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Bearing Capacity of Eccentrically Loaded Footings adjacent to Cohesionless Slopes

by

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Introduction

A special problem that may be encountered occasionally is that of a footing located adjacent to a slope (Fig. 1). It can be seen frcm the figure that the lack of soil on the slope side of the footing will tend to reduce the stability of the footing. In such a situation, the problem beccmes that of obtaining the minimum value of the bearing capacity: (*i*) frcm foundaticn failure and (*ii*) from overall stability of the slope.

Problem of obtaining ultimate bearing capacity of a footing adjacent to a slope considering foundation failure has been solved by using three different approaches namely: (*i*) slip line analysis (Sokolovski, 1960; Siva Reddy and Mogliah, 1975), (*ii*) Limit equilibrium analysis (Meyerhof, 1957; Mizuno *et al.* (1960); Siva Reddy and Mogliah (1976); Bowles (1984) Myslivec and Kysela (1978) and (*iii*) Limit analysis (Chen, 1975). Critical evaluation of these methods has been given by Sud, 1984.

Eccentrically loaded footings on flat ground have been analysed by many investigators (Meyerhof, 1953; Hansen, 1956; Saran, 1969). Critical review of these methods is presented by Saran, 1969.

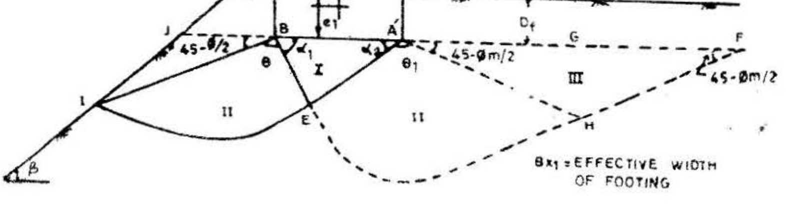

FIGURE 1 Rupture Surface

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No method is available so far giving the bearing capacity of eccentrically loaded footings adjacent to cohesionless slopes (Reddy, 1986).

In this paper, an analytical solution has been presented to obtain the bearing capacity of eccentrically loaded footings adjacent to cohesionless slopes using limit equilibrium approach. The overall slope stability should be checked for the effect of footing load.

Theory

Assumptions

The following assumptions have been made in the analysis:

1. Footing is a shallow strip footing having rough base and the weight of the soil above the base of the foundation is replaced by an equivalent uniform surcharge.

2. One sided failure is assumed to occur along surface, A'EI (Fig. 1). The failure region is divided into three zones. Zone-I represents an elastic region, Zone-II is a radial shear zone and curved portion EI of this zone is a log spiral having its centre on the edge of footing *i.e.* B or its extension (Saran 1970). Zone-III is a passive zone indicating that the soil in this zone is in passive state.

3. Shear strength of soil on the side of flat ground (right of point A, Fig. 1) is taken as partially mobilized and it is characterised by a mobilization factor m which is less than unity. Shear resistance of soil is then expressed as

$$\tau = m\,(c + \sigma \tan \phi) \tag{1}$$

   To compute the partial resistance offered by this side, a rupture surface as shown by dotted lines is considered. The curved portion EH is a logarithmic spiral having its center at A' and Zone $A'\,HF$ is a passive Rankine Zone (Saran, 1969).

4. Footing loses contact with the soil in a characteristic manner as eccentricity of load increases.

5. Principle of superposition holds good.

Analytical solutions are developed for a general case where the footing has lost some contact with soil. The contact width of footing is assumed to be $B.x_1$. For full footing contact x_1 = unity. Solutions are developed as given below:

Geometry of the Failure Surface

In Fig. 1, considering triangle $BA'E$,

$$BE = \frac{B.\ X_1.\ \mathrm{Sin}\ \alpha_2}{\mathrm{Sin}\ (\alpha_1 + \alpha_2)} = r_o \tag{2}$$

$$A'E = \frac{B.\ X_1\ .\ \mathrm{Sin}\ \alpha_1}{\mathrm{Sin}\ (\alpha_1 + \alpha_2)} \tag{3}$$

$$BJ = De + D_f/\tan \beta \tag{4}$$

From the triangle BIJ

$$BI = \frac{BJ\ \mathrm{Sin}\ (180° - \beta)}{\mathrm{Sin}\ (\beta + \theta + \alpha_1 - 180°)} \tag{5}$$

From the log spiral, $BI = BE.\ e^{\theta \tan \phi} = r_1$ (6)

From equations 5 and 6

$$BE.\ e^{\theta \tan \phi} = \frac{BJ.\ \mathrm{Sin}\ (180° - \beta)}{\mathrm{Sin}\ (\beta + \theta + \alpha_1 - 180°)} \tag{7}$$

Putting the values of BE and BJ from equations (3) and (4) in Eq. 7, we get,

$$\frac{B.\ x_1\ \mathrm{Sin}\ \alpha_2}{\mathrm{Sin}\ (\alpha_1 + \alpha_2)}\ e^{\theta \tan \phi} = \frac{\left(De + \dfrac{D_f}{\tan \beta} \right) \mathrm{Sin}\ \beta}{\mathrm{Sin}\ (\beta + \theta + \alpha_1 - 180°)}$$

or
$$\frac{x_1\ \mathrm{Sin}\ \alpha_2}{\mathrm{Sin}\ (\alpha_1 + \alpha_2)}\ .\ e^{\theta \tan \phi} = \frac{\left(\dfrac{De}{B}\ \mathrm{Sin}\ \beta + \dfrac{D_f}{B}\ .\ \mathrm{Cos}\ \beta \right)}{\mathrm{Sin}\ (\beta + \theta + \alpha_1 - 180°)} \tag{8}$$

Bearing Capacity Expressions

The bearing capacity expression is then developed by considering the equilibrium of elastic wedge $A'EB$. The forces acting on the wedge are : (*i*) Passive earth pressure p_p on side BE (*ii*) Earth pressure p_m at partial mobilization factor m on side $A'E$ and (*iii*) Eccentric load Qd (Fig. 2).

Neglecting the weight of soil wedge $A'BE$, footing equilibrium requires that:

$$Qd = p_p\ \mathrm{Cos}\ (\alpha_1 - \phi) + p_m\ \mathrm{Cos}\ (\alpha_2 - \phi_m) \tag{9}$$

Passive earth pressure p_p can be divided into two parts p_{pr} and p_{pq}. Force p_{pr} represents the resistance due to weight of soil mass $A'EIJB$. The point of application of p_{pr} is located at lower third point of BE. Force p_{pq} represents the resistance due to surcharge only. As pressure p_{pq} is uniformly distributed, its point of application is located at the mid point of

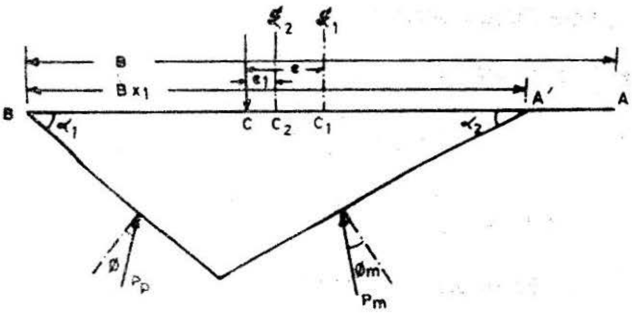

FIGURE 2 Forces on Elastic Wedge A′BE

contact face *BE*. Similarly, earth pressure, p_m, at partial mobilization factor *m* can be divided into two parts p_{mr} and p_{mq}. Thus

$$Qd = (p_{pr}+p_{pq}) \text{ Cos } (\alpha_1-\phi)+(p_{mr}+p_{mq}) \text{ Cos } (\alpha_2-\phi_m) \tag{10}$$

Average surcharge intensity q can be expressed as:

On sloping side

$$q = \frac{\gamma D_e.D_f \tan\beta + \frac{1}{2}\gamma D_f^2}{D_e \tan\beta + D_f} = \frac{\gamma D_f\left[\frac{D_e}{D_f}\tan\beta + \frac{1}{2}\right]}{\frac{D_e}{D_f}\tan\beta + 1} \tag{11}$$

On flat ground side

$$q' = \gamma D_f \tag{12}$$

By introducing symbols

$$N_\gamma = \frac{2 p_{pr} + 2 p_{mr}}{\gamma B^2} \tag{13}$$

and

$$N_q = \frac{p_{pq} + p_{mq}}{\gamma . D_f . B} \tag{14}$$

Substituting in equation (3)

$$Qd = B\left[\tfrac{1}{2}\gamma B N_\gamma + \gamma D_f Nq\right] \tag{15}$$

The quantities N_γ and Nq are termed as bearing capacity factors. These are dimensionless quantities that depend on ϕ, β, $\frac{De}{B}$ and $\frac{D_f}{B}$.

Computations of Passive Pressures P_{pr} and P_{pq}

For the determination of passive earth pressures P_{pr} and P_{pq}, the equilibrium of soil mass BEIJ needs only to be considered (Fig. 3). The forces acting on this wedge are :

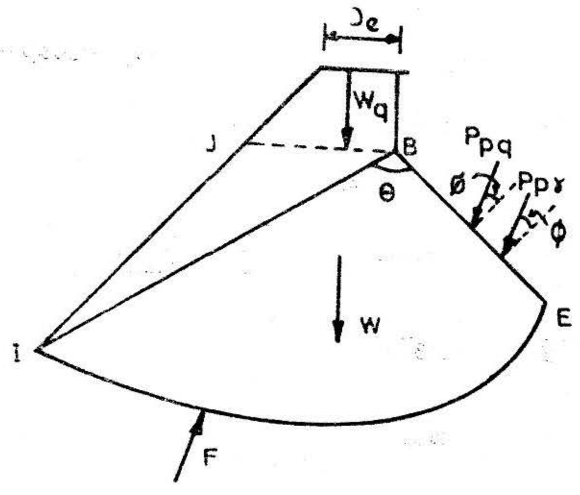

FIGURE 3 Forces on Soil 'BEIJ'

1. Weight, W, of soil mass which acts vertically downward at the centre of gravity of soil mass BEIJ.

2. Surcharge weight W_q acting on BJ. This surcharge is assumed as uniformly distributed on the length BJ.

3. Passive earth pressure, P_{pr}, acting on face BE. It acts at lower third point of BE and makes an angle ϕ anticlockwise with the normal at that point.

4. Passive earth pressure, Ppq, acting on face BE. It acts at mid point of BE at an angle ϕ, anticlockwise with the normal at that point.

5. Resultant, F, of the normal and frictional forces. It will pass through the centre of log-spiral since it makes an angle ϕ with the normal at the point of application.

Passive earth pressures P_{pr} and P_{pq} are determined by taking the moments of all the forces about the centre of log-spiral (i.e. edge of footing B), and therefore, the moment of force F gets eliminated. The equation obtained thus is :

$$P_{pr}.\ B.\ T_1 + P_{pq}.\ B.T_2 = N_{pr}.\ \gamma\ B^3 + N_{pq}.\ q.B^2 \tag{16}$$

where

$$T_1 = x_1 \operatorname{Cos} \phi \left[\frac{2}{3}\ \frac{\operatorname{Sin} \alpha_2}{\operatorname{Sin} (\alpha_1+\alpha_2)} \right] \tag{17}$$

$$T_2 = \tfrac{3}{4}.\ T_1 \tag{18}$$

$$N_{pr} = \frac{r_o'^3}{3(1+9\tan^2\phi)} [e^{3\theta\tan\phi}\{3\tan\phi . \mathrm{Sin}(\theta-90°+\alpha_1) - \mathrm{Cos}(\theta-90°+\alpha_1)\}$$

$$+ \mathrm{Cos}(90-\alpha_1) + 3\tan\phi \ \mathrm{Sin}(90-\alpha_1)]$$

$$+ \tfrac{1}{3} r_1'^3 \mathrm{Cos}^2(180-\theta-\alpha_1). \mathrm{Sin}(180-\theta-\alpha_1)$$

$$- \frac{1}{4}\left[r_1' \mathrm{Cos}^2(180-\phi-\alpha) - \left(\frac{De}{B} + \frac{D_f}{B\tan\beta}\right)\right] r_1' \mathrm{Sin}(180-\theta-\alpha).$$

$$\left[\frac{De}{B} + \frac{D_f}{B\tan\beta}\right] \tag{19}$$

$$N_{pq} = \tfrac{1}{2}\left(\frac{De}{B} + \frac{D_f}{B\tan\beta}\right)^2 \tag{20}$$

$$r_o' = \frac{r_o}{B}$$

$$r_1' = \frac{r_1}{B}$$

The equation (16) is solved considering two independent cases :

1. Soil having weight only (q = 0)

i.e.

$$P_{pr}. B. T_1 = N_{pr}. \gamma B^3$$

or $$P_{pr} = \frac{N_{pr}}{T_1} . \gamma B^2, \tag{21}$$

2. Weightless soil having surcharge only (i.e. $\gamma = 0$)

$$P_{pq}. B. T_2 = N_{pq}. qB^2$$

or $$P_{pq} = \frac{N_{pq}}{T_2} . qB \tag{22}$$

Computation of Passive Pressures P_{mr} and P_{mq}

For the determination of passive pressures P_{mr} and P_{mq} equilibrium of soil mass A'EHG is considered. The weight of soil mass HGF and surcharge on HG are taken equivalent to lateral earth pressure on HG. (Fig. 4).

By proceeding exactly in the same manner as discussed above, values of P_{mr} and P_{mq} are given by :

$$P_{mr} = \frac{N_{mr}}{T_1'} . \gamma B^2 \tag{23}$$

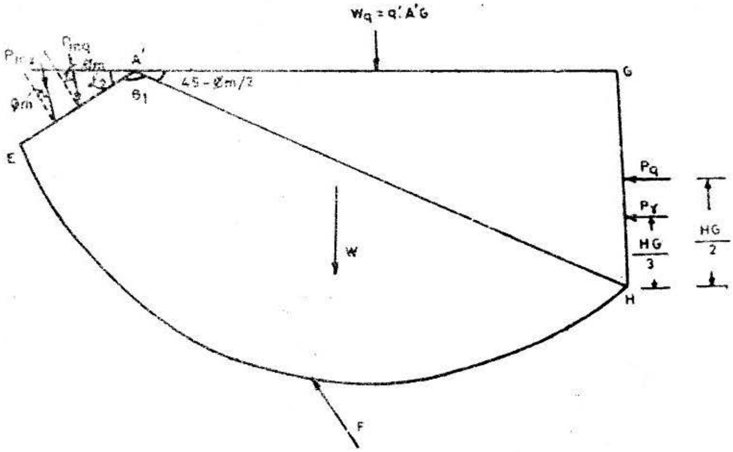

FIGURE 4 Forces on Mass Aejg

$$P_{mq} = \frac{N_{mq}}{T_2'} \cdot q'B \tag{24}$$

where $$T_1' = \tfrac{2}{3} \cdot x_1 \cdot \text{Cos}\phi_m \cdot \frac{\text{Sin}\ \alpha_1}{\text{Sin}\ (\alpha_1+\alpha_2)} \tag{25}$$

$$T_2' = \tfrac{3}{4} \cdot T_1' \tag{26}$$

$$N_{m_r} = \tfrac{1}{2} \cdot r_1''^3 \tan^2\left(45 + \frac{\phi_m}{2}\right) \cdot \text{Sin}^3\left(45 - \frac{\phi_m}{2}\right)$$

$$+ \frac{r_o''^3}{3(1+9\tan^2\phi_m)} \left[e^{3\theta' \tan\phi_m}\{3 \tan\phi_m\ \text{Sin}\ (\theta'-90+\alpha_2)\right.$$

$$\left. -\ \text{Cos}(\theta'-90+\alpha_2)\} + \text{Cos}(90-\alpha_2) + 3\tan\phi_m \cdot \text{Sin}\ (90-\alpha_2)\right] \tag{27}$$

$$N_{mq} = r_o''\ e^{\theta' \tan\phi_m} . \text{Cos}^2\left(45 - \frac{\phi_m}{2}\right) \tag{28}$$

where $$r_o'' = \frac{x_1\ \text{Sin}\ \alpha_1}{\text{Sin}\ (\alpha_1+\alpha_2)} \tag{29}$$

$$r_1'' = r_o''\ e^{\theta' \tan\phi_m} \tag{30}$$

It may be noted that in the above equations (22) and (24) values of q and q' are taken respectively from equations (11) and (12) respectively.

Relationship Between Wedge Angles

The relationships between wedge angles α_1 and α_2 are then obtained by solving three equilibrium equations obtained by the statics of elastic wedge A'BE (Fig. 2). The equation so obtained are :

For weight only (i.e. q = 0)

$$\frac{2 \text{ Sin } (\alpha_1-\phi)}{3 \text{ Sin } (\alpha_2-\phi_m)} \cdot \text{Cos } \phi_m + \frac{\text{Cos}\phi \text{ . Sin } \alpha_2}{3 \text{ Sin } \alpha_1} + \text{Cos}(\alpha_1+\alpha_2-\phi)$$

$$= \frac{\text{Sin}(\alpha_1+\alpha_2-\phi-\phi_m)}{\text{Sin}(\alpha_2-\phi_m)} \cdot \frac{\text{Sin}(\alpha_1\alpha_2)}{\text{Sin } \alpha_1}\left(1+\frac{e}{Bx_1}-\frac{1}{2x_1}\right) \quad (31)$$

For surcharge only (i.e. $\gamma = 0$)

$$\frac{1}{2} \frac{\text{Sin}(\alpha_1-\phi)}{\text{Sin}(\alpha_2-\phi_m)} \cdot \text{Cos } \phi_m + \frac{\text{Cos}\phi\text{. Sin}\alpha_2}{2 \text{ Sin}\alpha_1} + \text{Cos}(\alpha_1+\alpha_2-\phi)$$

$$= \frac{\text{Sin}(\alpha_1+\alpha_2-\phi-\phi_m)}{\text{Sin } (\alpha_2-\phi_m)} \cdot \frac{\text{Sin}(\alpha_1+\alpha_2)}{\text{Sin } \alpha_1}\left(1+\frac{e}{Bx_1}-\frac{1}{2x_1}\right) \quad (32)$$

Ultimate bearing Capacity

The ultimate bearing capacity is then evaluated for the case when all the three conditions of equilibrium are satisfied and when mattains the maximum value.

Computation

The range and interval of variables employed in computing bearing capacity factors are given in Table 1.

TABLE 1

Range and Interval of Parameters Used in Computations

Parameter	Range	Interval
ϕ	0° to 40°	5°
e/B	0 to 0.3	0.1
β	0 to 30°	10°
De/B	0 to 5	1.0
D_f/B	0 to 1	0.5

Figure 5 shows the variation of contact width factor X_1 for three different types of contact width variation, i.e. (*i*) triangular variation, (*ii*) conventional variation, and (*iii*) full contact width variation. Computations of $N\gamma$ and Nq factors were done assuming variation of X_1 from either of the three. It will be discussed later that there is no effect of contact width variation on $N\gamma$ and Nq values.

The following steps are performed to find the values of N_γ for given

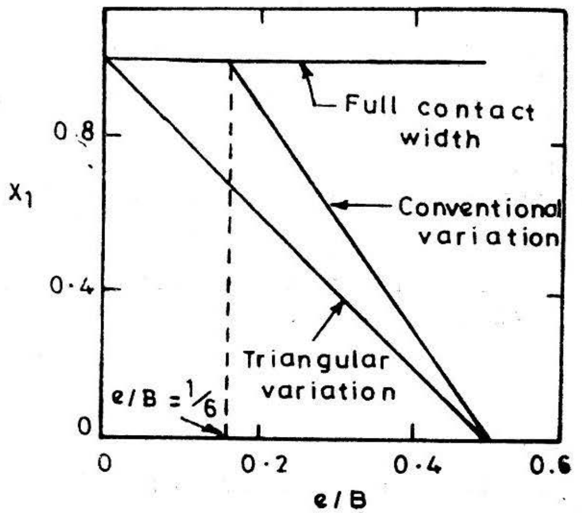

FIGURE 5 Variations of *X* Width *e/B* for different Types of Contact Width Variation

values of angle of internal friction ϕ, slope angle β, depth factor D_f/B and edge distance factor D_e/B.

1. X_1 is taken from the assumed contact width variation (Fig. 5).

2. A particular value of mobilization factor 'm' is assumed. ϕ_m is computed as :

$$\phi_m = \tan^{-1}(m \tan \phi) \qquad (33)$$

3. A particular value of α_1 is assumed. For assumed value of α_1; α_2 is computed using wedge angle relationship given by Eq. (31). Value of θ is then obtained by trial and error method using Eq. (8). Value of θ_1 is taken simply as given below:

$$\theta_1 = 180° - (45 - \phi_m/2) - \alpha_2 \qquad (34)$$

4. For a set of wedge angles (α_1 and α_2), the values of the passive earth pressures ($P_{pr}/\gamma B^2$ and $P_{pmr}/\gamma B^2$) are determined using Eqs. (21) and (22).

5. The above computed values of the passive earth pressures will satisfy the two conditions $\Sigma V = 0$ and $\Sigma M = 0$ simultaneously as the former is used for determining the bearing capacity Q_{dr} and the later is used in obtaining the wedge angle relationship. The only equilibrium equation that remains to be satisfied is $\Sigma H = 0$. If this is satisfied, the values of α_1 and α_2 adopted in the computations are in order. Otherwise steps 3 to 5 are repeated for other values of α_1 till all equilibrium conditions are satisfied.

6. Steps 2 to 5 are repeated for different values of the mobilization factor m. The passive pressures for maximum value of 'm' satisfying the equilibrium conditions ($\Sigma H = 0$, $\Sigma V = 0$ and $\Sigma M = 0$) are adopted.

The maximum value of m is chosen because for failure the soil must develop maximum possible resistance compatible with stability. The corresponding bearing capacity factors are smallest in this case.

Proceeding in exactly similar way as outlined above, the bearing capacity factor N_q is evaluated.

Interpretation

Evaluation of Assumptions

Of the five assumptions made in the development of the analysis, two are discussed herein viz., the assumptions listed at serial no. 2 and 3. The other assumptions are commonly used in bearing capacity computations by limit equilibrium analysis.

As the soil available is less on the side of the slope, the resistance offered from this side of the footing will be lesser than that from the other side. Due to this fact, it seems reasonable to assume that one sided failure occurs. Limited data is available for footings on slopes. However, observations made in model tests performed by Peynircoiglu (1948) and Mizuno et al. (1960) have clearly shown that the failure occurs only on the side of the slope. Some pressures do develop on the other side as well. At equilibrium the resistance developed on the other side will not reach the full mobilization value. Hence, pressure on this side has been considered at partial mobilization of strength for computation of the bearing capacity.

According to the assumption 3, the centre of the log spiral has been taken on AE or its extension (Fig. 2.1), while in Terzaghi's analysis, the centre was considered on IA or its extension. As discussed by Saran (1970), for footing on a level ground, if the wedge angles are equal to ϕ, then the log-spiral will be tangential to the vertical only when the centre of the log spiral lies on the line AE or its extension. This is because the log spiral always makes an angle of $(90^\circ + \phi)$ with its radius vector.

$N\gamma$ Factor

Mobilization factor m

To evaluate the pressures developed on the level side, partial mobilization, characterised by a mobilization factor m as given in assumption 3, has been considered. The values of m for a typical case are given in Table 2. It can be seen from column 6 of this table that $\Sigma H = 0$ condition is satisfied

TABLE 2

Illustration of the Details of Computation of N_γ —Factor for $\phi=40°$, $e/B=0.1$ and $x_1=1.0$

m	α_1	α_2	Pressures $\frac{p_{p\gamma}}{\gamma B^2}$	Pressures $\frac{p_{m\gamma}}{\gamma B^2}$	Value of ΣH	N_γ	value for $\Sigma H=0$ condition
1	2	3	4	5	6	7	8
0.0	40.00	51.54	83.10	00.00	0.000	83.40	83.40
	45.00	52.85	72.59	00.40	+6.003	72.56	
	50.00	54.99	66.09	00.51	+11.055	65.38	
	55.00	57.98	64.05	00.67	+16.010	62.23	
0.2	40.00	51.54	83.40	00.96	−0.642	84.11	82.90
	45.00	51.33	74.77	1.10	+6.440	73.50	
	50.00	52.21	66.00	1.30	+10.8280	65.80	
	55.00	54.32	55.95	1.50	+13.450	55.11	
0.4	40.00	51.54	83.40	2.53	−1.378	85.52	80.80
	45.00	49.04	64.27	2.27	+4.443	65.97	
	50.00	47.73	51.10	2.71	+7.604	52.99	
	55.00	48.38	44.59	3.35	+9.874	45.98	
0.6	40.00	51.54	83.40	4.70	−1.973	87.66	76.10
	45.00	43.69	53.53	6.63	+2.729	60.16	
	45.85	39.37	44.17	7.47	+2.972	51.47	
0.8	40.00	51.54	83.40	12.79	−3.883	95.59	—
	42.08	42.74	58.42	18.10	−0.670	76.26	
1.0	40.00	51.54	83.10	36.01	−7.204	118.68	—
	40.73	*40.78*	*68.60*	*46.69*	*−3.828*	*115.05*	

upto a certain maximum value of m. In this particular case, m = 0.73 beyond which $\Sigma H = 0$ condition is not satisfied. Further $N\gamma$ values listed in column 8 indicate that it decreases with the increase in m.

The values of m for different values of $\frac{De}{B}$, β and e/B for a value of $\phi = 40°$ are given in Table 3.

It can be seen from this table that value of m increases with (*i*) increase in $\frac{De}{B}$ (*ii*) decrease in β and (*iii*) decrease in e/B. For $\beta = 0$ and $e/B = 0$ 'm' becomes unity.

TABLE 3

Values of m for N_γ ; $\phi = 40^\circ$

D_e/B	β	e/B	m
0.0	30°	0.0	0.635
1.0	30°	0.0	0.805
2.0	30°	0.0	0.908
3.0	30°	0.0	0.967
0.5	30°	0.0	0.733
0.5	20°	0.0	0.822
0.5	10°	0.0	0.929
0.5	5°	0.0	0.972
1.0	20°	0.0	0.890
1.0	20°	0.1	0.660
1.0	20°	0.2	0.430
1.0	20°	0.3	0.220

Contact Width Factor x_1

Computations show that the bearing capacity factors are not affected by the pattern of variation of x_1 while the extent of failure surface (L_f in Fig. 1) is significantly affected. This is in accordance to the findings of the work reported by Saran (1969). To ascertain the realistic pattern of contact width factor x_1, model tests have to be performed on the eccentrically loaded footings adjacent to slopes.

Nγ-Charts

Figs. 6 to 9 show $N\gamma$ versus ϕ charts for various values of β, e/B, $\frac{D_f}{B}$ and $\frac{De}{B}$. A careful study of these charts clearly indicate that Nγ-factor increases with (*i*) decrease in β, (*ii*) decrease in e/B, (*iii*) increase in $\frac{D_f}{B}$ and (*iv*) increase in $\frac{De}{B}$. On analysing these charts, minimum edge distance ratio $\left(\frac{De}{B}\right)_{min}$ can be obtained beyond which the presence of slope ceases to influence Nγ-value. In other words the footing will behave as if it is

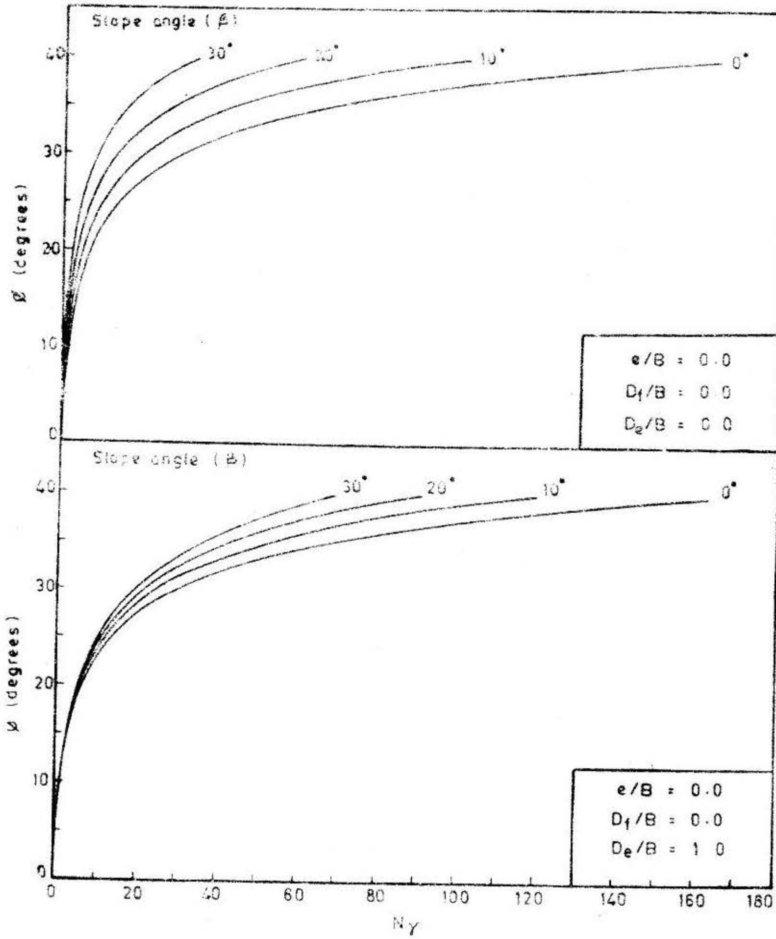

FIGURE 6 N_γ *vs.* ϕ

placed on a level ground. The minimum edge distances for various values of ϕ, β, e/B and $\frac{D_f}{B}$ are given in Fig. 10.

(*iv*) The values of Nγ obtained from proposed theory for β=0 and e/B =0 case i.e. centrally loaded footing resting on flat ground is compared with Terzaghi's theory. Table 4 shows such comparison. It is evident from this table that Nγ values obtained from proposed approach are higher than Terzaghi's values. The difference is significant for higher values of ϕ ($\phi > 20°$).

It is generally known that Terzaghi's values give conservative estimates. Experiments performed on model and full scale footings by Muhs and Kahl (1954), Feda (1961), Selig and Mckee (1961), DeBeer (1965) and many others

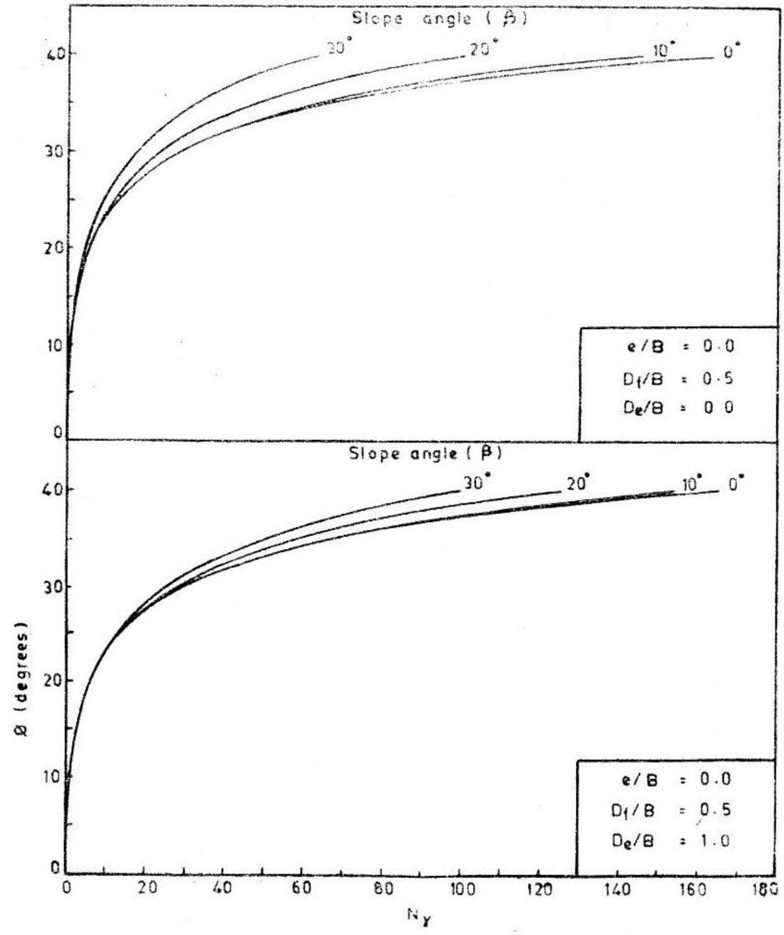

FIGURE 7 N_γ *vs.* ϕ

have shown that the Terzaghi's analysis underestimates the bearing capacity. Hence increased $N\gamma$ values in the proposed analysis may be more nearer to the realistic values.

Comparison of the Proposed Limit Equilibrium Analysis with Previous Investigations

Table 5 shows the comparison between the N_γ values obtained from the proposed method and those obtained from other existing solutions. The table shows that the values of N_γ obtained by the present study are higher than that of the values of the previous analytical investigations. The difference may be attributed to the difference in the rupture surface, in the methodology applied for estimating the N_γ value and its optimization e.g., in case of Meyerhof (1957), Chen (1975) and most of the other methods, the passive

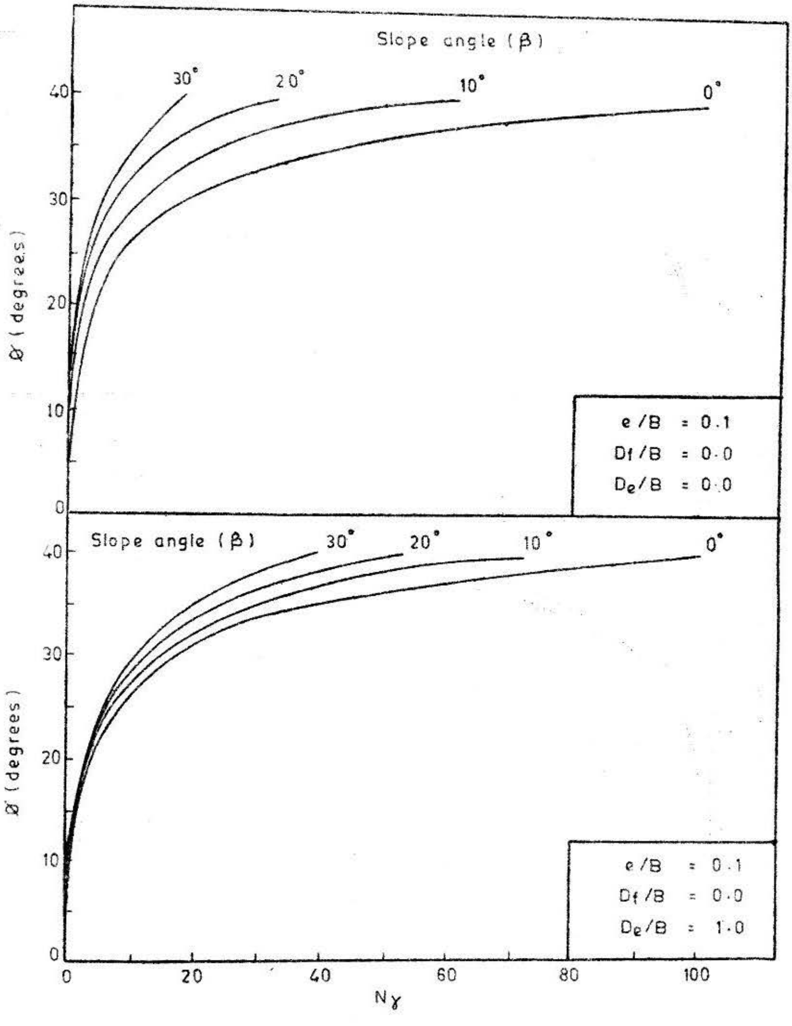

FIGURE 8 N_γ *vs.* ϕ

earth pressures developed on both the sides of triangular wedge have been considered equal, while in the present study the pressures developed on the two sides are different from each other; the higher value being on the side without slope.

N_q Factor

Mobilization factor

Table 6 gives the value of the mobilization factor m for evaluation of N_q. The values of m follow the same trend as was observed in the case of N_γ .

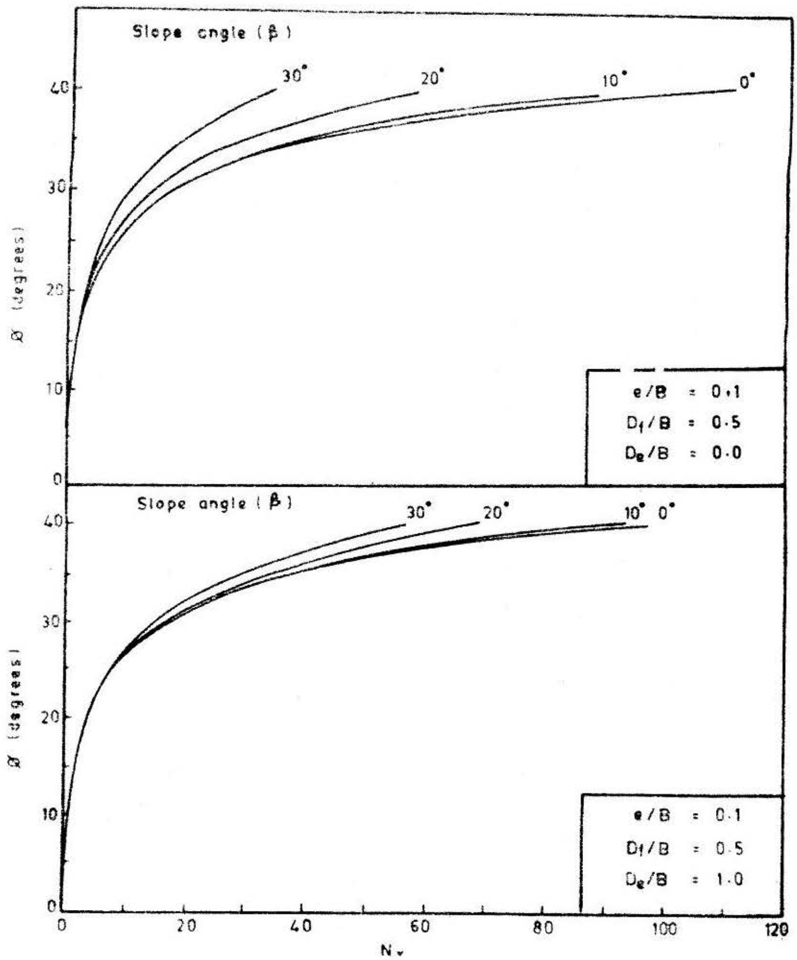

FIGURE 9 N_γ *vs.* ϕ

TABLE 4

Comparison of N_γ values with Terzaghi's values for $\beta=0^\circ$

ϕ	Present Analysis N_γ	Terzaghi's values N_γ
20°	6.05	5.00
30°	29.35	19.70
40°	165.38	100.40

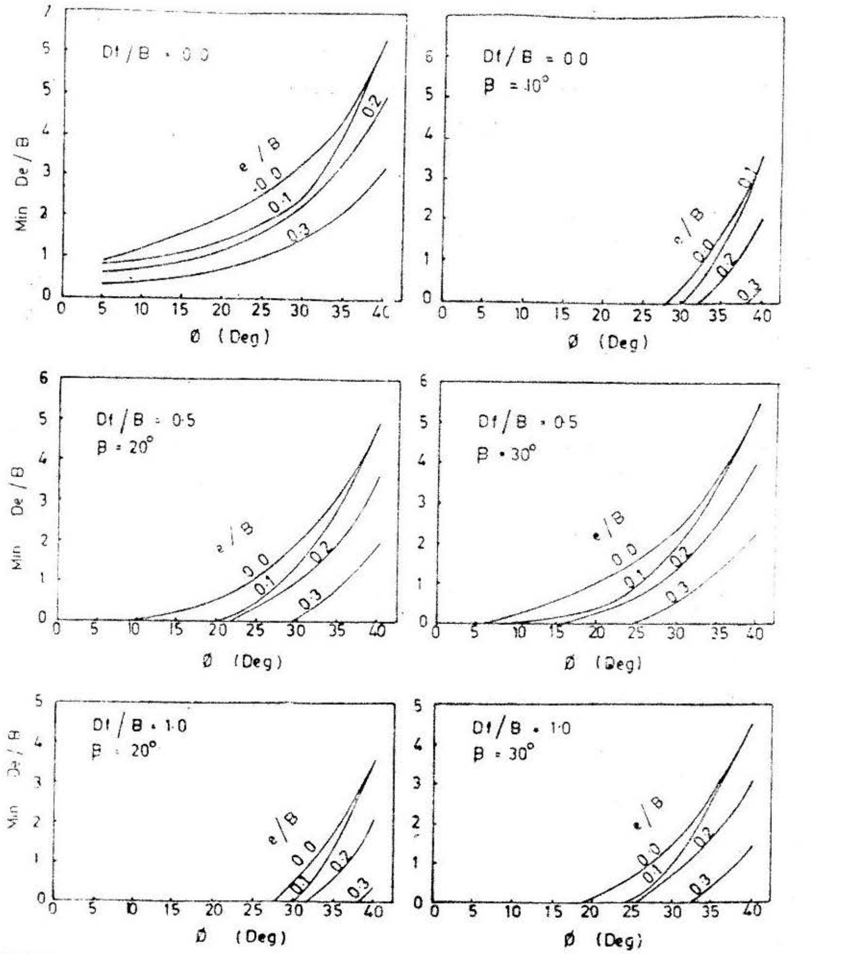

FIGURE 10 Minimum Edge Distance for N_γ Value to become Independent of Slope

TABLE 5

Comparison of Present Theory with other Existing Solutions

ϕ	β	D_e/B	D_f/B	Meyerhof	Mizuno	Siva Reddy Mogaliah	Chen	Proposed Theory
40°	30°	0.0	0.0	20.0	17.0	—	19.5	25.37
40°	30°	1.0	0.0	40.0	—	—	—	62.20
40°	20°	0.0	0.0	34.0	44.0	—	55.0	53.47
40°	20°	1.0	0.0	55.0	—	—	—	85.98
40°	20°	2.0	0.0	70.0	—	—	—	121.22
40°	20°	0.0	1.0	125.0	—	—	—	168.00
30°	30°	0.0	0.0	3.1	—	5.01	—	6.14
30°	20°	0.0	0.0	7.5	8.0	—	10.0	11.61
30°	15°	0.0	0.0	10.0	11.0	13.76	12.0	15.25
30°	15°	0.0	0.68	30.0	—	33.60	—	32.20

TABLE 6

Mobilization Factor for Evaluation of N_q

ϕ	β	D_e/B	D_f/B	m
40°	30°	1.0	0	0.565
		1.0	0.5	0.631
		1.0	1.0	0.738
30°	20°	0	0.5	0.599
		1.0	0.5	0.775

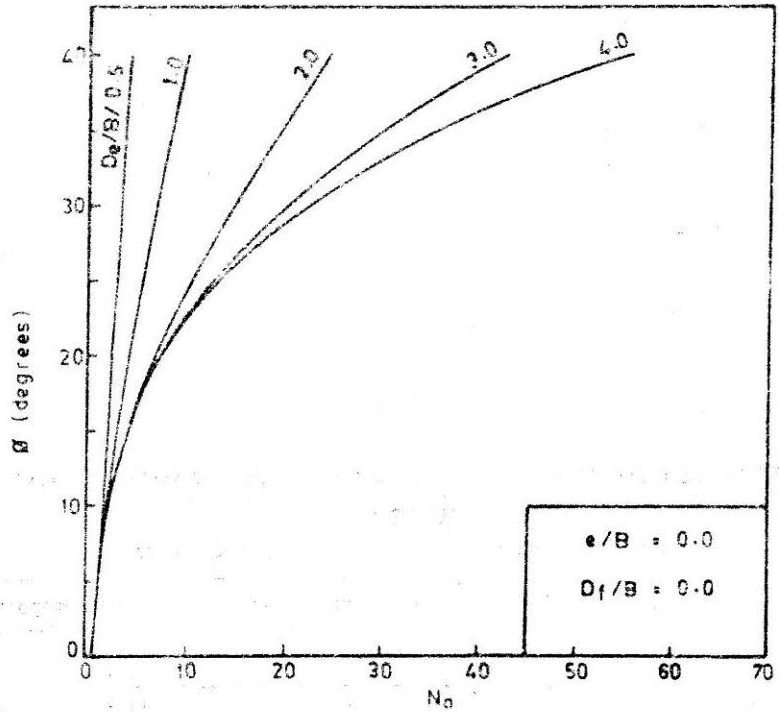

FIGURE 11 N_q *vs.* ϕ

N_q Charts

It can be seen from the figures 11 to 14 that the shape of the charts of N_q those factor is different than of N_γ . With the increase in ϕ value, the rate of increase of the N_q value is small in comparison to those of N_γ values. Minimum edge distance factor for N_q are shown in Fig. 15.

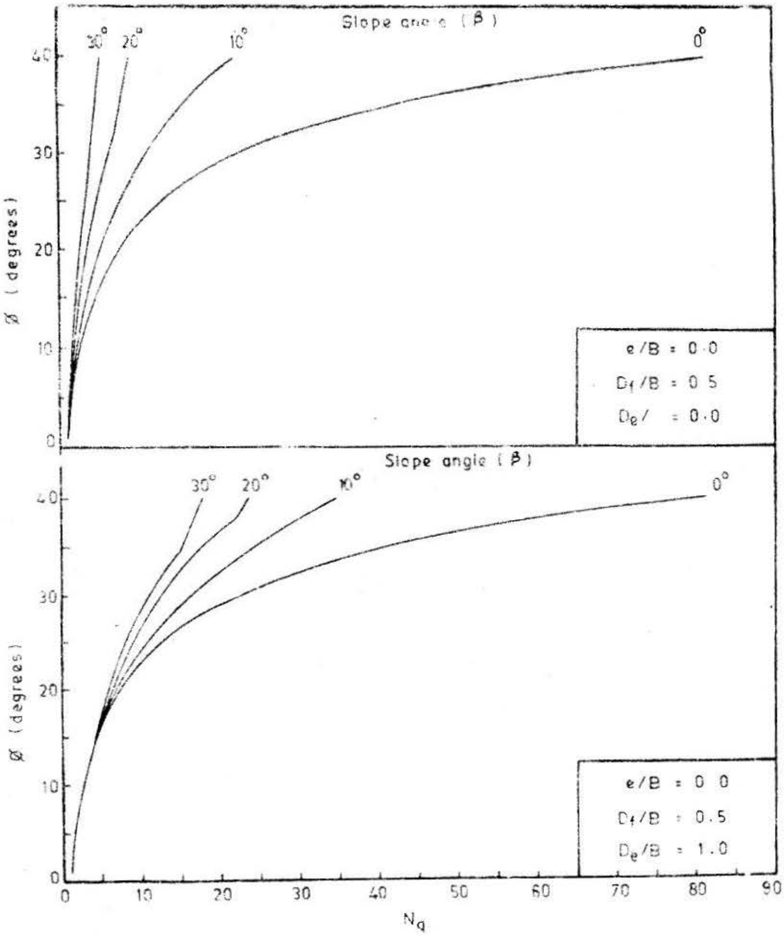

FIGURE 12 N_q *vs.* ϕ

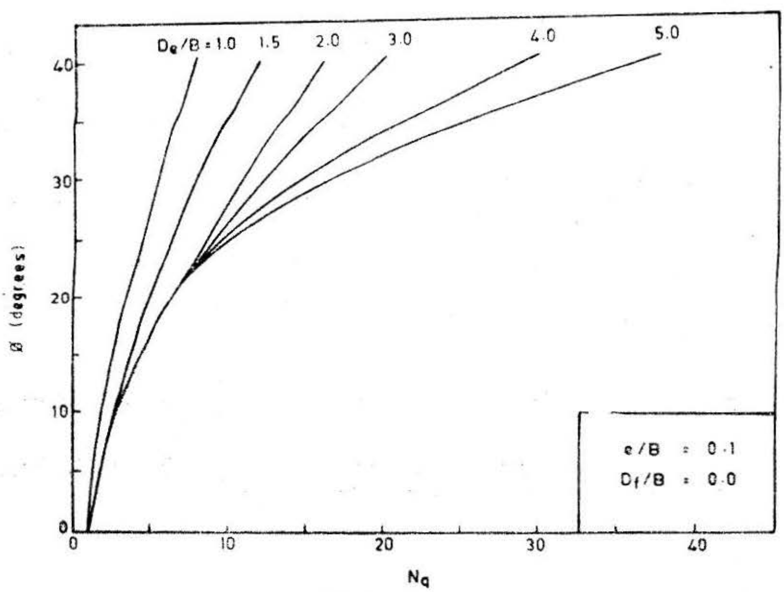

FIGURE 13 N_q *vs.* ϕ

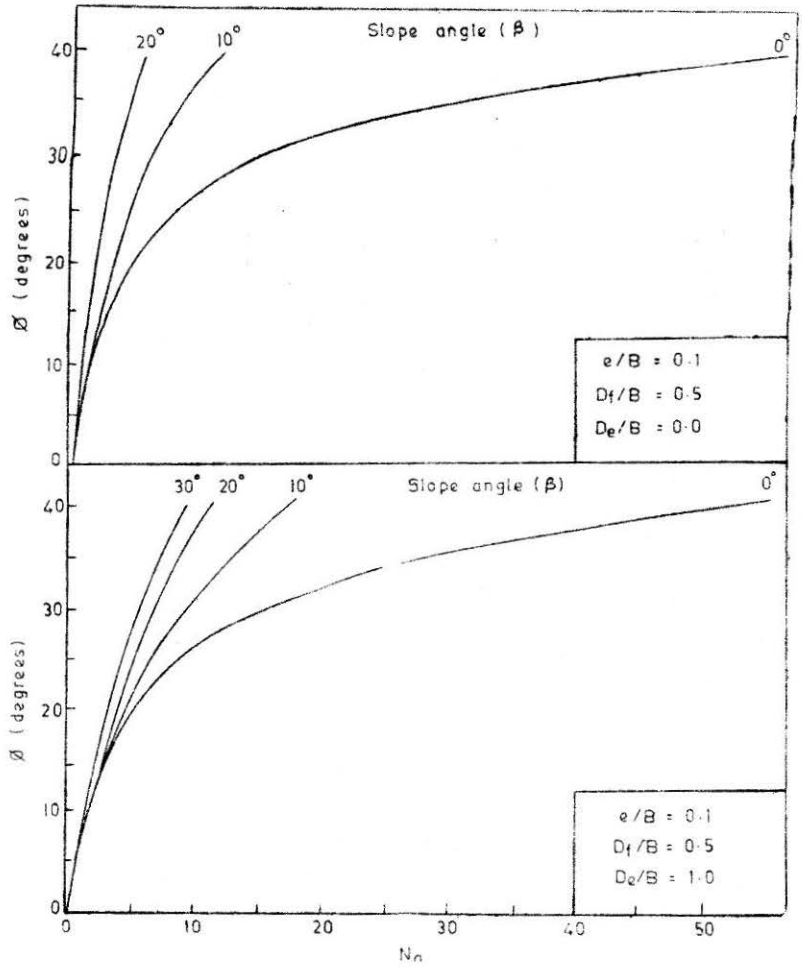

FIGURE 14 Minimum Edge Distance for N_q Value to become Independent of Slope

(*iii*) Comparison of N_q values with Terzaghi's values for $\beta = 0$ Case

Table 7 gives the comparison of N_q values from the present study for footings on level ground ($\beta = 0$) with Terzaghi's values. The values of N_q from the two methods are same.

Comparison of bearing capacity values with Siva Reddy et al. and Meyerhof

Table 8 gives the comparison of bearing capacity values with those of Siva Reddy and Meyerhof. The values obtained by the present study are higher.

TABLE 7

Comparison of N_q values with Terzaghi's value for $\beta = 0$

ϕ	Present Analysis N_q	Terzaghi's Value N_q
20°	7.4	7.4
30°	22.46	22.5
40°	81.3	81.3

TABLE 8

Comparison of Bearing Capacity Values with those of Siva Reddy *et al.* (1975) and Meyerhof (1957)

ϕ degrees	β degrees	D_e/B	D_f/B	Siva Reddy et al. kpa	Meyerhof kpa	Present study kpa
30	30	1	1	—	64.80	86.8
30	15	0	0.68	53.80	48.00	62.0
30	15	0	0.31	32.16	27.71	38.7
30	30	0	0	8.16	5.05	9.5
40	20	0	1	—	203.80	268.9

Conclusions

Bearing capacity of an eccentrically loaded footing adjacent to a slope increases with (*i*) increase in edge distance, (*ii*) increase in depth of footing, (*iii*) decrease in slope angle and (*iv*) decrease in eccentricity.

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NOTATIONS

Symbol		Description	Unit
B	=	Width of footing	m
D_e	=	Distance of the edge of the foundation from the shoulder of the slope	m
D_f	=	Depth of foundation	m
e	=	Eccentricity	m
m	=	Mobilization factor	
N_q	=	Bearing capacity factor for surcharge part	
N_r	=	Bearing capacity factor for weight part	
N_{pq}, N_{pr}	=	Non-dimensional factors for surcharge and weight	
N_{mrl} N_{mq}	=	Mobilized non-dimensional factors for wieght and surcharge	
P_p	=	Passive pressure	N
P_m	=	Mobilized passive pressure	N
P_{pq}	=	Passive pressure for surcharge part	N
P_{pr}	=	Passive pressure for weight part	N
P_{mq}	=	Mobilized passive pressure for surcharge part	N
P_{mr}	=	Mobilized passive pressure for weight part	N
q	=	Surcharge intensities	KN/m^2
q'			
r_o	=	Initial radius of logarithmic spiral	m
r_1	=	Radius of logarithmic spiral at angle θ ground side	m
r_o'', r_1''	=	Non-dimensional radius of logarithmic spiral on flat ground side	
r_o'	=	r_o/B	
r_1'	=	r_1/B	
X_1	=	Contact width factor	
α_1	=	Wedge angle towards the slope side	*Deg.*

α_2	= Wedge angle towards the level ground side	*Deg.*
β	= Slope angle	*Deg.*
γ	= Unit weight of soil	*Deg.*
ϕ	= Angle of internal friction	*Deg.*
ϕ_m	= Mobilized angle of internal friction	*Deg.*
o	= Stress	*k Pa*
τ	= Shear stress	*k Pa*
θ	= Angle of logarithmic spiral on the side of the slope	*Deg.*
θ'	= Angle of logarithmic spiral on the side of flat ground	*Deg.*