+(r/Z)^2}\right]^{5/2} = \frac{3q}{Z^2}\int_o^R r\left[\frac{1}{\frac{Z^2+r^2}{Z^2}}\right]^{5/2} dr$$

$$= \frac{3qZ^5}{Z^2}\int_o^R \frac{rdr}{(Z^2+r^2)^{5/2}} = 3qZ^3\left[\frac{(Z^2+r^2)^{1-5/2}}{2-5}\right]_o^R$$

$$= 3qZ^3\left[\frac{(Z^2+r^2)^{-3/2}}{-3}\right]_o^R = -qZ^3\left[(Z^2+R^2)^{-3/2} - (Z^2)^{-3/2}\right]$$

$$= -qZ^3\left[\frac{1}{(Z^2+R^2)^{3/2}} - \frac{1}{(Z^2)^{3/2}}\right]$$

$$= -qZ^3\left(\frac{1}{Z^2}\right)^{3/2}\left\{\left[\frac{1}{1+(R/Z)^2}\right]^{3/2} - 1\right\}$$

$$= q\left\{1 - \left[\frac{1}{1+(R/Z)^2}\right]^{3/2}\right\} \qquad (31)$$

Applied Formula: $\displaystyle\int\frac{udu}{(u^2 \pm a^2)^{n/2}} = \frac{(u^2 \pm a^2)^{1-n/2}}{2-n} + C$

(b). The computation of Equation (35), from equation (33)

$$\rho_c = m_v Uq \int_o^Z \left\{ 1 - \left[\frac{1}{1 + (R/Z)^2} \right]^{3/2} \right\} dz \tag{33}$$

$$\int_o^Z \left\{ 1 - \left[\frac{1}{1 + (R/Z)^2} \right]^{3/2} \right\} dz = \int_o^Z \left[1 - \left(\frac{Z^3}{Z^2 + R^2} \right)^{3/2} \right] dz$$

$$= \int_o^Z \left[1 - \frac{Z^3}{(Z^2 + R^2)^{3/2}} \right] dz = \int_o^Z dz - \int_o^Z \frac{Z^3}{(Z^2 + R^2)^{3/2}} dz$$

$$= \Big[2 \Big]_o^Z - \left[\frac{Z^{3-1}}{(3-3+1)(Z^2 + R^2)^{3/2-1}} - \frac{R^2(3-1)}{3-3+1} \int_o^Z \frac{Z^{3-2}\, dz}{(Z^2+R^2)^{3/2}} \right]$$

$$= Z - \left[\frac{Z^2}{(Z^2+R^2)^{1/2}} - 2R^2 \int_o^2 \frac{Z}{(Z^2+R^2)^{3/2}} dz \right]$$

$$= Z - \left\{ \frac{Z^2}{(Z^2+R^2)^{1/2}} - 2R^2 \left[\frac{(Z^2+R^2)^{1-3/2}}{2 - 3} \right]_o^2 \right\}$$

$$= Z - \left\{ \frac{Z^2}{(Z^2+R^2)^{1/2}} - 2R^2 \left[\frac{1}{(Z^2+R^2)^{1/2}} \right] \right\}$$

$$= Z - \left[\frac{Z^2 + 2R^2}{(Z^2+R^2)^{1/2}} - 2R \right]$$

$$\therefore \rho_c = m_v Uq \int_o^Z \left\{ 1 - \left[\frac{1}{1 + (R/Z)^2} \right]^{3/2} \right\} dz$$

$$= m_v Uq \left\{ Z - \left[\frac{Z^2 + 2R^2}{(Z^2 + R^2)^{1/2}} - 2R \right] \right\} \tag{35}$$

Applied formulas:

$$\int \frac{U^m du}{(u^2 \pm a^2)^{n/2}} = \frac{U^{m-1}}{(m-n+1)(u^2 \pm a^2)^{n/2-1}} - \frac{\pm a^2(m-1)}{(m-n+1)} \int \frac{u^{m-2} du}{(u^2 \pm a^2)^{n/2}}$$

$$\int \frac{u du}{(u^2 \pm a^2)^{n/2}} = \frac{(u^2 \pm a^2)^{1-n/2}}{2 - n} + C$$

(c). The computation of equation (36), from equation (34).

$$\rho_c = m_v Uq \int_o^Z \frac{B^2}{(B + Z)^2} \, dz \qquad (34)$$

$$\int_o^Z \frac{B^2}{(B + Z)^2} \, dz,$$

Let $B + Z = x$, $Z = x - B$, $dz = dx$

$$\int_o^Z \frac{B^2}{(B + Z^2)} \, dz = B^2 \int_o^Z x^{-2} \, dx$$

$$= B^2 \left[\frac{-1}{x} \right]_o^Z = - B^2 \left[\frac{1}{B + Z} \right]_o^Z$$

$$= -B^2 \left[\frac{1}{B + Z} - \frac{1}{B} \right] = \frac{BZ}{B + Z} = \frac{Z}{1 + Z/B}$$

$$\therefore \; \rho_c = m_v Uq \int_o^Z \frac{B^2}{(B + Z)^2} \quad dz = m_v Uq \, \frac{Z}{1 + Z/B} \qquad (36)$$

APPENDIX B

(Taking from Ref. 16)

Compressibility of Various Types of Clays

Type	Qualitative description	Coefficient of Vol.compressibility (m_v) ft per ton
Heavily over-consolidated boulder clays (e.g. many Scottish boulder clays) and stiff weathered rocks(e.g. weathered siltstone), hard London Clay, Gault Clay and Oxford Clay (at depth)	Very low compressibility	Below 0.005
Boulder clays (e.g. Tees-side, Cheshire) and very stiff "blue" London Clay, Oxford Clay, Keuper Marl	Low compressibility	0.005 to 0.01
Upper "blue" London Clay, weathered "brown" London Clay, fluvio-glacial clays, Lake clays, weathered Oxford Clay, weathered Boulder Clay, weathered Keuper Marl, normally consolidated clays (at depth)	Medium compressibility	0.01 to 0.03
Normally-consolidated alluvial clays(e.g. estuarine clays of Thames, Firth of forth, Bristol Channel, Shatt-al-Arab, Niger Delta, Chicago Clay), Norwegian "Quick" Clay	High compressibility	0.03 to 0.15
Very organic alluvial clays and peats	Very high compressibility	Above 0.15

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AN EXAMINATION OF THE INFLUENCE OF FOUNDATION SIZE ON ULTIMATE BEARING CAPACITY AND SETTLEMENT

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ABSTRACT

The object of this report is to present methods for the examination of the influence of foundation size on bearing capacity and settlement.

The first part presents a theory of bearing capacity based on elastic and plastic equilibrium of shallow foundations. The bearing capacity of foundations in purely cohesive material is found to be a constant and is independent of the width of the footing while in cohesionless material, bearing capacity increases rapidly with foundation size.

The second part is concerned with the settlement of a foundation. If the contact pressure is constant, settlement is directly propertional to the foundation size.

In cohesive soils settlement consists of an "immediate" settlement without volume change and the more important "consolidation" occuring by the dissipation of excess pore pressure over a long period of time. In cohesionless soils all settlements are of the "immediate" type since excess pore pressure can not develop. In all soils the amount of settlement is a function of the contact pressure, the slope of the compression curve, and the initial void ratio.