Bearing Capacity of Strip Footing Supported by Two-Layer c-ϕ Soils

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The ultimate bearing capacity of an embedded strip footing supported by two-layer c-ϕ soils has been investigated using an elasto-plastic finite-element computer program. In the program the footing material is treated as linear elastic and the foundation soils are idealized as nonlinear elastic perfectly plastic materials that obey the yield criterion of Drucker and Prager. The program also considers initial stresses, interface behavior, and tension failure of soil. The analysis was performed on a VAX 11/785 computer. Three soils—a commercial kaolin, a silty clay, and a clayey sand—were selected for analysis of a representative number of soil layer combinations. The analysis was performed for a constant footing width and varying levels of top-layer thickness. The propagation of plastic yield zones under progressive increments of footing load was investigated, and a generalized procedure for determining bearing capacity from computer-generated pressure-settlement curves was proposed. Meanwhile, on the basis of the analysis results, a semiempirical equation was developed for determining ultimate bearing capacity. A comparison with existing theories indicates that the developed equation can provide more-reasonable results than the existing ones can. Additionally, the developed equation is relatively simple and can be easily applied.

Shallow foundations are sometimes located on a soil layer of finite thickness overlying a thick stratum of another soil. The underlying stratum may be either a bedrock or another soil possessing different strength properties. The bearing stratum of the two-layer deposits can be either softer or stiffer than the underlying stratum. If the footing rests on a relatively thin stiff layer above a soft deposit, it may punch through the top layer into the underlying stratum. In such a case, the settlement and ultimate bearing-capacity behavior of the footing will, to a large extent, be governed by the strength characteristics of the underlying stratum. On the other hand, for a footing resting on a relatively thick soft layer overlying a stiff layer, bearing-capacity failure may be limited to the top layer. In both situations, the settlement and bearing-capacity characteristics of the footing will be greatly influenced by the thickness of the top stratum and the strength properties of the two soil layers.

Bearing-capacity theories for two-layer soil deposits are scarce in published literature, and the theories that have been proposed have limitations that restrict their use to idealized conditions. For example, most of the proposed equations apply either to purely cohesive ($\phi = 0$) two-layer soils or to soil layers made up of a cohesionless ($c = 0$) and a cohesive ($\phi = 0$) soil. Furthermore, the proposed equations for two-

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layer clay soils usually assume that the top layer is stiffer than the bottom layer, that is, that $c_1 > c_2$.

This paper presents the results of a study involving two-layer c-ϕ soils in which the top layer may be either stiffer or softer than the bottom layer. The results of analyses have been used to formulate a semiempirical bearing-capacity equation, the results of which have been compared with the existing equations.

EXISTING THEORIES

Researchers have proposed relationships for predicting the ultimate bearing capacity of footings on homogeneous two-layer soils. However, as mentioned, all of the proposed equations have limitations. A review of the available bearing-capacity equations and their limitations for two-layer soils is presented in subsequent sections.

Bearing-Capacity Theories for Purely Cohesive Two-Layer Soils

The bearing capacity of strip footings on two-layer clay soils, for a stiff layer overlying a soft stratum and for its converse, has been analyzed by Button (1). He assumed a general shear failure along cylindrical slip surfaces that emanate from the edges of the footing. He presented modified bearing-capacity factors (N_c) for ϕ-soils for various values of c_2/c_1, where c_1 and c_2 are the undrained cohesions (or undrained shear strengths) of the top and bottom layers, respectively. However, experimental data of Brown and Meyerhof (2) showed that Button's (1) assumed failure modes were unrealistic and that the resulting bearing-capacity factors were on the unsafe side. They proposed the following bearing-capacity equation for conditions in which the bearing stratum can be softer or stiffer than the underlying stratum:

$$q_u = c_1 N_m + q \tag{1}$$

where

q_u = ultimate bearing capacity,
q = overburden pressure equal to the unit weight of soil times the depth of foundation, and
N_m = modified bearing-capacity factor that depends on c_2/c_1, the relative thickness of the upper layer (H_1/B), and the footing shape.

Meyerhof and Hanna (3) have proposed a bearing-capacity theory for a strip footing supported by a two-layer clay in which the top layer is stronger than the bottom layer. In their failure mechanism, it is assumed that if the top layer is relatively thin, failure takes place when it is punched through and the bottom layer undergoes a general shear failure. However, if the top layer is relatively thick, the failure surface will be fully contained in the top clay layer. Thus, the ultimate bearing capacity of a strip footing can be expressed as

$$q_u = c_2 N_c + \frac{2c_a H_1}{B} + \gamma_1 D_f \leq q_t \tag{2}$$

where

$q_t = c_1 N_c + \gamma_1 D_f,$
$N_c = \text{bearing capacity factor} = 5.14,$
$D_f = \text{depth of foundation, and}$
$c_a = \text{soil adhesion, a function of } c_2/c_1.$

Bearing-Capacity Theories for Sand Overlying Cohesive Soils

Tcheng (4) has proposed a bearing-capacity equation for a long rectangular footing resting on a sand layer that is underlain by a purely cohesive soil layer. He reported good agreement between his test results and the proposed equation within the domain $H_1 < 1.5B$. He further reported that the influence of the bottom clay layer on bearing capacity becomes negligible when $H_1 \geq 3.5B$. Tcheng's equation is as follows:

$$q_o = q_o'' \left\{ 1 - 2(H_1/B)\tan \phi (1 + \sin \phi) \right.$$
$$\left. \times \exp\left[-\left(\frac{\pi}{4} - \frac{\phi}{2}\right) \tan \phi \right] \right\} \tag{3}$$

where

$q_o = \text{bearing capacity of a long rectangular footing resting on the sand layer,}$
$q_o'' = \text{bearing capacity of the same footing resting on the underlying clay layer, and}$
$\phi = \text{angle of internal friction.}$

Vesic (5) has proposed a more general bearing-capacity equation, which is valid for rectangular footings resting on a top layer with strength parameters (c_1, ϕ_1) that is underlain by a weaker layer with strength parameters (c_2, ϕ_2). It is shown as

$$q_o = [q_o'' + (1/K)c_1 \cot \phi_1] \exp[2(1 + B/L)$$
$$\times K \tan \phi_1 (H_1/B)] - (1/K)c_1 \cot \phi_1 \tag{4}$$

where

$$K = \frac{1 - \sin^2\phi_1}{1 + \sin^2\phi_1},$$
$L = \text{footing length, and}$

$q_o'' = \text{bearing capacity of a fictitious footing of the same size and shape as the actual footing but resting on the top of bottom layer.}$

Meyerhof (6) and Hanna and Meyerhof (7) have studied the ultimate bearing capacity of footings on either a loose or a dense sand layer overlying a clay and have compared the different modes of soil failure with the results of model tests on circular and strip footings. For a strip footing resting on a dense sand that overlies a soft clay, the bearing capacity was found to increase with sand-layer thickness until about $H_1/B \geq 2.5$ (6); thereafter, it remained constant at a value equal to the ultimate bearing capacity of an infinitely thick dense sand layer. For a strip footing resting on a dense sand layer that overlies a firm clay, the test results showed a decrease in the ultimate bearing capacity with increasing sand-layer thickness. The bearing capacity decreases from the initial maximum value (for a footing on an infinite clay layer) to the minimum for a footing on a thick sand deposit. For this case, Meyerhof (6) has proposed the following equations:

$$q_u = q_t + (q_b - q_t)(1 - H_1/H_f)^2 \tag{5}$$

with a maximum for $H_1/B = 0$

$$q_u = q_b = cN_c + \gamma D \tag{6}$$

and a minimum for $H_1/B \geq H_f/B$

$$q_u = q_t = \frac{1}{2} \gamma B N_\gamma + \gamma D N_q \tag{7}$$

where H_f is the failure depth.

Satyanarayana and Garg (8) have proposed a simplified bearing-capacity theory for shallow foundations in c-ϕ soils. According to their theory, the ultimate bearing capacity of a two-layer soil is given by

$$q_u = c_{av} N_c + \gamma_1 D N_q + \frac{1}{2} \gamma_1 B N_\gamma \tag{8}$$

where

$$c_{av} = \frac{H_1 c_1 + H_2 c_2}{H_1 + H_2},$$

$$\phi_{av} = \tan^{-1}\left(\frac{H_1 \tan \phi_1 + H_2 \tan \phi_2}{H_1 + H_2}\right),$$

$$H_2 = (2B - H_1)\left(\frac{c_1 + \tan \phi_1}{c_2 + \tan \phi_2}\right), \text{ and}$$

$N_c, N_q, N_\gamma = \text{bearing capacity factors based on } \phi_{av}.$

As is seen from the review, only one equation proposed by Satyanarayana and Garg (8) is available for predicting the ultimate bearing capacity of two-layer c-ϕ soils. For this equation, the proposers even caution abut applying the equation to all soils.

FINITE-ELEMENT ANALYSIS

A two-dimensional plane strain elasto-plastic finite-element computer program was used for analysis. The program uses incremental stress-strain relations for elastic perfectly plastic materials and either Mohr-Coulomb or Drucker-Prager (9) yield criteria to define soil yielding. It was specifically adapted and modified to include (a) capability for incorporating geostatic (initial) stresses to represent the stress state in the soil mass before the application of boundary loads, (b) interface elements to satisfy displacement compatibilities along the vertical boundaries between the foundation and soil, (c) footing material (reinforced concrete) modeled as a linearly elastic material, and (d) soil modeled as nonlinearly elastic in the elastic range. A complete description of the numerical and mathematical formulations of the program, including validation, is available elsewhere (10).

The finite-element computer program was used to investigate the behavior of a reinforced concrete strip footing having $B = 3.0$ ft (91.4 cm) and $D_f = 3.0$ ft (91.4 cm), shown schematically in Figure 1. The footing was supported by a two-layer soil deposit having four different layer combinations: a soft clay underlain by a stiff clay, a stiff clay underlain by a soft clay, a clayey sand underlain by a stiff clay, and a stiff clay underlain by a clayey sand. The stiff clay was a compacted kaolin, the soft clay was a compacted silty clay, and the clayey sand was a compacted mixture of 90 percent sand and 10 percent kaolin. The strength properties of the three soils used in the analysis are summarized in Table 1; determinations of these strength properties are documented elsewhere (10).

In the analysis, the entire soil-footing system was represented by a finite number of eight-node quadrilateral elements interconnected at the nodal points. Because in a continuous footing, a plane of symmetry exists along the vertical-footing axis, only half of the model needed to be analyzed. The nodal
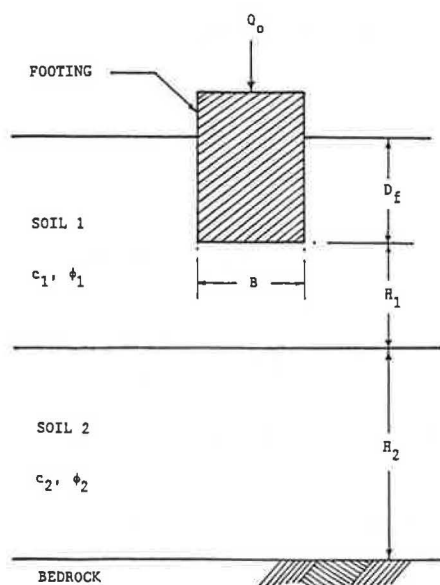

FIGURE 1 Schematic of footing-soil geometry.

points along the vertical side boundaries were restrained in the x-direction; those on the bottom boundary were restrained in x- and y-directions.

To accommodate the nonlinear stress-strain characteristics of the foundation soils in the finite-element analysis, the geostatic stress and footing load were applied in increments. At the initial stage of loading, increments of no more than 10 percent of the total footing load were used; at the final stage, the increments were reduced to 1 percent. For each increment, generally three to five iterations of computations were needed. The computer analysis was performed on a VAX 11/785 computer.

The finite-element analysis provides stresses, strains, and yield condition of each element, pressure-versus-settlement (p-δ) relationship of the footing and contact pressure at footing-soil interface, among others. The p-δ relationships were used to obtain the ultimate bearing capacity for each condition analyzed. As an illustration, a set of typical p-δ curves for soft clay underlain by stiff clay, and its converse, for $H_1/B = 1$ and 8 are shown in Figure 2. These curves clearly demonstrate the variation in slope (stiffness) and the change in ultimate bearing capacity with changing H_1/B for each layer combination. Such curves are plotted for each condition analyzed, and from these curves the ultimate bearing capacities are determined using the criterion discussed in the following. These ultimate bearing-capacity values form the basis for subsequent presentation and discussion.

ULTIMATE LOAD CRITERION

The p-δ curves obtained from the finite-element analysis do not always show a distinct point for the determination of ultimate bearing capacity. The point on the p-δ curve indicating ultimate bearing capacity depends on the mode of failure, such as general, local, or punching shear failure, which in turn depends on soil type, its relative density, compressibility, footing width, depth of foundation, and so on. Determining ultimate bearing capacity from p-δ curves, therefore, requires some degree of judgment and experience. Researchers have proposed different approaches. For example, Vesic (11) has recommended that ultimate load can be taken as the point where the slope of the p-δ curve first reaches zero or steady minimum value. Desai and Christian (12) have proposed either using the concept of critical (or tolerable) settlement or choosing the load corresponding to the intersection of the tangents with the initial and ultimate portions of the p-δ curve. However, no single criterion is general enough to apply to all types of soils and all modes of failure.

Consideration of the critical-settlement criterion is justified by the basic philosophy of foundation design, which regards the excessive settlement as a failure of the foundation. There is no single value of critical settlement that satisfies failure conditions in all types of soils and all modes of failure. Observations in saturated clays (13) indicate that these settlements may be between 3 and 7 percent of footing width (B) for surface footings, increasing to 15 percent for embedded footings. For footings in sand, somewhat higher values—ranging from 5 to 15 percent for surface footings and as high as 25 percent for embedded footings—have been proposed (14,11). Das (15) has suggested that for foundations at shallow depth

TABLE 1 MATERIAL PROPERTIES OF FOUNDATION SOILS (1 psi = 6.9 kN/m²)

Material Properties	Silty Clay	Kaolin	Clayey Sand
Initial Modulus in Compression, psi	667	2,880	6,100
Poisson's ratio	0.28	0.39	0.32
Dry Unit weight, pci	0.058	0.052	0.061
Unit cohesion, psi	9.5	23.0	1.33
Internal friction angle, deg.	13.5	8.0	31.0
Initial earth pressure coefficient	0.60	0.64	0.86
Tensile strength, psi	2.00	7.00	0.54
Initial modulus in tension, psi	1,505	7,000	11,300
Soil constant (R$_f$)	0.80	0.77	0.86
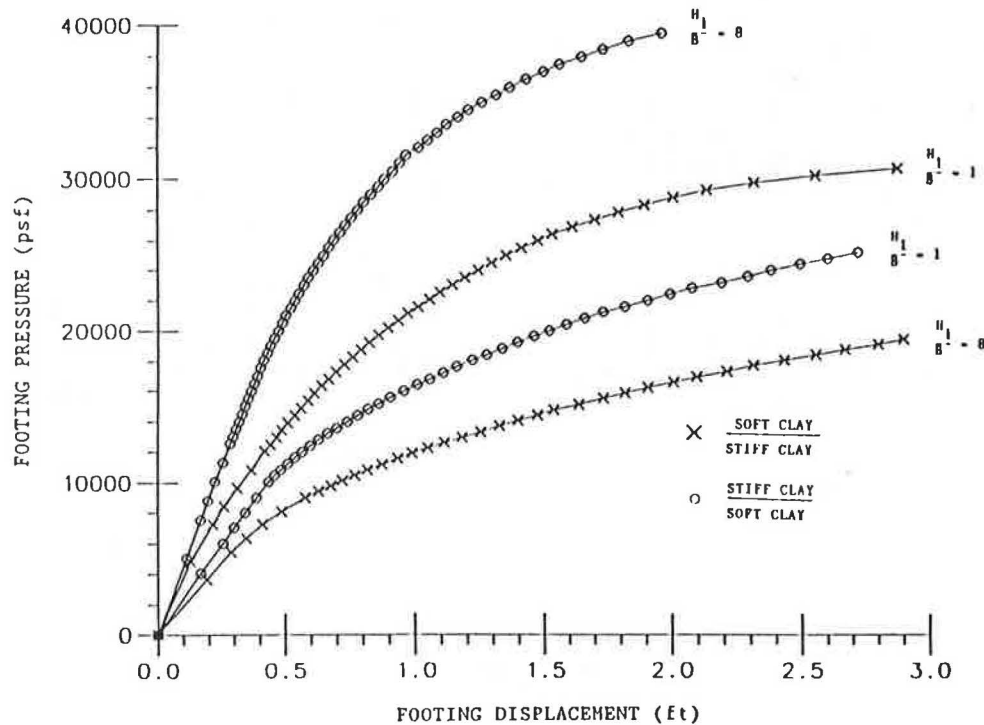
FIGURE 2 Comparison of p-δ curves for soft clay underlain by stiff clay and its converse for $H_1/B = 1$ and 8 ($B = 3.0$ ft, $D_f = 3.0$ ft).

and for those likely to fail by general shear, the ultimate load may occur at foundation settlement of 4 to 10 percent of footing width (B); however, for local or punching shear failure, the ultimate load may occur at settlements of 15 to 24 percent of B. Thus, in the selection of a particular value of critical settlement, one should take into account not only the depth of foundation but also the soil type and its likely failure mode. On the basis of these considerations and the fact that the soils of this study are not uniform but layered and contain a highly compressible clay and a sandy soil, a critical settlement equal to 20 percent of B was chosen as one of the criteria for determining the ultimate bearing capacity.

In addition, because the ultimate load takes place in the state of failure, it is reasonable to take into account the rate of yielding in the determination of the ultimate bearing capacity. Such an approach, in which the yielding rate of soil

elements is considered in conjunction with other techniques, has been adopted here. The number of yielded elements is plotted against the applied pressure, and the yielding rate is observed from the slope of this curve. The point at which the curve exhibits a distinct change in the average slope, signifying accelerated plastic yielding, is an indication of the ultimate bearing capacity. The overall technique for determining the ultimate bearing capacity from p-δ curves adopted in this study is summarized in the following:

1. If the p-δ curve shows a distinct yield point, then that point is taken to signify the ultimate bearing capacity. Such a curve would satisfy Vesic's criterion (11).

2. If the curve exhibits no distinct yield point, as in Figure 3, but instead appears to suggest continued penetration, then the following steps are executed:

–The point given by the intersection of tangents to the initial and ultimate portions of the p-δ curve is noted (Figure 3). This gives one possible value of ultimate bearing capacity.

–The point that indicates an increase in the rate of element yielding on the pressure–versus–number of yielded elements curve, such as Point A in Figure 4, is noted. This gives a second value of ultimate bearing capacity.

–Whichever point is the lesser value is chosen as the ultimate bearing capacity.

3. If the chosen value is greater than that dictated by settlement considerations (i.e., pressure at a settlement of 20 percent of B), then the latter value is taken as the ultimate bearing capacity.

SPREAD OF PLASTIC YIELD ZONES

Study of the plastic flow behavior gives insight into the progressive yielding of soil under a load and can indicate the dominant failure mode. Selected figures depicting the spread of plastic yield zones in two-layer soils are presented here. Figures 5–8 illustrate plastic yielding for a weak clay layer underlain by a stiff clay and its converse for two levels of the ratio of top-layer thickness to footing width, that is, $H_1/B = 1$ and 4.

Figure 5 shows that when the top layer is thin ($H_1/B = 1$), the yield zones at the collapse stage extend into the bottom layer and propagate to the surface. The spread of yield zones to the ground surface indicates that at the collapse stage the soil yielding is analogous to a plastic flow under squeezing action. A yield-zone pattern of this type indicates a possible general shear failure involving both soil layers. When the top clay layer becomes thick ($H_1/B = 4$), as in Figure 6, the

yielding is confined to the top layer. The yield zones appear to stop spreading into the stiff clay layer below, indicating that the elements in the lower layer have not yet reached their yield state. This behavior suggests that at $H_1/B = 4$, the ultimate bearing capacity will be dictated predominantly by the strength properties of the top layer, though some influence of the lower layer may still be present. The yield pattern of Figure 6 further indicates that the failure mode is a possible local shear failure confined to the top layer.

Figures 7 and 8 show a stiff clay underlain by a weak clay for $H_1/B = 1$ and 4, respectively. These figures reveal that the yield zones extend deep into the weaker layer below even when the top layer is sufficiently deep (e.g., $H_1/B = 4$). This yield pattern is typical of a punching shear failure of the top layer followed by a general shear failure of the bottom layer. Thus, the bearing capacity of such a layer combination will be largely controlled by the strength properties of both layers.

DISCUSSION OF RESULTS

The objective of this analysis was to establish the relationship between the ultimate bearing capacity (q_o) and top-layer thickness for various soil layer combinations. Accordingly, four soil-footing systems ($H_1/B = 1, 2, 3,$ and 8) were analyzed for the four two-layer combinations. Figure 9 shows the variation of ultimate bearing capacity (q_o) with H_1/B for two clay layer soils—soft clay over stiff clay and stiff clay over soft clay. It is noted from this figure that when the top layer is a soft clay, the bearing capacity decreases with an increase in the top-layer thickness and ultimately attains a steady value equal to its own bearing capacity at approximately $H_1/B = 6$. The yield patterns given in Figures 5 and 6 can be used to
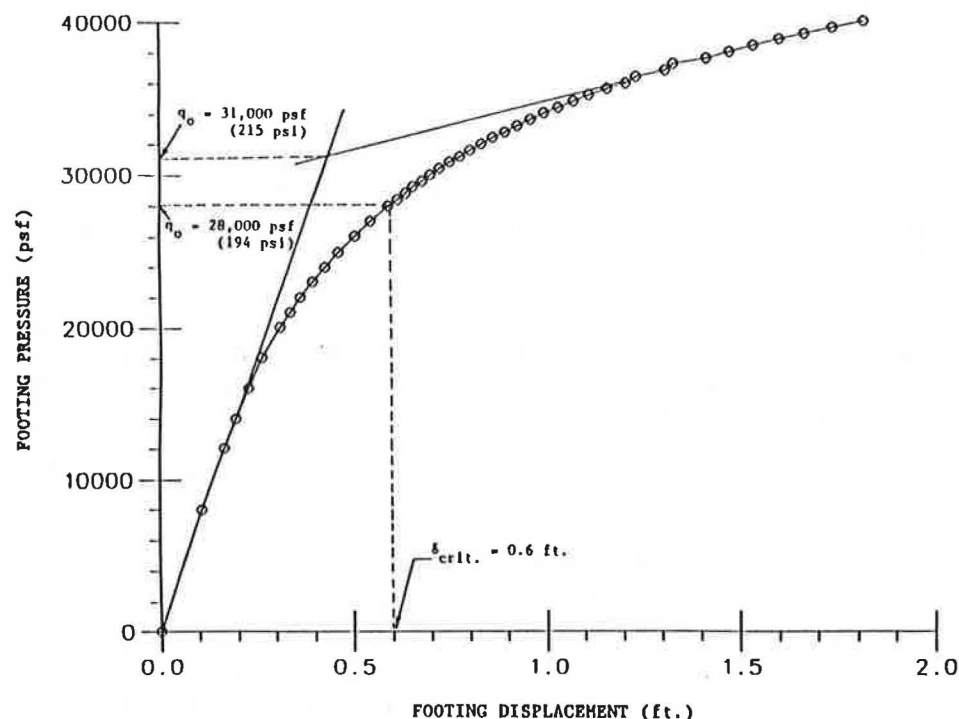
FIGURE 3 p-δ curve for uniform kaolin, illustrating the application of ultimate-bearing-capacity criterion ($B = 3.0$ ft, $D_f = 3.0$ ft).
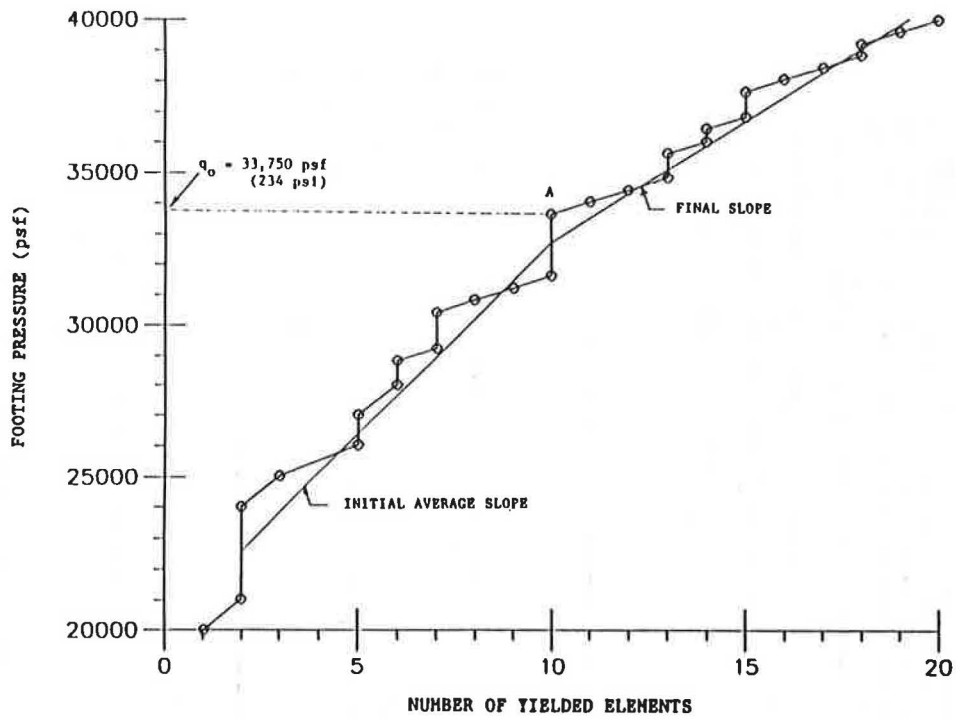
FIGURE 4 Pressure-element yield rate curve for uniform kaolin, illustrating the application of ultimate-bearing-capacity criterion.
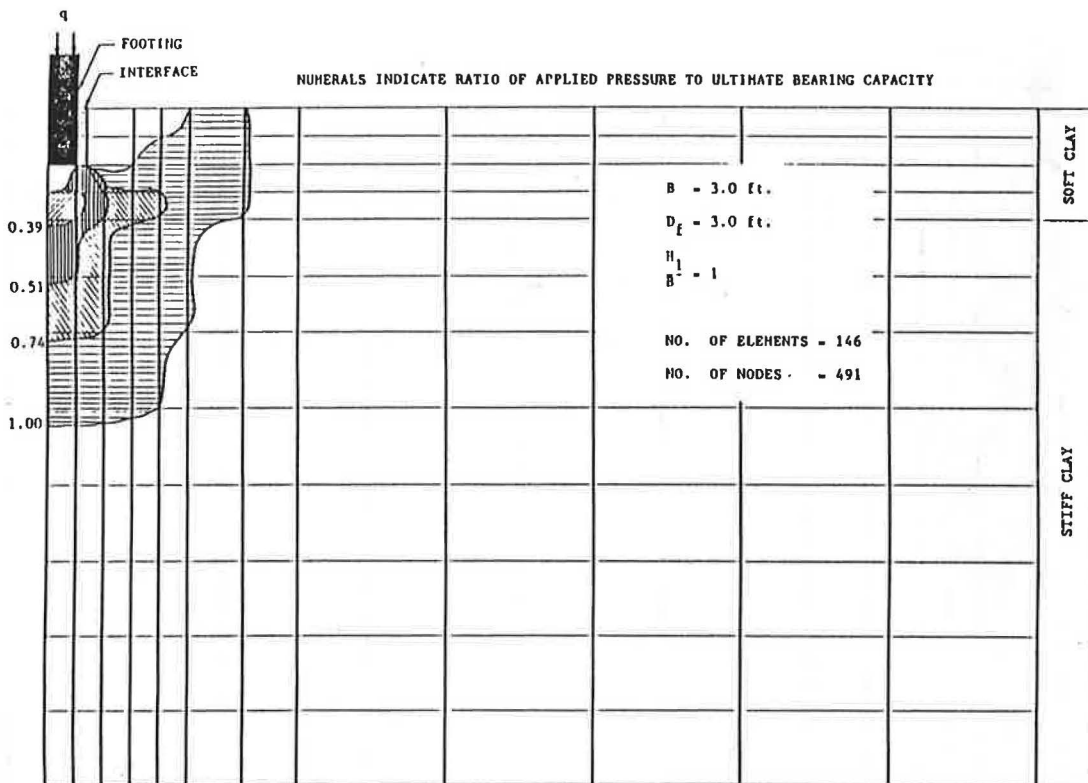

FIGURE 5 Spread of plastic yield zones at various pressure levels in two-layer soil with soft over stiff clays and $H_1/B = 1$.
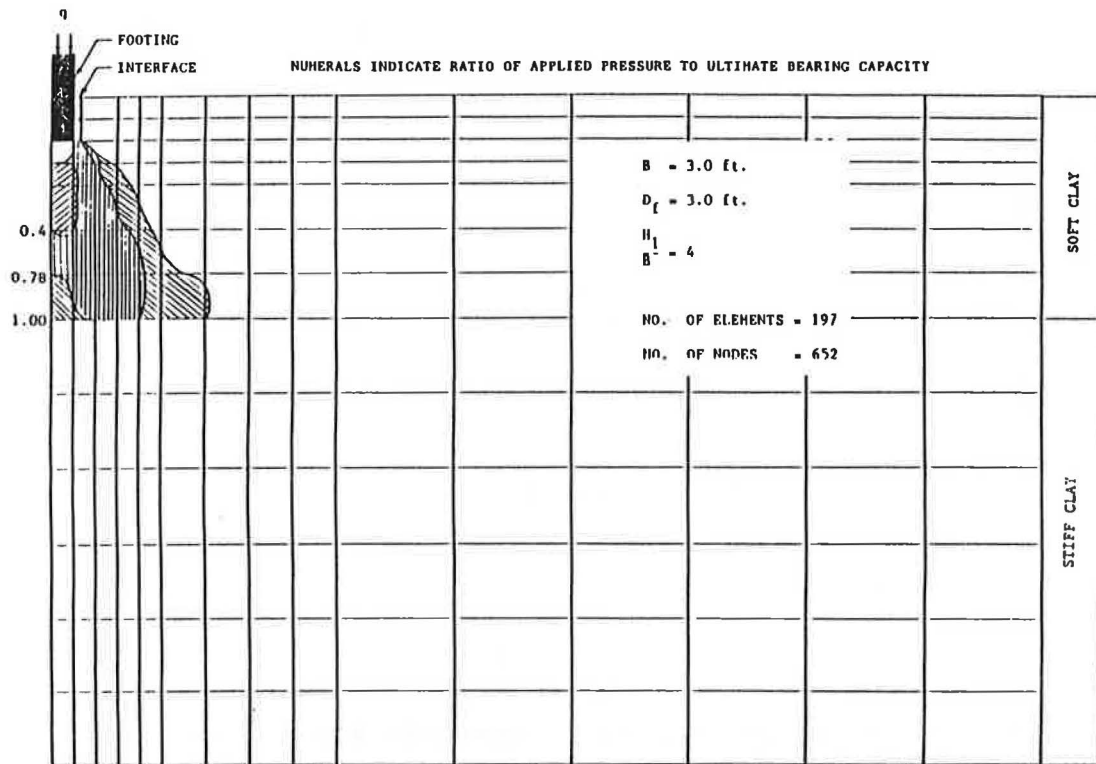
FIGURE 6 Spread of plastic yield zones at various pressure levels in two-layer soil with soft over stiff clays and $H_1/B = 4$.
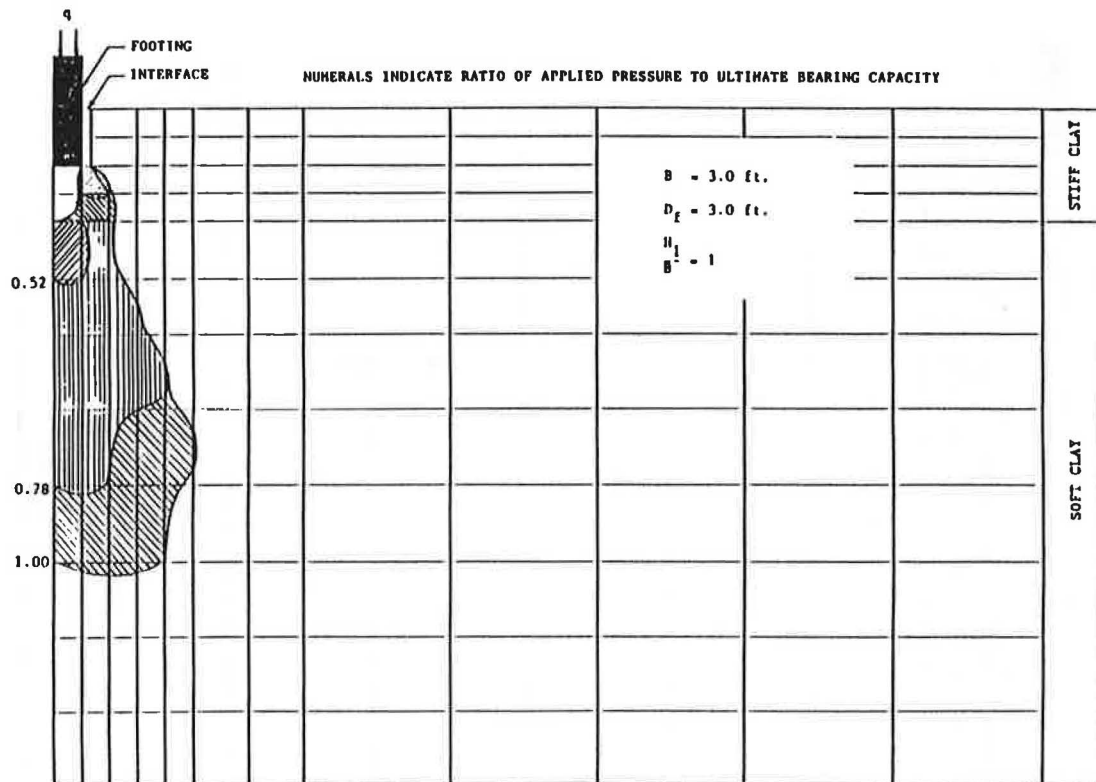

FIGURE 7 Spread of plastic yield zones at various pressure levels in two-layer soil with stiff over soft clays and $H_1/B = 1$.
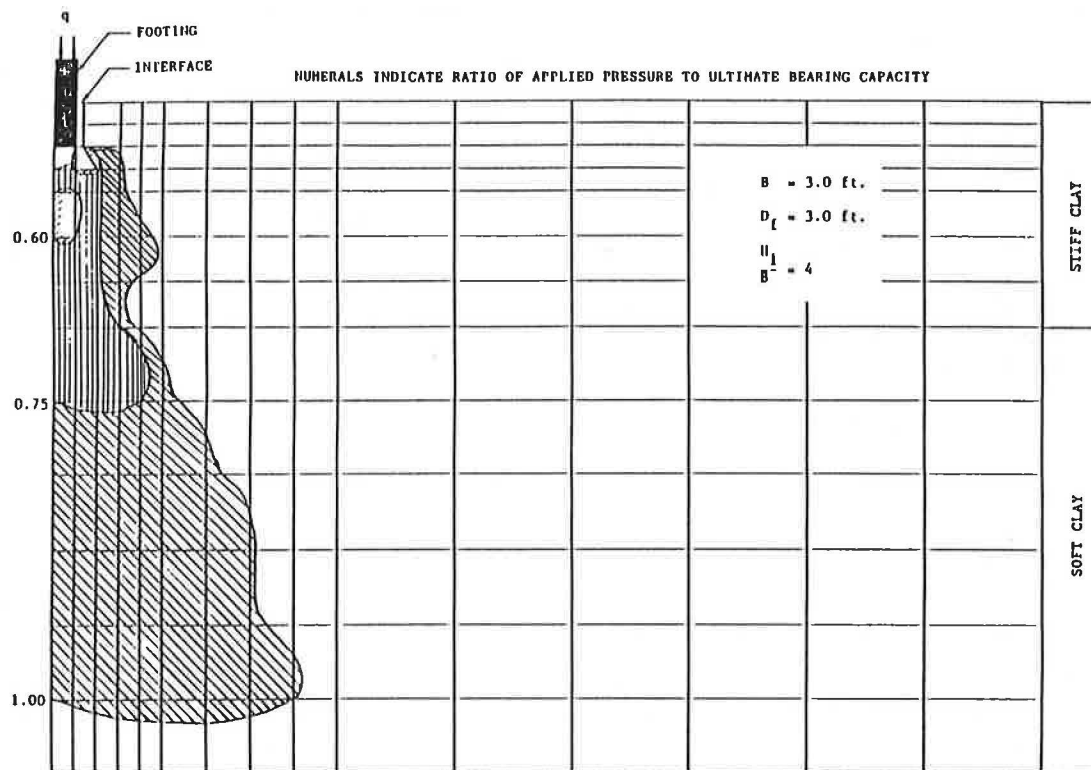
FIGURE 8 Spread of plastic yield zones at various pressure levels in two-layer soil with stiff over soft clays and $H_1/B = 4$.
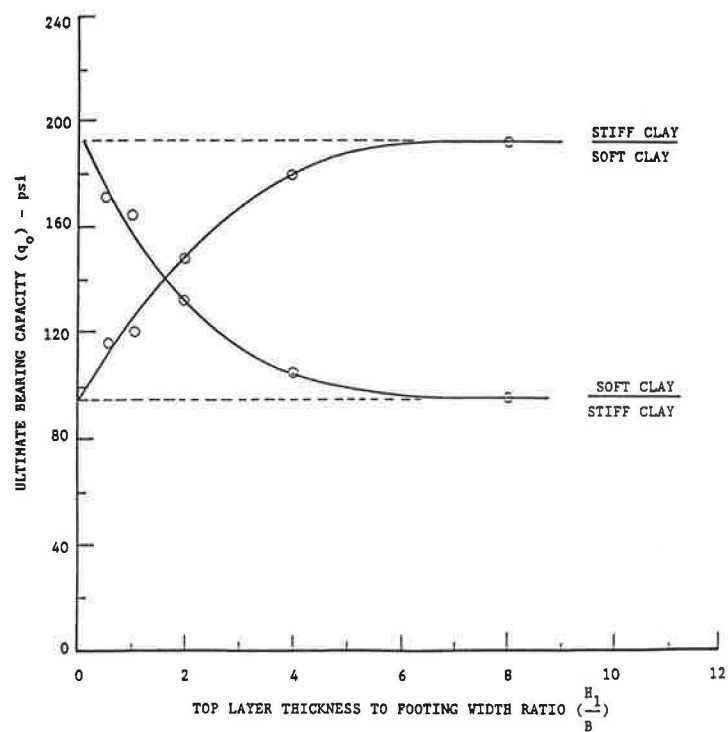

FIGURE 9 Variation of ultimate bearing capacity with top-layer thickness for soft clay over stiff clay and its converse ($B = 3.0$ ft, $D_f = 3.0$ ft, $H_2 = 30.0$ ft).

explain the observed behavior. The greater bearing capacity at a smaller H_1/B ratio is possibly because at small H_1/B ratios, the failure zone extends into the lower stiff clay layer and the higher strength of the lower layer contributes toward a greater ultimate bearing capacity. However, when the top soft clay becomes thicker, the major portion of the failure zone is within the top layer. As a result, the lower strength of the top layer reduces the ultimate bearing capacity.

The trend of bearing-capacity variation for a stiff clay underlain by a soft clay layer (Figure 9) is almost the exact opposite of the preceding case. The ultimate bearing capacity increases with top-layer thickness and ultimately attains a steady value equal to the bearing capacity of the top layer at approximately $H_1/B = 6$. The explanation for this behavior, as before, is that at small H_1/B ratios the failure zone extends into the bottom layer, whose lower strength reduces the ultimate bearing capacity; at higher H_1/B ratios the failure zone remains confined within the top clay layer, resulting in a greater ultimate bearing capacity.

The variations of ultimate bearing capacity with top-layer thickness for a sand underlain by a stiff clay and its converse are illustrated in Figure 10. The bearing capacity of the former case decreases with increasing sand-layer thickness from a maximum at $H_1/B = 0$ (i.e., the footing rests on the clay layer) to a minimum value equal to that for a footing resting on an infinitely thick sand deposit. The ultimate bearing capacity in this case attains a steady value at about $H_1/B = 3$. This H_1/B value is in good agreement with the value of approximately 2.5 obtained by Meyerhof (9) from model tests on loose sand overlying stiff clay and the value of 3.5 suggested by Tcheng (4).

PROPOSED EQUATION

The bearing-capacity equation is formulated by curve fitting. The data base for formulation is the bearing-capacity–versus–top-layer-thickness relationship discussed earlier and presented in Figures 9 and 10. The variations of bearing capacity seen in these figures dictate that a semiempirical approach is most appropriate for formulating a single equation for all four curves. Of the various relationships attempted, the empirical parabolic interaction relationship appears to best fit the data base. On the basis of this relationship, the following bearing-capacity equation is proposed:

$$q_o = q_t + (q_b - q_t)[1 - m(H_1/B)]^2 \qquad (9)$$

where

q_o = ultimate bearing capacity of strip footing over two-layer soil;

q_t = ultimate bearing capacity of the footing supported by an infinitely thick top-layer soil, computed by the traditional bearing-capacity equation using factors recommended by Vesic (5);

q_b = ultimate bearing capacity of the footing supported by an infinitely thick bottom-layer soil, computed by the same method as q_t;

m = layer factor, which is 0.17–0.23 for two layers of clay (use of the lower value is recommended if one clay layer is highly compressible, otherwise use the average value) and 0.30 for a sand-clay layer combination; and
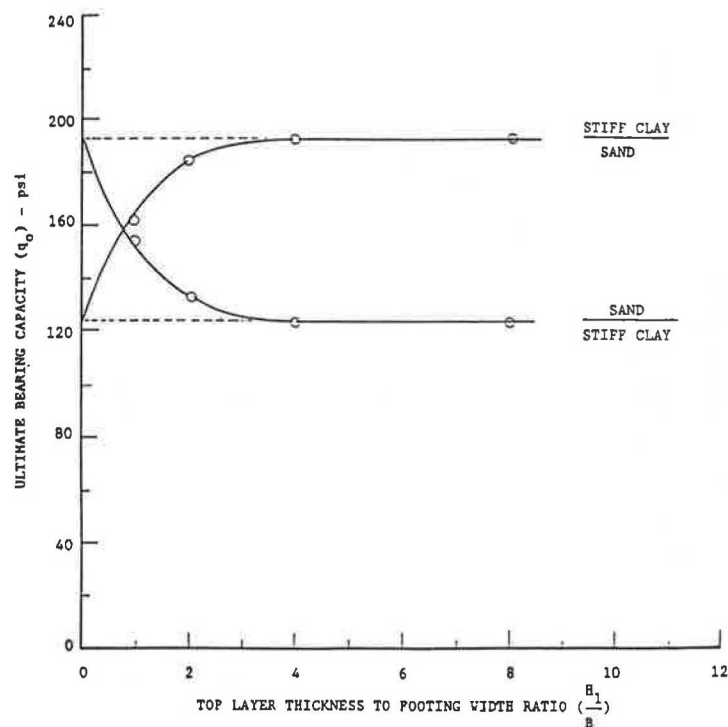

FIGURE 10 Variation of ultimate bearing capacity with top-layer thickness for clayey sand over stiff clay and its converse ($B = 3.0$ ft, $D_f = 3.0$ ft, $H_2 = 30.0$ ft).

H_1/B = top-layer-thickness–to–footing-width ratio, which is no more than 6 for clay-clay layers and no more than 3.5 for sand-clay layers.

The results computed from Equation 9 are compared with the available theories in Figures 11 and 12. The comparison reveals that the bearing-capacity variations predicted from equations of Vesic (5) and Satyanarayana and Garg (8) attain steady values at approximately $H_1/B = 1$ and 2, respectively, signifying that the bottom layer ceases to have any influence beyond these top-layer thicknesses. On the other hand, the bearing capacity determined from Equation 9 approaches a steady value at a gradual rate, indicating a pronounced influence of the bottom-layer soil up to approximately $H_1/B = 6$. Such a gradual variation of bearing capacity with the top-layer thickness appears to be more reasonable.
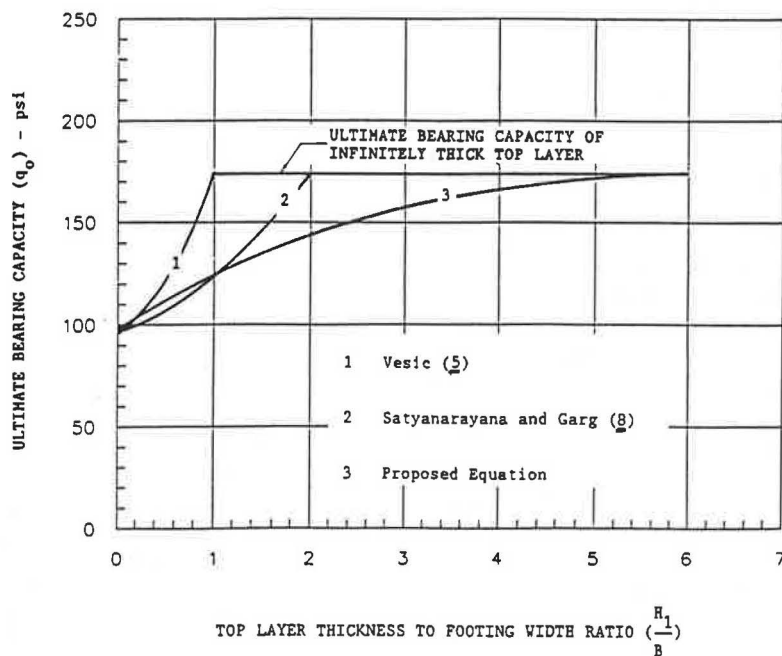
FIGURE 11 Comparison of proposed equation with available theories for stiff clay underlain by soft clay.
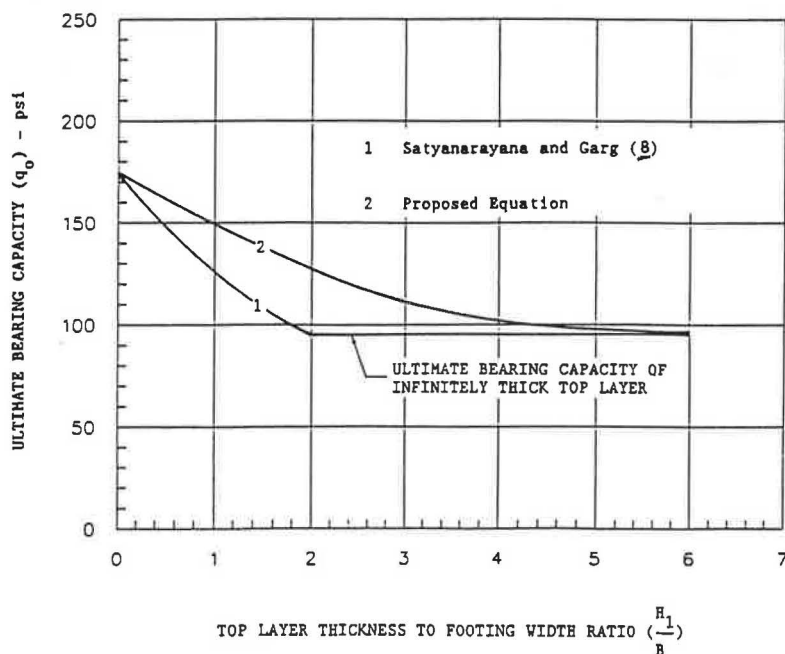
FIGURE 12 Comparison of proposed equation with available theories for soft clay underlain by stiff clay.

SUMMARY AND CONCLUSIONS

The ultimate bearing capacity of a shallow continuous footing supported by two-layer c-ϕ soils has been investigated using an elasto-plastic finite-element computer program. Three soils—a soft clay (silty clay), a stiff clay (kaolin), and a clayey sand (90 percent sand + 10 percent kaolin)—were used to form representative soil layer combinations. The effect of top-layer thickness (H_1) on bearing capacity was investigated for each layer combination. Footing width (B) and depth of foundation (D_f) were kept constant at 3.0 ft (91.4 cm). The results of the analysis led to the formulation of a semiempirical bearing-capacity equation that is simple in its application.

The results of the analysis indicate that when the top layer is a thin weak clay underlain by a stiff clay, the failure mode is predominantly a general shear failure involving both clay layers. But when the top layer is a thin stiff clay underlain by a weak clay, the plastic yield pattern suggests a predominantly punching shear failure of the top clay layer.

For a thin sand layer underlain by a stiff clay, the yield patterns suggest a local shear failure limited to the sand layer. For a thin layer of stiff clay underlain by sand, the failure mode is predominantly a punching shear failure of the top clay layer.

The thickness of top soil layer has a marked influence on the ultimate bearing capacity. The bearing capacity decreases steadily with increasing top-layer thickness when the top layer is weaker than the bottom layer, and vice versa. The bearing capacity attains a steady value at a specific top-layer thickness, depending on the strength properties of the two soils.

The developed bearing-capacity equation is compared with available solutions. Results of the comparisons indicate that the developed equation can provide a more reasonable bearing-capacity value than existing ones.

ACKNOWLEDGMENT

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