Indian Geotechnical Journal, 20(3), 1990

Bearing Capacity of Footings on Two Layered Soil

by

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Introduction

MOST of the soils are stratified, and understanding the behaviour of foundation on such soils is important.

Meyerhof (1974) studied the bearing capacity of both circular and strip footings placed on a two layered soil media with sand layer overlying clay. Yamaguchi (1963) investigated the bearing capacity of a sandy layer of finite thickness resting on a soft clayey layer. Assuming a dispersion angle for pressure in sand layer below footing and taking uniform pressure at the top of clay layer, he has arrived at approximately the ultimate bearing capacity of the two layered system. Bowles (1988) suggests, for arriving at ultimate bearing capacity of layered soils, the use of modified value of ϕ based on thickness of upper layer below the footing and depth of soil wedge that will be formed below the footing. Reddy and Rao (1983), Reddy and Srinivasan (1967) have given procedures to assess the bearing capacity of stratified soil in which only cohesion is different in the layers. Purushothamaraj *et al.*, (1974) presented limit analysis approach for determining the ultimate bearing capacity of two layered soils. However, they presented results for two layered system with same value of angle of internal friction for both the layers and different values of cohesion for the layers. Straganov (1974) studied the bearing capacity of a surface rigid base footing in a two layered soil. The method of characteristics was used in the analysis. The bottom layer was assumed to be stronger than the top. Meyerhof (1974) presented a solution for a weak sand layer overlying a strong layer by assuming that the lower layer acted as a rigid base and employed the theory of Mandel and Salencon (1972) to determine the bearing capacity factors. Hanna (1982) investigated the ultimate bearing capacity of footings resting on subsoils consisting of weak sand layer overlying a strong layer of sand. Based on model tests of strip and circular footings in

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(The modified manuscript of paper was received in April 1990 and is open for discussion till end of April 1991).

a loose or compact sand layer overlying a dense sand deposit the classical equation for bearing capacity of footings on homogeneous sand was extended to cover cases of these footings in layered sands where the upper layer is weaker. It is observed that in all the cases the free surface of the ground has been assumed to be horizontal and in most of the above methods analysis is done by assuming a definite mode of failure of the soil below the base of the footing.

In this paper, the bearing capacity of strip footing placed in a localised depression is considered when the soil is stratified with bottom layer stronger than the top layer.

Statement of the Problem

Fig. 1 shows the schematic diagram of a strip footing of width $2B$, placed at the interface of a two layered c—ϕ soil. The top soil layer is assumed to be weaker than the bottom layer. The cohesion, angle of internal friction and unit weight of top soil layer are c_t, ϕ_t and γ_t. The corresponding values for the bottom layer are c_b, ϕ_b and r_b. The top layer extends upto a depth D_f, which is equal to depth of footing. Just adjacent to the footing the sloping surface of the ground starts at an angle β to the horizontal such that $0 \leqslant \beta \leqslant \phi_t$. The distance between the face of the footing and the starting point of the slope is Y_1.

The experimental work of De Beer and Vesic (1958) shows that the inclination of failure surface under the footing with horizontal is nearer to $45° + \phi/2$ rather than to ϕ assumed by Terzaghi (1943). In this paper, the inclination of the failure surface under the footing with the horizontal

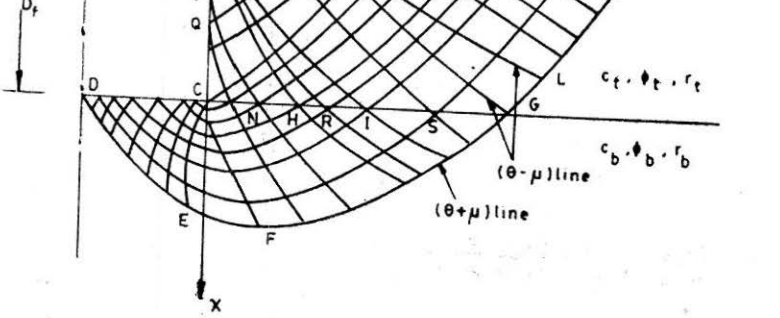

FIGURE 1 Definition Sketch.

is taken as $45^\circ + \phi/2$. According to Meyerhof (1951) the shaft resistance of a footing is to be taken into account in determining the ultimate bearing capacity of a footing. This is done in the present paper. The vertical sides of the footing are hence assumed to be rough. The total bearing capacity of the footing is the sum of base bearing capacity and the shaft resistance.

Analysis

The analysis has been done by the method of characteristics. The analysis satisfies the continuity of normal and shear stresses at the interface of the two layers. The bearing capacity factors are determined with respect to the cohesion and angle of internal friction of the bottom layer. The equations of equilibrium are given by:

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} = \gamma$$

$$\frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_y}{\partial y} = 0 \qquad (1)$$

where γ = body force in x direction per unit volume of the soil, σ_x, σ_y = normal stresses along x and y axis directions and τ_{xy} = shear stress. The three dependent variables σ_x, σ_y and τ_{xy} can be expressed in terms of two variables satisfying the Mohr-Coulomb failure condition as:

$$\sigma_x = \sigma\,(1+\sin\phi \;\; \cos 2\theta) - c\cot\phi$$

$$\sigma_y = \sigma\,(1-\text{ein}\phi \;\; \cos 2\theta) - c\cot\phi$$

$$\tau_{xy} = \sigma \sin\phi \, \sin 2\theta$$

where $\sigma = (\sigma_x + \sigma_y + 2c\cot\phi)/2$ and θ = angle between the direction of the major principal stress and x axis taken to be positive when measured in the counterclockwise direction.

Substituting the expressions for σ_x, σ_y and τ_{xy} in Eq. 1 and simplifying, the equations along the characteristics are obtained as (Sokolovsky, 1960):

$$\frac{dy}{dx} = \tan(\theta+\mu); \quad \frac{d\xi}{dx} = b$$

$$\frac{dy}{dx} = \tan(\theta-\mu)\,; \quad \frac{d\eta}{dx} = a \qquad (2)$$

where $$\frac{a}{b} = \pm \frac{\gamma \sin(\theta \pm \mu)}{2\sigma \sin\phi \cos(\theta \pm \mu)}$$

$$\mu = 45^\circ - \frac{\phi}{2}$$

$$\xi = \chi + \theta;\ \eta = \chi - \theta;\ \chi = \cot\frac{\phi}{2}\ln\frac{\sigma}{\sigma_o}\ ;\text{ and}$$

σ_o = arbitrary constant stress.

Boundary Conditions

Referring to Fig. 1, along OA and AJ the normal and shear stresses are zero. Hence, θ along $OA = 90°$ and along AJ, $\theta = 90 + \beta$.

Along OC, the shear stress τ_{nt} is given by

$\tau_{nt} = \sigma_n \tan\delta$ where σ_n is normal stress

on OC and δ is angle of wall friction.

Along the interface CG,

$$\sigma_{nt} = \sigma_{nb} \text{ and } \tau_{nt} = \tau_{nb}$$

where σ_{nt} and τ_{nt} are respectively normal and shear stresses at the interface in top layer respectively and σ_{nb} and τ_{nb} are the corresponding normal and shear stresses at the interface in bottom layer.

Starting from the horizontal surface OA and using the finite differences method, a network of characteristic lines is formed until the tip C of the footing is reached. In doing so, the computations at the singular point O and at the junction point A are made. First the point C is fixed by an iterative procedure. The iteration is made for a particular $(\theta + \mu)$ line, MC, which originates from a point M (Fig. 1) and ends up at point C with sufficient accuracy. Depending on the values of D_f, γ_1, β and ϕ_t, the location of point M changes. After the iteration, values of σ and θ at point C for the top layer viz., σ_t and θ_t are known.

In order to satisfy the continuity of stresses, the normal and shear stresses on any given plane should be same for each of the two layers. Thus, knowing the values of σ_t and θ_t at point C, the values of σ and θ for the bottom layer, viz., σ_b and θ_b could be found by equating the normal and shear stresses in x—y plane for each of the layers as explained below. If σ_{x_t} and τ_{xy_t} be the normal and shear stresses for top layer at that point. Then:

$$\tau_{xy_t} = \sigma_t \sin\phi_t \sin 2\theta_t \tag{3}$$

$$\sigma_{x_t} = \sigma_t (1+\sin\phi_t \cos 2\theta_t) - c_t \cot\theta_t \tag{4}$$

If σ_{x_b} and τ_{xy_b} be the normal and shear stresses at the same point for the bottom layer,

$$\tau_{xy_b} = \sigma_b \sin\phi_b \sin 2\theta_b \tag{5}$$

$$\sigma_{x_b} = \sigma_b (1+\sin\phi_b \cos 2\theta_b) - c_b \cot\phi_b \tag{6}$$

From continuity condition,

$$\sigma_{x_t} = \sigma_{x_b} \tag{7}$$

and,

$$\tau_{xy_t} = \tau_{xy_b} \tag{8}$$

Substituting Eqs. 3 and 5 into Eq. 8,

$$\sigma_b = \frac{\sigma_t \sin \phi_t \sin 2\theta_t}{\sin \phi_b \sin 2\theta_b} \tag{9}$$

Substituting Eqs. 4 and 6, in Eq. 7,

$$\sigma_t [1+\sin \phi_t \cos 2\theta_t)] - c_t \cot \phi_t$$
$$= \sigma_b [1+\sin \phi_b \cos 2\theta_b] - c_b \cot \phi_b \tag{10}$$

On further substitution of Eq. 9 in Eq. 10 and simplification, the following quadratic expression for θ_b is obtained,

$$M z^2 + N z + P = 0 \tag{11}$$

wherein,

$$z = \operatorname{cosec} 2\, \theta_b$$

$$M = \operatorname{cosec}^2 \phi_b - 1$$

$$N = -2\, N_1 \operatorname{cosec} \phi_b$$

$$N_1 = \frac{\sigma_t [1+\sin \phi_t \cos 2\, \theta_t] - c_t \cot \phi_t + c_b \cot \phi_b}{\sigma_t \sin\phi_t \sin 2\theta_t}$$

and $P = (N_1^2 + 1)$

Knowing the coefficients M, N and P of Eq. 11, the quadratic equation is solved to get a pair of roots. The corresponding values of θ_b are also found using,

$$\theta_b = \frac{1}{2} \sin^{-1}\left(\frac{1}{z}\right) \tag{12}$$

Of the two values of θ_b one is for active case and the other for passive case. The appropriate root is chosen. The values of χ, ξ and η are found using,

$$\chi = \frac{\cot\phi_b}{2} \ln\left(\frac{\sigma_b}{\sigma_o}\right) \tag{13}$$

$$\xi = \chi + \theta_b \tag{14}$$

$$\eta = \chi - \theta_b \tag{15}$$

At point C in bottom layer, the change in the value of θ will be from θ_b to zero, as we move from right to left of point C. Zone CEF in Fig. 1 corresponds to the radial shear zone.

The soluti n for other $(\theta + \mu)$ lines which originate from the free boundary OAJ involves considerable iterative procedures. This is because the $(\theta + \mu)$ line, originating from the free surface should meet any other $(\theta - \mu)$ line, at the interface, in order to extend the solution for the bottom layer. The iterative procedure which has been developed could be best explained by dividing the solution domain of the top layer into different zones. Referring to Fig. 1, *OCH, OHI, OAGI, AGL* and *ALJ* are the different zones which are considered. Of the above, zone *OCH* corresponds to the band of $(\theta - \mu)$ lines originating from *OC* and zone *OHI* is the radial shear zone. Zone *OAGI* is the band of $(\theta - \mu)$ lines originating from horizontal boundary *OA*. Zone *AGL* corresponds to the band of $(\theta - \mu)$ lines originating from point A as shown in Fig. 1. Consider a point N which lies in the zone *OCH*. This point is fixed by the iterative procedure for a $(\theta + \mu)$ line such that the intersection point is exactly at the interface. The $(\theta + \mu)$ line which meets at point N could start from either horizontal surface or from the inclined surface, for which corresponding iterations are made by changing the y—coordinate of starting point. If the starting point is at point A, then the iteration is made by changing the θ value at point A. Depending on the value of Y_1 and D_f the end points P and Q of $(\theta + \mu)$ lines starting from A can meet the interface also. Thus, the state of stress for the top layer at points like N, I, G, R and S etc., are obtained. Knowing the values of σ_t and θ_t, values of σ_b and θ_b are obtained, using the continuity conditions at the interface.

Computations are continued using the earlier procedure until the base of the footing is reached. In using the equations along the characteristic lines in bottom layer, the values of ϕ, μ, c and γ correspond to the values of bottom layer. Knowing the value of σ at the end point of $(\theta + \mu)$ lines which are at the base of the footing, the normal stresses are obtained. Similar procedure is used until the centre line of the footing is reached. The algebraic sum of the total normal force over the width $2B$. viz., P_{PV} represents the total passive force at the base of the footing.

As the footing is resting at the top of the bottom layer, the average normal pressure (q) at the base of the footing could be expressed as:

$$q = c_b\, N_c + \gamma_b\, B\, N_q \tag{16}$$

The values of N_c and $N_{\gamma q}$ are computed using,

$$N_c = \frac{P_{PC}}{2\,B\,c_b} \tag{17}$$

$$N_{\gamma q} = \frac{P_{P\gamma} - P_{PC}}{2\, B^2\, \gamma_b} \tag{18}$$

As the sides of the footing are rough, shaft resistance also develops, in addition to the base bearing capacity. The shaft resistance developed on the footing for a given geometry and for a given type of soil is constant and is equal to T_v.

Non-dimensionalisation

Stresses are non-dimensionalised by dividing them by a characteristic stress (c) and distances are non-dimensionalised by dividing them by a characteristic length ($l = c/\gamma$). The following non-dimensional quantities have been used.

$$c'_b = c_b/c;\ c'_t = c_t/c$$

$$\sigma'_{x_b} = \sigma_{x_b}/c \text{ and } \sigma'_{x_t} = \sigma_{x_t}/c$$

By doing so, the Eq. 16 is written as,

$$q' = c'_b\, N_c + \frac{\gamma_b\, l}{c}\, B'\, N_{\gamma q} \tag{19}$$

Results and Discussion

First, N_c is obtained by assuming the soil in both the layers to be weightless and finding the non-dimensional passive force P'_{PC}.

Next the non-dimensional force $P'_{P\gamma}$ below the footing is found by considering the weight of the soil taking $\gamma_b l/c = 1.0$. The value of $N_{\gamma q}$ is computed using Eq. 18, after expressing it in non-dimensional form.

Numerical results have been presented for $\phi_t = 20°$ with $\phi_b = 1.5\ \phi_t$ and $2\ \phi_t$ and for $\phi_t = 30°$, with $\phi_b = 1.33\ \phi_t$ and $B' = 1.0$. In each case values of β are assumed to be $1/2\ \phi_t$ and ϕ_t, with $D_f' = 0.50$ and 1.0. Results have been presented in the form of graphs for different values of Y_1' by assuming $\delta = 0.5\ \phi_t$ along the vertical sides of the footing. These results have been given for $c_b' = c_t' = 1.0$ and $\gamma_b/\gamma_t = 1.0$. However, the same analysis could be used when $c_b' \neq c_t'$ and $\gamma_b \neq \gamma_t$.

Fig. 2 shows the variations of σ' and θ along the interface of the two layers for the above case. Referring to Fig. 2, it is observed that the values of σ_b' are higher than σ_t' upto some distance, after which the σ_b' is found to be less than σ_t'. These variations in the values of σ_b' and σ_t' with Y' mainly depend on the zone in which the point in question lies.

Figures 3 through 7 show the variations of N_c and $N_{\gamma q}$ with the said parameters. The influence of each parameter is studied by keeping the other

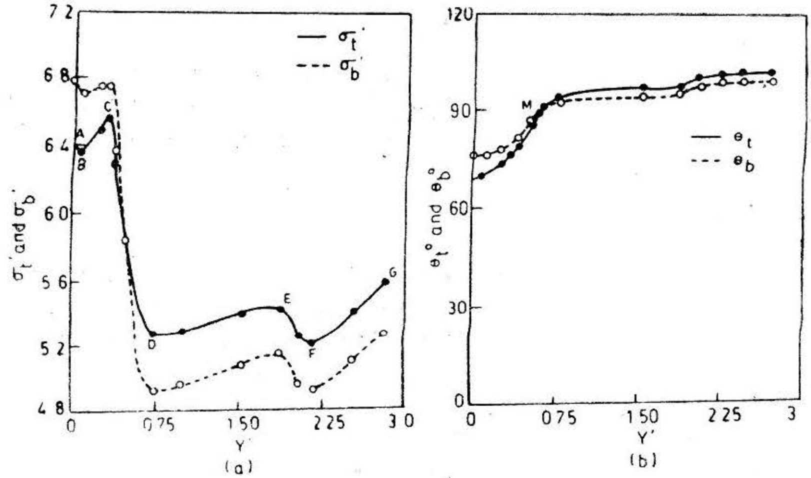

FIGURE 2 Values of σ' and θ at the interface for $\phi_t = 20^\circ$, $\phi_b = 30^\circ$, $\delta = \beta = 10^\circ$, $Y_1' = 1.0$ and $D_f' = 0.5$ with $\gamma_t l/c = \gamma_b l/c = 1.0$ and $c_t' = c_b' = 1.0$

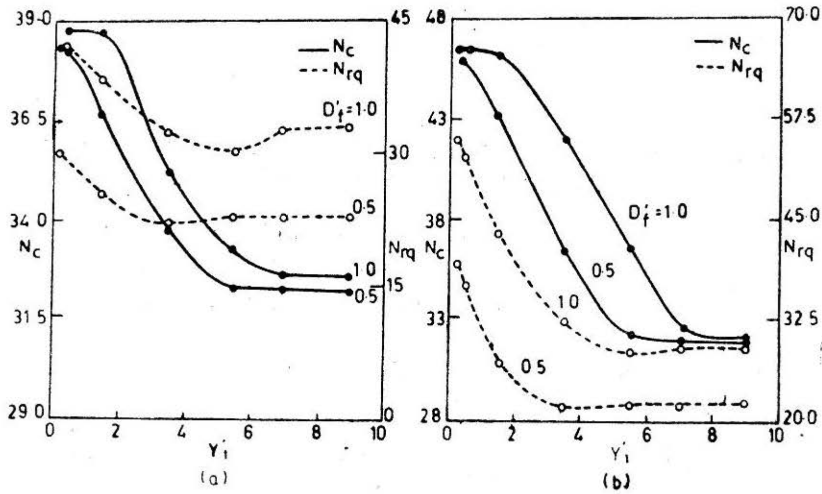

FIGURE 3 Values of N_c and $N_{\gamma q}$ for $\phi_t = 20^\circ$, $\phi_b = 1.5\,\phi_t$, $\delta = 10^\circ$ and (a) $\beta = 10^\circ$ and (b) $\beta = 20^\circ$

parameters constant and changing the particular parameter between the two extreme values.

(a) Influence of Y_1'

It is seen from Figs 3 through 7 that, for a given set of other parameters, the values of N_c and $N_{\gamma q}$ decrease with increase in Y_1' and attain constant values thereafter. When Y_1' is changed from 0.25 to 10.0, the decrease in the value of N_c when $\phi_t = 30^\circ$, $\phi_b = 40^\circ$, $\beta = 30^\circ$, and $D_f' = 0.50$ is 91

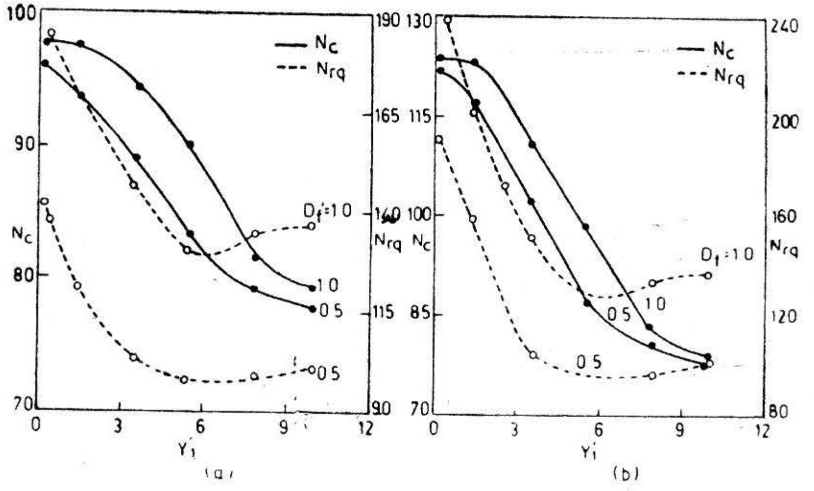

FIGURE 4 Values of N_c and $N_{\gamma q}$ for $\phi_t = 20°$, $\phi_b = 2\phi_t$, $\delta = 10°$ and (a) $\beta = 10°$ and (b) $\beta = 20°$

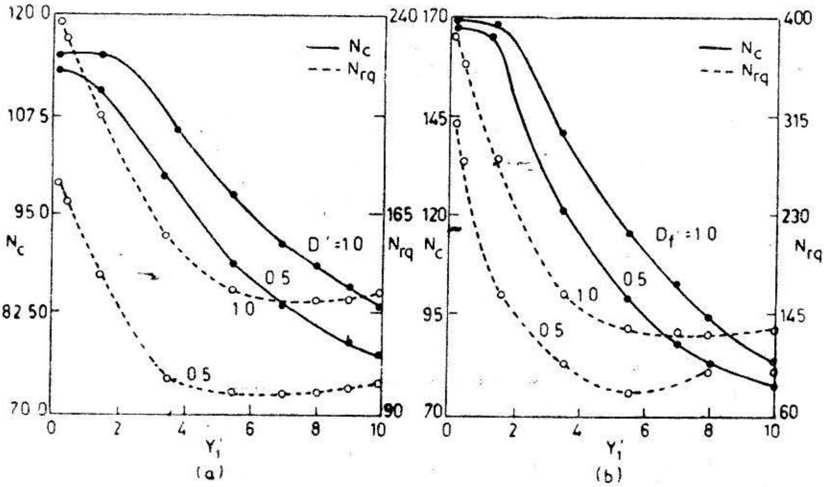

FIGURE 5 Values of N_c and $N_{\gamma}q$ for $\phi_t = 30°$, $\phi_b = 40°$, $\delta = 15°$ and (a) $\beta = 15°$ and (b) $\beta = 30°$

percent. The corresponding increase in the value of $N_{\gamma q}$ for $\phi_t = 20°$, $\phi_b = 30°$, $\beta = 20°$ and $D_f' = 0.50$ is equal to 88 percent.

(b) *Influence of D_f'*

It is observed from Figs. 3 through 7 that the values of N_c and $N_{\gamma q}$ are higher for higher values of D_f'. At very low values of Y_1' an increase in the value of D_f' can cause appreciable increase in the values of $N_{\gamma q}$ while a little change in N_c. The increase in $N_{\gamma q}$ value at $Y_1' = 0.50$ when D_f' is changed from 0.50 to 1.0 for $\phi_t = 20°$, $\phi_b = 30°$, $\beta = 20°$ is 50 percent.

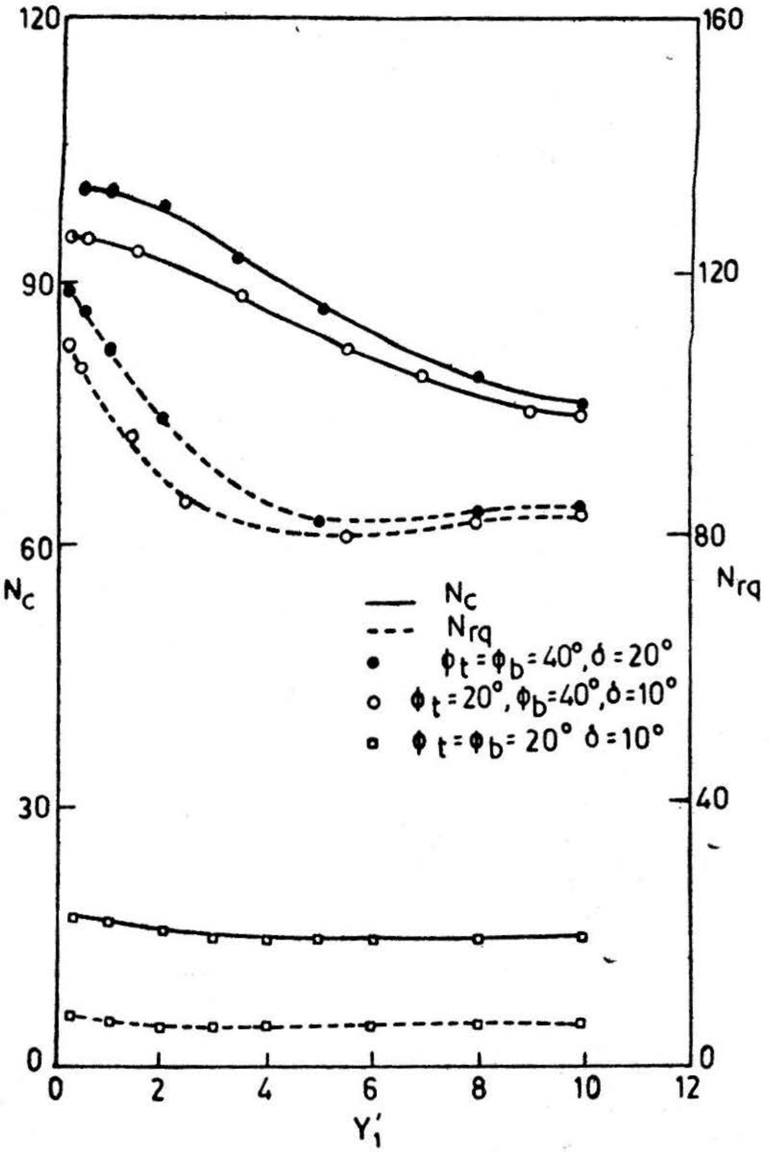

FIGURE 6 Comparison of N_c and $N_{\gamma q}$ for $\phi_t = 20°$, $\phi_b = 40°$, $\delta = 10°$, $D_f' = 0.5$ and $\beta = 10°$ with those of a uniform soil

(*c*) *Influence of* β

Referring to Figs. 3(*a*) and 3(*b*) it observed that the values of N_c and $N_{\gamma q}$ at low values of Y_1' are higher for higher values of β. For $Y_1' = 0.5$ percentage increases of 47 and 62.5 in the values of N_c and $N_{\gamma q}$ are observed for $\phi_t = 30°$, $\phi_b = 40°$ when β is changed from 0.5 ϕ_t to ϕ_t.

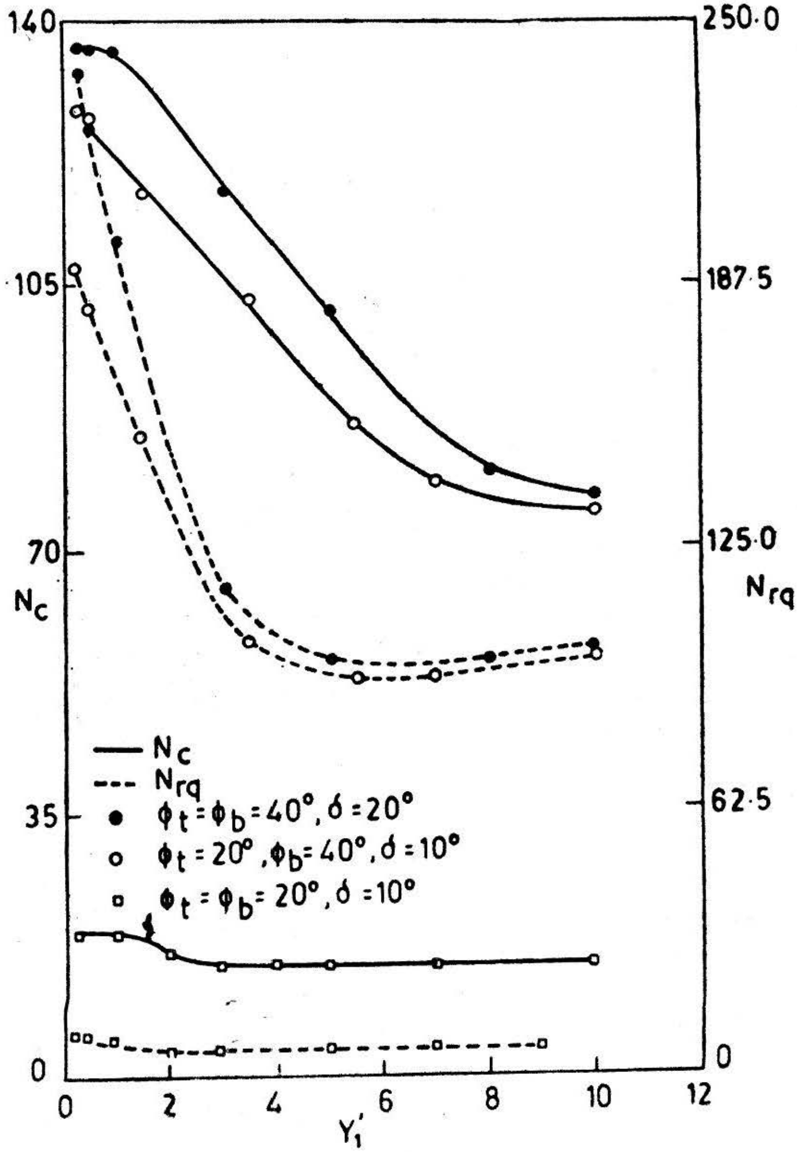

FIGURE 7 Comparison of N_c and $N_{\gamma q}$ for $\phi_t = 20°$, $\phi_b = 40°$, $\delta = 10°$, $D_f' = 0.5$ and $\beta = 20°$ with those of a uniform soil.

(d) Influence of ϕ_b

It is also observed from Figs 3 through 7 that, for a set of values of D_f', β, ϕ_t and Y_1', an increase in the value of ϕ_b will result in an increase in the values of N_c and $N_{\gamma q}$. For $\phi_t = 20°$, $\phi_b = 30°$ the increases in the values of N_c and $N_{\gamma q}$ are 133 and 457 percent, respectively when ϕ_b is changed from 1.0 to 1.50 ϕ_t for $D_f' = 1.0$, $\beta = 20°$ and $Y_1' = 0.50$.

Figure 6 gives the comparison of the values of N_c and $N_{\gamma q}$ of a layered soil with those of a uniform soil for $D_f' = 0.50$, $\phi_t = 20°$ and $\beta = 10°$. It is seen from the figure that the values of N_c and $N_{\gamma q}$ are higher for a uniform soil with ϕ equal to ϕ_b of the layered soil. This trend is obvious, because of the presence of a weaker soil upto the base of the footing. The plotted values of N_c and $N_{\gamma q}$ for a uniform soil have been obtained by taking $\phi_t = 40°$, $\phi_b = 40.0005°$, $D_f' = 0.50$, $\delta = 20°$ and $\beta = 10°$. Also, it is observed that the values of N_c and $N_{\gamma q}$ for a layered soil are higher than those for a uniform soil with ϕ of the uniform soil is equal to ϕ_t of the layered soil. Similar trend is observed when $\beta = 20°$ (Fig. 7).

(e) Influence of γ_b/γ_t

In all the results presented above, the ratio γ_b/γ_t is assumed to be 1.0. However, in practice, the density of the stronger layer will be more than that of the weaker layer. Hence, the ratio γ_b/γ_t will be greater than 1.0. A ratio of 1.05 seems not unrealistic. Hence, typical results to study the influence of γ_b/γ_t are given in Table 1. It is observed from the Table that the values of N_c remain constant irrespective of the value of γ_b/γ_t as they are obtained for $\gamma = 0$ conditions. However, a decrease in the value of $N_{q\gamma}$ is observed when the ratio is increased from 1.0 to 1.05. A maximum of 2.9 percent decrease in the value of $N_{\gamma q}$ is observed which is small as compared with the influence of other parameters. Hence, it can be said that γ_t/γ_t has little influence on $N_{\gamma q}$.

TABLE 1

Influence of γ_b/γ_t for $\phi_t = 20°$, $\phi_b = 30°$, $\delta = 10°$ $\beta = 10°$ and $D_f' = 0.50$

Y_1'	$\gamma_b/\gamma_t = 1.0$		$\gamma_b/\gamma_t = 1.5$		Percentage Decreases
	N_c	$N_{\gamma q}$	N_c	$N_{\gamma q}$	$N_{\gamma q}$
0.50	36.69	29.66	36.69	28.80	2.9
1.50	36.24	25.50	35.24	24.82	2.6
3.50	32.51	21.88	32.51	21.38	2.3
5.50	30.53	22.83	30.53	22.32	2.2

Conclusion

Based on the results and discussion given above the following conclusions are drawn.

Numerical results given show that the values of N_c and $N_{\gamma q}$ decrease with increase in Y_1'

The values of N_c and $N_{\gamma q}$ are higher for higher slope angles. Increase in the value of D_f' does not appreciably increase the value of N_c, while, the increase in the value of $N_{\gamma q}$ is considerable.

A comparison of N_c and $N_{\gamma q}$ for a footing on uniform soil extending to infinity with those for a layered soil shows that for the ranges of parameters considered, the values of N_c and $N_{\gamma q}$ are higher for a uniform layer with value of angle of internal function equal to ϕ_b of a layered soil Also, it is observed that, the values of N_c and $N_{\gamma q}$ for a uniform soil with value of ϕ equal to ϕ_t of a layered soil are lower than those for a layered soil.

A change in the value of ϕ_b from ϕ_t to 1.5 ϕ_t can increase N_c and $N_{\gamma q}$ values by 133 percent and 457 percent, respectively.

Thus, it can be concluded that the presence of an upward slope just adjacent to the footing and presence of a stronger layer below the base of the footing will increase the bearing capacity considerably.

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