Indian Geotechnical Journal, 18(3) 1988

Footings on Slopes and Constitutive Laws

by

Swami Saran*

V.K. Sud**

S.C. Handa***

Introduction

Foundations are some times placed on slopes, near the top edge of a slope or near a proposed excavation. Several theories are available to compute the ultimate bearing capacity of foundations adjacent to slopes (Meyerhof (1957), Mizuno et al. (1960), Sokolovski (1960), Bowles (1977), Kusakabe et al. (1981), Sud (1984). However, the best estimation of both bearing capacity and the settlement is possible only if the pressure settlement characteristics of the foundation soil are known. No method is reported to evaluate the settlement of a footing on or near the slopes.

Soil, in general, is an anisotropic material and Young's modulus E is dependent upon confining pressure. Methods are not available where variations in E with confining pressure, are considered for computing the stresses in the soil medium. This has relevance in estimation of foundation settlements adjacent to a slope. Hence a semi-empirical method is formulated to estimate the settlement of a footing adjacent to a slope.

Since confining pressures are the major criterion to evaluate the settlement of soil mass, it is assumed that these are provided by the passive earth pressure developed on the side of the slope. Maximum shearing resistance has been assumed to develop at the base of the footing and minimum at the depth where the stresses due to foundation loads become zero. The degree to which the strength can be mobilized is directly dependent on the movement of the soil mass (Terzaghi 1943). The movement will be maximum at the base of the foundation and it decreases with depth.

The constitutive relations of soil represented by hyperbola were established from triaxial compression tests.

---

*Professor, Department of Civil Engineering, University of Roorkee, U.P., India

**Associate Professor, Department of Civil Engineering, Thapar Engineering College, Patiala, Punjab, India

***Professor & Co-ordinator Q.I.P., University of Roorkee, U.P., India.

This modified manuscript of this paper was received in January 1988 and is open for discussion till end of Dec 1988)

Analysis

Assumptions

The following assumptions have been made in the analysis.

1. The footing base has been assumed as flexible so as to have uniform contact pressure distribution for computing pressure settlement curves. The average settlement of a flexible footing is reported to be almost the same as that of settlement of a rigid footing for average pressure intensity (Sharan, 1977). Therefore the pressure versus average settlement curve computed by the suggested semi-empirical approach may be taken as for rigid strip footing.

2. The soil mass supporting a footing has been assumed to be a vertical column of soil as shown in Fig. 1 and this column of soil has been divided into large number of thin horizontal strips in which stresses and strains have been assumed to be uniform along any vertical section.

3. The passive earth pressure has been evaluated considering the failure surface as a log spiral with the centre of rotation at the edge of the footing.

4. Shear stresses have been considered to vary linearly along the length of a strip.

5. The ultimate bearing capacity q_u has been computed from limit equilibrium analysis as proposed by Sud (1984).

6. A coefficient 'F' has been introduced such that at all stress levels, the following relationship is satisfied.

$$\frac{q_u}{q} = \frac{\sigma_u}{\sigma_1 - \sigma_3} = F \qquad \dots (1)$$

   where q is the intensity of load, σ_u is the ultimate stress from hyperbolic relationship and is equal to $\left(\frac{1}{b}\right)$ in which 'b' is Kondner's coefficient found from triaxial testing (Kondner, 1963). σ_1, σ_2 are the major and minor principal stresses in the soil mass due to the load q and weight of soil.

   This indicates that the same factor of safety exists between the applied pressure with respect to ultimate bearing pressure and deviator stress at a point with respect to the ultimate stress at confining stress σ_3.

7. There is no slip between sucessive strips of the soil column.

Formulation and Procedure

The procedure for evaluation of settlement of uniformly loaded strip footing near the edge of slope on soils is described in the following steps.

Step 1. For a given load, the depth H of the soil mass under the footing at which the stresses become zero is assumed (Fig. 1).

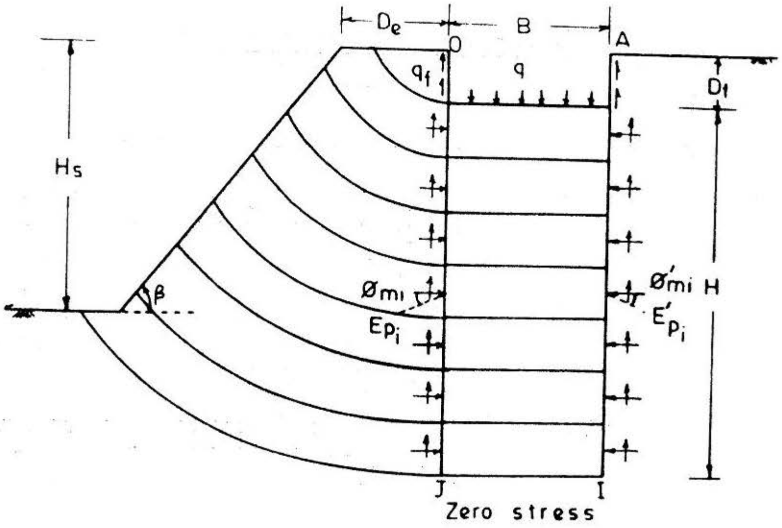

FIGURE 1 Soil Mass Below Footing

Step 2. The column of soil is divided into n number of thin horizontal strips.

Step 3. Angle of shearing resistance and cohesion values are varied from full value of ϕ and c at the base of the footing to zero at depth H where the stresses become negligible. Average values of ϕ and c on i^{th} strip are given as below:

$$\phi_{m_i} = \phi - \frac{\phi\,[i\,(\Delta H) - \frac{\Delta H}{2}]}{H} \qquad \ldots (2)$$

$$c_{m_i} = c - \frac{c[i\,(\Delta H) - \frac{\Delta H}{2}\,]}{H} \qquad \ldots (3)$$

Where ϕ_{mi} = mobilized value of ϕ at i^{th} strip,

ΔH = the thickness of the strip, and

i = number of the strip.

c_{mi} mobilized value of cohesion at i^{th} strip.

Step 4. The confining pressure (σ_3) acting at the centre of each strip is taken as

$$\sigma_3 = E_{pi} \text{ Cos } \phi_{mi} \tag{4}$$

and E_{pi} is the passive resistance offered to each strip and has been evaluated by calculating the moments of resistance due to soil mass as shown in Fig. 1. ϕ_{mi} is the average ϕ for i^{th} strip.

$$E_{pi} = \frac{M_\gamma(i)+M_c(i)-M_\gamma(i-1)-M_c(i-1)}{\Delta H \dfrac{[(i+(i-1)]}{2}+D_j} \quad \ldots (5)$$

Where E_{pi} = passive earth pressure on *ith* strip,

$M_\gamma(i)$ = Moment of resistance due to weight only considering the height of face equal to $[i\,\Delta H + D_f]$

$M_\gamma(i-1)$ = Moment of resistance due to weight only considering the height of face equal to $[(i-1)\,(\Delta H) + D_f]$

$M_c(i)$ = Moment of resistance due to cochesion only considering the height of face equal to $[i(\Delta H) + D_f]$

$M_c(i-1)$ = Moment of resistance due to cohesion only considering the height of face equal to $[(i-1)\Delta H + D_f]$

To evaluate passive earth pressure E_p for a particular depth, the failure surface has been taken as log spiral having centre of rotation at O (Fig. 2) the edge of the footing at the ground surface.

$$R_0 = OD = i\Delta H + D_f \tag{6a}$$

$$R_1 = OE = R_0\, e^{\theta} \;\; tan\; \phi_{mi} \tag{6b}$$

Where θ is the angle of the log spiral. R_0 is the initial radius of the log spiral and R_1 is the final radius. There are two cases, case I where the rupture surface meets the slope (Fig. 2*a*) and case II the rupture surface meets the base of the slope (Fig. 2*b*).

Case I Rupture surface meeting the slope

$$R_1 = \frac{D_e \sin\beta}{\sin(\beta+\theta-90)} \quad \ldots (7)$$

From Eqs. 6 and 7

$$R_o\, e^{\theta} \tan \phi_{mi} = \frac{D_e \sin\beta}{\sin(\beta+\theta-90)} \quad \ldots (8)$$

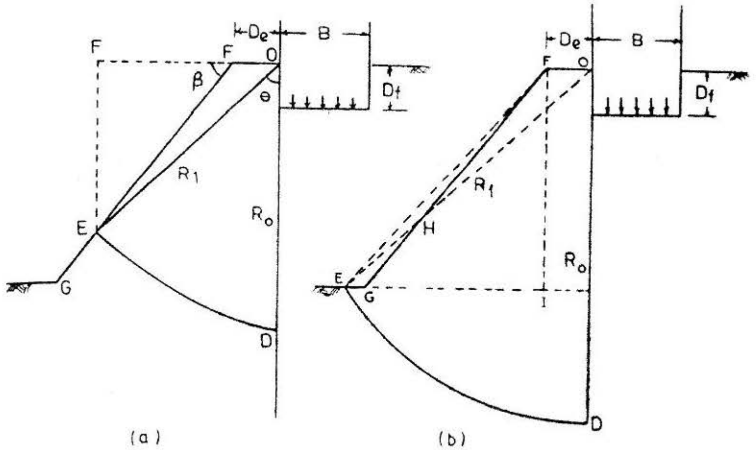

FIGURE 2 Rupture Surface for Evaluating E_p

D_e =Distance of the edge of the foundation from the shoulder of the slope.

β = Angle which the slope makes with the horizontal.

From the above transcedental equation the value of θ is obtained by trial and error.

Taking moments about the point O, the centre of rotation.

$$M_\gamma (i) = \frac{\gamma \; R_0^3}{3\,(9 \tan^2 \phi+1)} \left[e^{3\theta \tan \phi_{mi}} (3 \tan \phi \operatorname{Sin} \theta - \operatorname{Cos} \theta) + 1\right]$$
$$+ \tfrac{1}{3} \gamma R_0^3 e^{3\theta \tan \phi_{mi}} \operatorname{Cos} \theta \operatorname{Sin}^2\theta$$
$$- \tfrac{1}{2} \gamma R_0 e^{\theta \tan \phi_{mi}} (R_0 e^{\theta \tan \phi} - D_e) \operatorname{Cos} \theta$$
$$\times [\tfrac{2}{3} R_0 e^{\theta \tan \phi_{mi}} \operatorname{Sin} \theta + \tfrac{1}{3} D_e] \quad \ldots (9)$$

$$M_c (i) = \frac{c_{mi} R_0^2}{2 \tan \phi_{mi}} (e^{\theta \tan \phi_{mi}} - 1) \quad \ldots (10)$$

Case II Rupture surface meeting the base of the slope

$$R_1 = \frac{H_s}{\operatorname{Cos} \theta} \quad \ldots (11)$$

From the Eqs. 6 and 11 the value of θ is obtained by trial and error.

Taking moments about the point,

$$M_\gamma(i) = \frac{\gamma\ R_0^3\ e^{3\theta \tan \phi_{mi}}}{3\,(9 \tan^2 \phi + 1)} [e^{3\theta \tan \phi_{mi}}\ (3\ \tan \phi_{mi} \operatorname{Sin} \theta - \operatorname{Cos} \theta) + 1]$$

$$-\tfrac{1}{6}\,\gamma H_s\,(\operatorname{Sin} \theta\, R_0\, e^{\theta \tan \phi_{mi}} - D_e)\,(\operatorname{Sin} \theta\, R_0\, e^{\theta \tan \phi_{mi}} - 2D_e)$$

$$+ \tfrac{1}{2}\,\gamma\,\frac{H_s^2}{\tan \beta}\left(\tfrac{1}{3}\ \frac{H_s}{\tan \beta} + D_e\right)$$

$$+ \tfrac{1}{3}\,\gamma\ H_s \operatorname{Sin}^2 \theta\ R_0^2\ e^{2\theta \tan \phi_{mi}}$$

$$- \tfrac{1}{3}\,\gamma\ H_s\ (R_1 \operatorname{Sin} \theta - D_e)^2 \qquad \dots (12)$$

From the Eqs. 9 or 12 and 10 the value of $M_\gamma\,(i)$ and $M_c(i)$ are obtained for a particular depth $i\Delta H + D_f$. Similarly the value of $M_\gamma\,(i-1)$ and $M_c(i-1)$ are computed by using the value of R_0 as $[(i-1)\Delta H + D_f]$ in Eqs. 6*b* to 12. These values of moments when substituted in equation 5 give value of E_{pi}.

Step 5. Force on the sides of the wall of the footing is calculated by the equation

$$q_f = \tfrac{1}{2}\,\gamma\ D_f^2\ K_0 \tan \delta + c_a\ D_f \qquad \dots (13)$$

Where, δ = angle of wall friction

K_0 = coefficient of earth pressure at rest = $(1 - \operatorname{Sin} \phi)$

c_a = adhesion between the side wall and soil.

Step 6. The vertical load q taken up by any particular height of soil mass H can be computed by considering the overall equilibrium of soil column (Fig. 3). As the value of ϕ and cohesion c have been assumed to vary with depth, the earth pressure developed on the side of the column will also vary with depth. For this the earth pressure on different strip faces have been computed using Eq. 5.

$$q.B = \sum_{i=1}^{n} E_{pi} \operatorname{Sin} \phi_{mi} + \sum_{i=1}^{n} E'_{pi} \operatorname{Sin}\ \phi'_{mi} - \gamma BH + 2q_f$$

$$+ 2 \sum_{i=1}^{n} c_{mi}\,\Delta H \qquad \dots (14)$$

Where, q = intensity of loading

E_{pi} = earth pressure on the side of the slope at i^{th} strip inclined at mobilised angle ϕ_{mi}.

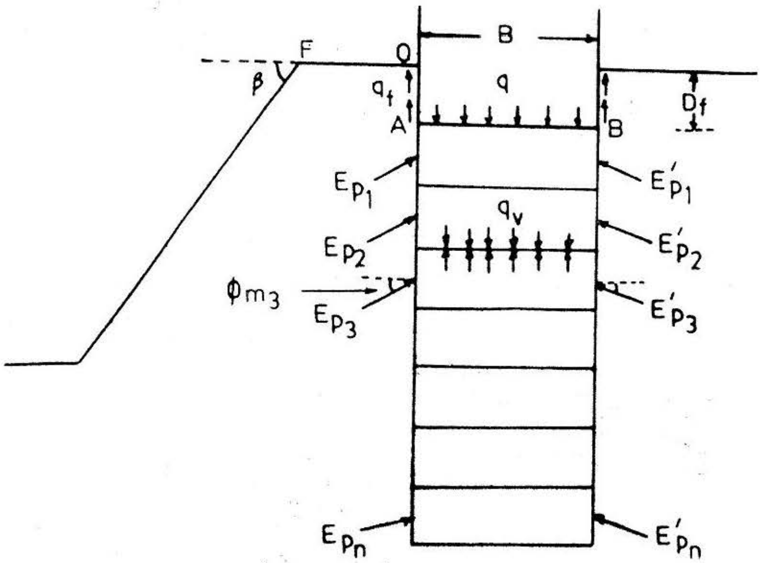

FIGURE 3 Overall Equilibrium of Soil Column

$E'pi$ = earth pressure on the side away from slope at the i^{th} strip inclined at an angle ϕ'_{mi}

$n = H/\Delta H$

Further, the sum of horizontal forces is also zero.
It gives:

$$\Sigma E_{pi} \text{ Cos } \phi_{mi} = \Sigma E_{pi} \text{ Cos } \phi'_{mi} \qquad \dots (15)$$

It is assumed that

$$E_{pi} = E'_{pi} \qquad \dots (16)$$

$$\phi_{mi} = \phi'_{mi} \qquad \dots (17)$$

Equation (14) may be written as:

$$\frac{q}{\gamma H} = \frac{2 \sum_{i=1}^{n} E_{pi} \text{ Sin } \phi_{mi}}{\gamma BH} - 1 + \frac{D_f^2 K_0 \text{ Tan } \delta}{BH} + \frac{2 \sum_{i=1}^{n} c_{mi} \Delta H}{\gamma BH}$$

$$+ \frac{2c_a D_f}{\gamma BH} \qquad \dots (18)$$

Step 7. A plot is drawn between the values of H/B and $q/\gamma H$ from equation (18) and the value of H is obtained from this plot for a given load intensity q.

Step 8. The vertical stress on any strip is calculated by considering the static equilibrium of the forces ($\Sigma V=0$) acting on the strip (Fig. 4).

$$q_{vi}\, B = 2 \sum_{i=1}^{i-1} E_{mi} \operatorname{Sin} \phi_{mi} - \gamma B\,(i-1)\,\Delta H + \gamma D_f^2\ K_0 \tan \delta + 2c_a\ D_f + 2 \sum_{i=1}^{i-1} c_{mi} \Delta H \qquad \ldots (19)$$

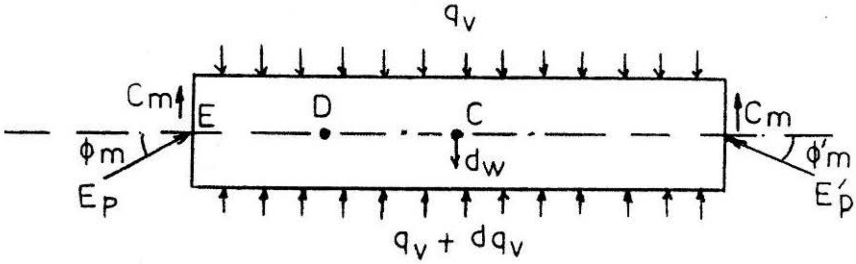

FIGURE 4 Stresses on the Strip

From this equation, vertical stress on a particular strip is obtained.

Step 9. The confining stress (σ_3) for the i^{th} strip is taken as $\sigma_3 = E_{pi}\ Cos\ \phi_{mi}$

Step 10. The ultimate strength (σ_u) of soil for a given confining pressure σ_3 is computed from the constitutive law of soil obtained by triaxial testing in the lab.

Step 11. The ultimate bearing capacity (q_u) is obtained from the limit equilibrium analysis given in reference 9.

Step 12. The shear stresses on the side of a strip are taken as ($E_{pi} \operatorname{Sin} \phi_{pi} + c_{mi} \Delta H$) and are assumed to vary linearly along the width of the strip. It becomes zero at the centre of the strip i.e. at point E (Fig. 4)

Step 13. The state of stresses are obtained for three points $C, D, E,$ on the horizontal plane passing through the centre of the strip. C is the centre of the strip, E is at the edge and D is the mid point between C and E (Fig. 4).

The principal stresses and their directions with respect to the vertical Z—axis have been computed using the equations of the theory of elasticity as given below.

$$\sigma_1 = \frac{\sigma_z + \sigma_x}{2} + \sqrt{\left(\frac{\sigma_z - \sigma_x}{2}\right)^2 + \tau_{xz}^2} \qquad \ldots (20)$$

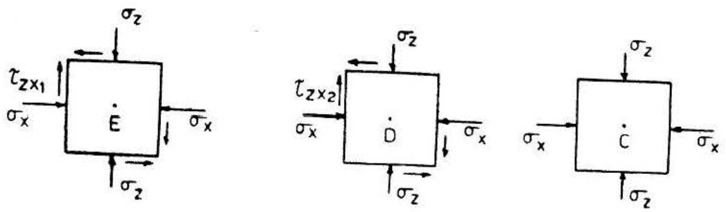

FIGURE 5 Stresses on the Elements

$$\sigma_3 = \frac{\sigma_z + \sigma_x}{2} - \sqrt{\frac{(\sigma_z - \sigma_x)^2}{(\ 2\ )} + \tau_{xz}^2} \quad \ldots (21)$$

$$\tan 2\theta = \frac{2\tau_{xz}}{\sigma_z - \sigma_x} \quad \ldots (22)$$

Positive value of θ is measured counter clockwise with direction of σ_z.

Where σ_1 and σ_3 are the major and minor principal stresses. σ_z is vertical stress, σ_x the stress in horizontal direction and $\mathbf{T}_{xz}$ the shear stress.

Step 14. A coefficient F for a given load intensity (q) is computed from the following relationship.

$$\frac{q_u}{q_v} = F \quad \ldots (23)$$

Where q_u is the ultimate bearing capacity of the footing on slope.

Step 15. The modulus of elasticity, E is calculated from the Fig. 6 at stress level of σ_u/F

$$E = \frac{1 - b\ (\sigma_u/F)}{a} \quad \ldots (24)$$

Where a and b are the constants of Kondner's hyperbolic functions obtained by Triaxial testing.

Step 16. The strain in each layer in the direction of major principal stress is calculated from the equation

$$\epsilon_1 = (\sigma_1 - \sigma_3)/E$$

The strain in the direction of minor principal stress is calculated by the following procedure.

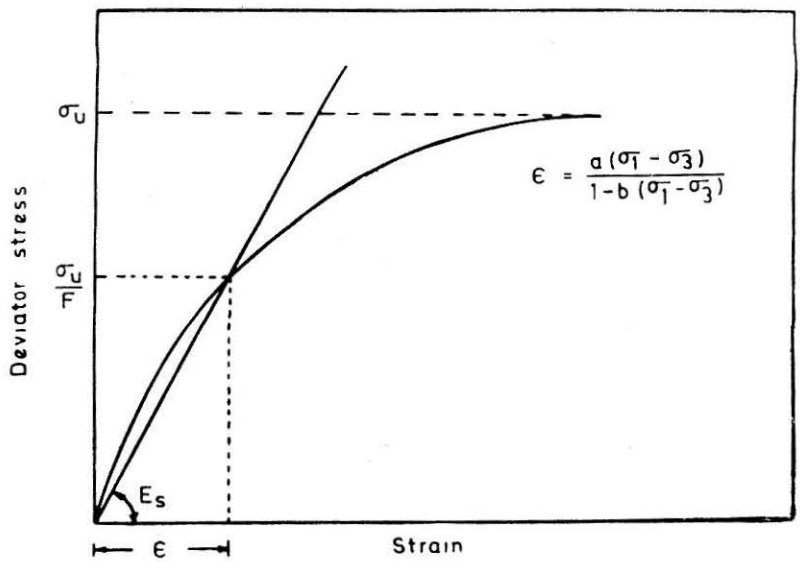

FIGURE 6 Hyperbolic Stress Strain Relationship

Strip footing is a case of plane strain condition. The strain ϵ_2 in the direction of intermediate principal stress σ_2 is zero.

$$\epsilon_2 = 0 = \frac{1}{E} = [\sigma_2 - \mu\,(\sigma_1 + \sigma_2)] \qquad \dots (25)$$

$$\text{or } \sigma_2 = \mu\,(\sigma_1 + \sigma_3) \qquad \dots (26)$$

$$\epsilon_1 = \frac{1-\mu^2}{E}\left[\sigma_1 - \frac{\mu}{1-\mu}\,\sigma_3\right] \qquad \dots (27)$$

$$\epsilon_3 = \frac{1-\mu^2}{E}\left[\sigma_3 - \frac{\mu}{1-\mu}\,\sigma_1\right] \qquad \dots (28)$$

$$\frac{\epsilon_3}{\epsilon_1} = \frac{\sigma_3 - \mu_1\,\sigma_1}{\sigma_1 - \mu_1 \sigma_3} = -\mu_2 \qquad \dots (29)$$

Therefore strain in the direction of minor principal stress is calculated from the *Eq* (29)

$$\epsilon_3 = \mu_2\,\epsilon_1 \qquad \dots (30)$$

Step 17. The strain in the vertical direction is calculated using the following equation.

$$\epsilon_{zi} = \epsilon_1 \text{Cos}^2\,\theta_1 + \epsilon_3 \text{Cos}^2\,\theta_3 \qquad \dots (31)$$

Where θ_1 = the angle which the major principal stress makes with the vertical direction.

θ_3 = the angle of the minor principal stress with the vertical.

Step 18. The settlement of each layer along any vertical section is calculated by the equation.

$$S = \epsilon_{zi} \Delta H \qquad \text{... (32)}$$

Step 19. The evaluation of the total settlement along any vertical section is done by the numerical summation of the settlement of n number of strips as

$$S = \sum_{i=1}^{n} \epsilon_{zi}\, dz \qquad \text{... (33)}$$

The total settlement was computed along vertical section passing through the centre of the footing, at B/4 from the centre and at the end points of the strip. The average settlement is computed by dividing the area of the settlement diagram by the width (B) of the footing.

Step 20. The footing load intensity is varied and steps 1 to 19 are repeated. The pressure settlement curve is obtained by plotting settlements obtained in step 19, against corresponding footing load intensity.

Constitutive Laws

To obtain the result from the analytical procedure for settlements, constitutive laws of the Ranipur sand were established. The physical properties of the Ranipur sand are given in Table I.

TABLE I

Properties of Ranipur Sand

Effective size (D_{10})	0.15 mm
Uniformity coefficient	1.73
Mean specific gravity	2.65
Minimum void ratio	0.57
Maximum void ratio	0.88
Average density at relative density of 84%	16.30 KN/m^3
Average density at relative density of 72%	15.95 KN/m^3
ϕ (triaxial) at 84% relative density	39°
ϕ (triaxial) at 72% relative density	37.5°

To determine the constitutive relationships for the Ranipur sand, a large number of triaxial tests were performed with confining pressures varying from 75 KPa to 500 KPa, at two relative densities of 84% and 72%. The parameters '*a*' and '*b*' of the Kondner's hyperbola were correlated with the confining pressures and relative densities. It was found that the following relationships hold good for Ranipur sand.

$$\frac{1}{a} = A_1 + K_1\, \sigma_3$$

$$\frac{1}{b} = A_2 + K_2\, \sigma_3$$

The values of A_1, A_2, K_1 and K_2 are given in Table 2.

TABLE 2

Parameters of constitutive laws for Ranipur sand

Ralative Density	A_1	A_2	K_1	K_2
84%	800	220	178.0	2.20
72%	500	200	137.5	1.44

Model Tests

Plane strain model tests were performed on Ranipur sand to study the load settlement behaviour of footings adjacent to slope and to compare the settlements obtained by the analytical procedure with that of experimental values. The tests were performed on three slopes of 30°, 26.56° (2 horizontal to 1 vertical) and 20° and at two relative densities of 84% and 72%.

The footing used was 0.12 m wide and 0.6 m long. This length was equal to the width of the tank. The inside dimensions of the tank used were 0.6 m in width, 3 m in length and 0.9 m in height. Settlement of the footings was measured and the loading was done up to failure. The experimental set up is shown in Fig. 7.

Interpretation

The average settlements for strip footings of width 10 cm, 15 cm, 30 cm and 45 cm resting on Ranipur sand and slope of 30° were computed. The footings were considered at an edge distance of $D_e/B = 1.0$. The pressure versus settlement curves are plotted in Fig. 8.

The curves show that for a particular footings, the settlement increases with load intensity and the rate of increase also increases at high load inten-

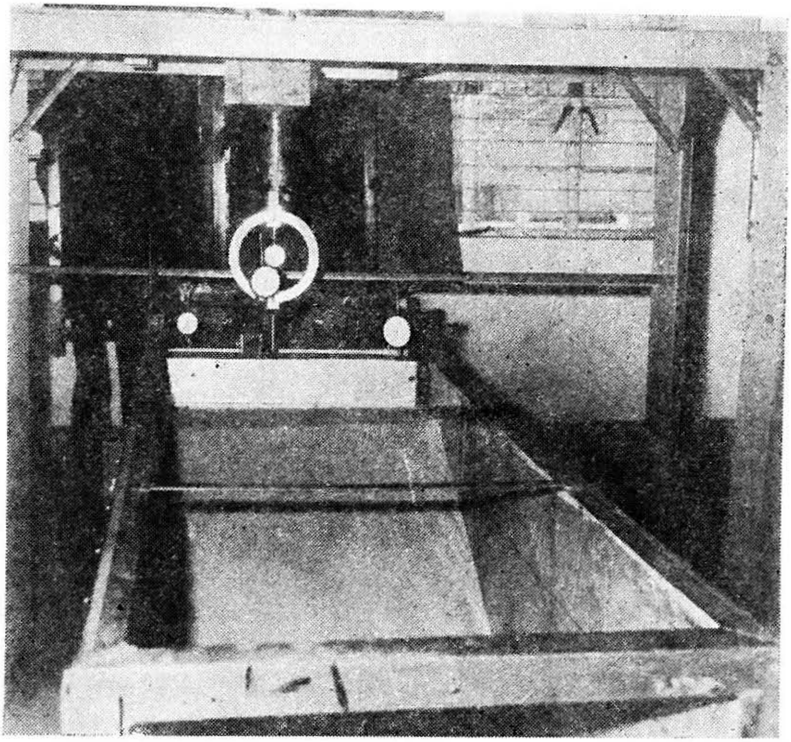

FIGURE 7 Experimental Set-up

sities. It can also be seen from the curves that as the footing size increases the settlement also increases.

The pressure settlement curves for a 30 cm footing for 30° slope have been plotted in Fig. 9. For $D_e/B = 0.0$, 1.0, 2.0 and 3.0 for the relative density of 84%. It is evident from this figure that the settlement of the footing at a given pressure decreases with the increase in edge distance.

The settlements of a 30 cm wide footing resting at edge distance De/B =1.0 for three slope angles 20°, 25° and 30° are shown in Fig. 10. The settlement at given pressure increases with the increase in slope angle.

Comparison With Test Results

The comparison of model test data with proposed analysis has been made by considering an increase of 2° in ϕ at 84% relative density and 1.5° increase in ϕ at 72 percent relative density to account for the difference in ϕ values obtained by the two types of tests i.e., triaxial and plane strain test (Cornforth 1964).

Pressure settlement curves obtained by this analysis have been compared with the experimental curves for 12 cm wide surface footings for different

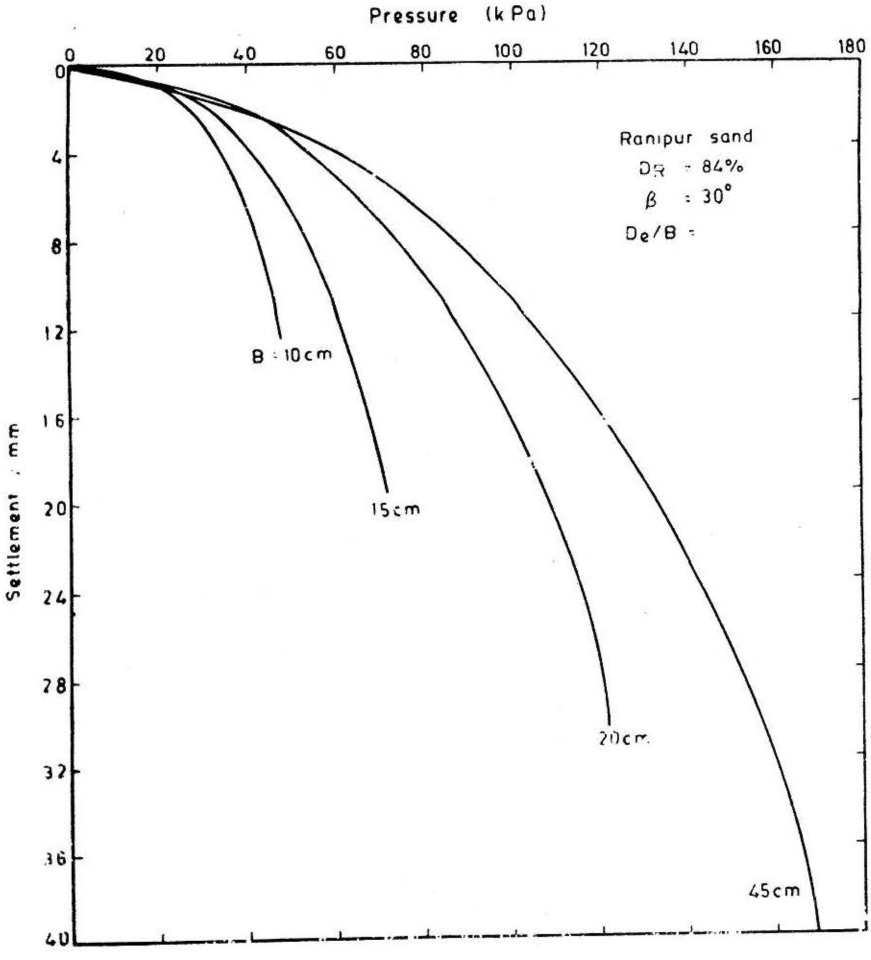

FIGURE 8 Pressure Settlement Curves for Strip Footings of Different widths

edge distances and slope angles in Figs. 11 to 13. In all, settlements were compared with 44 Model tests. The two curves are similar and show good agreement. In the initial portion of the curve, the predicted values of settlement give slightly lower values and in the later stages of the curve, the predicted values become higher than the experimental values. This discrepancy may be due to the use of hyperbolic form of stress-strain curve. The hyperbola remains below the asymptote at all finite values of strain.

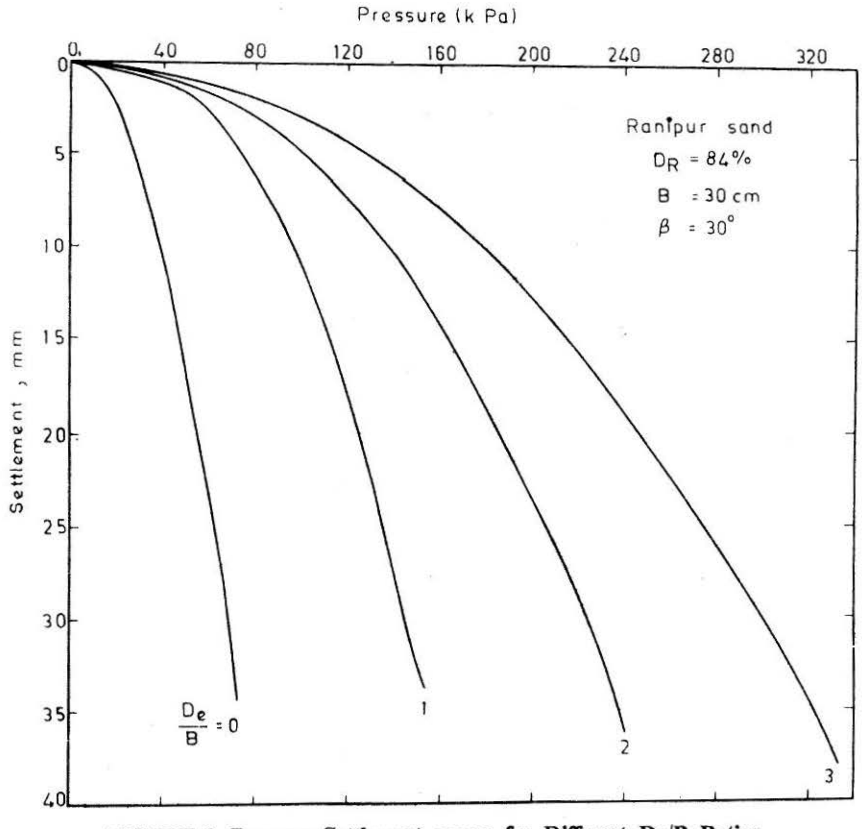

FIGURE 9 Pressure Settlement curves for Different De/B Ratios

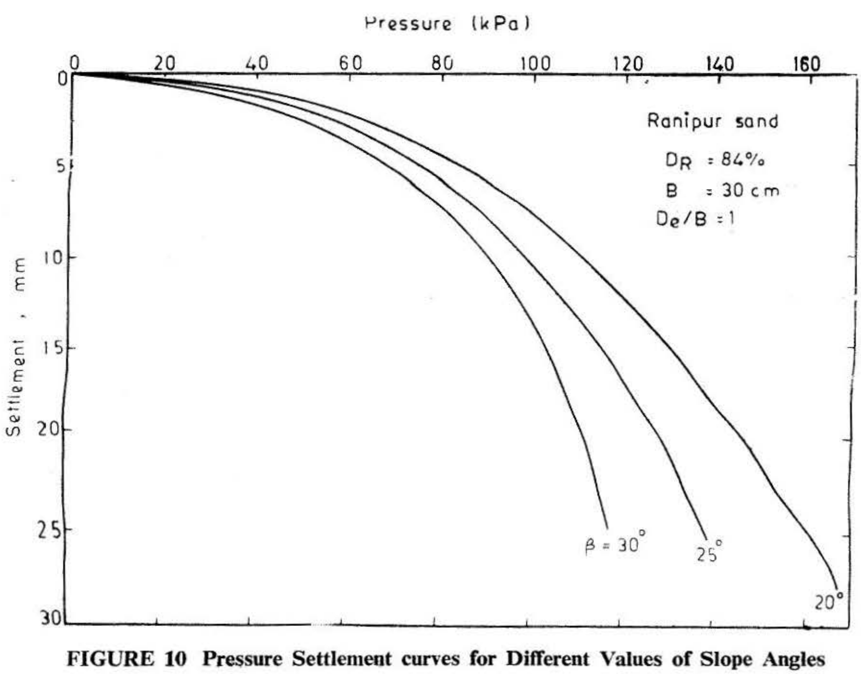

FIGURE 10 Pressure Settlement curves for Different Values of Slope Angles

27

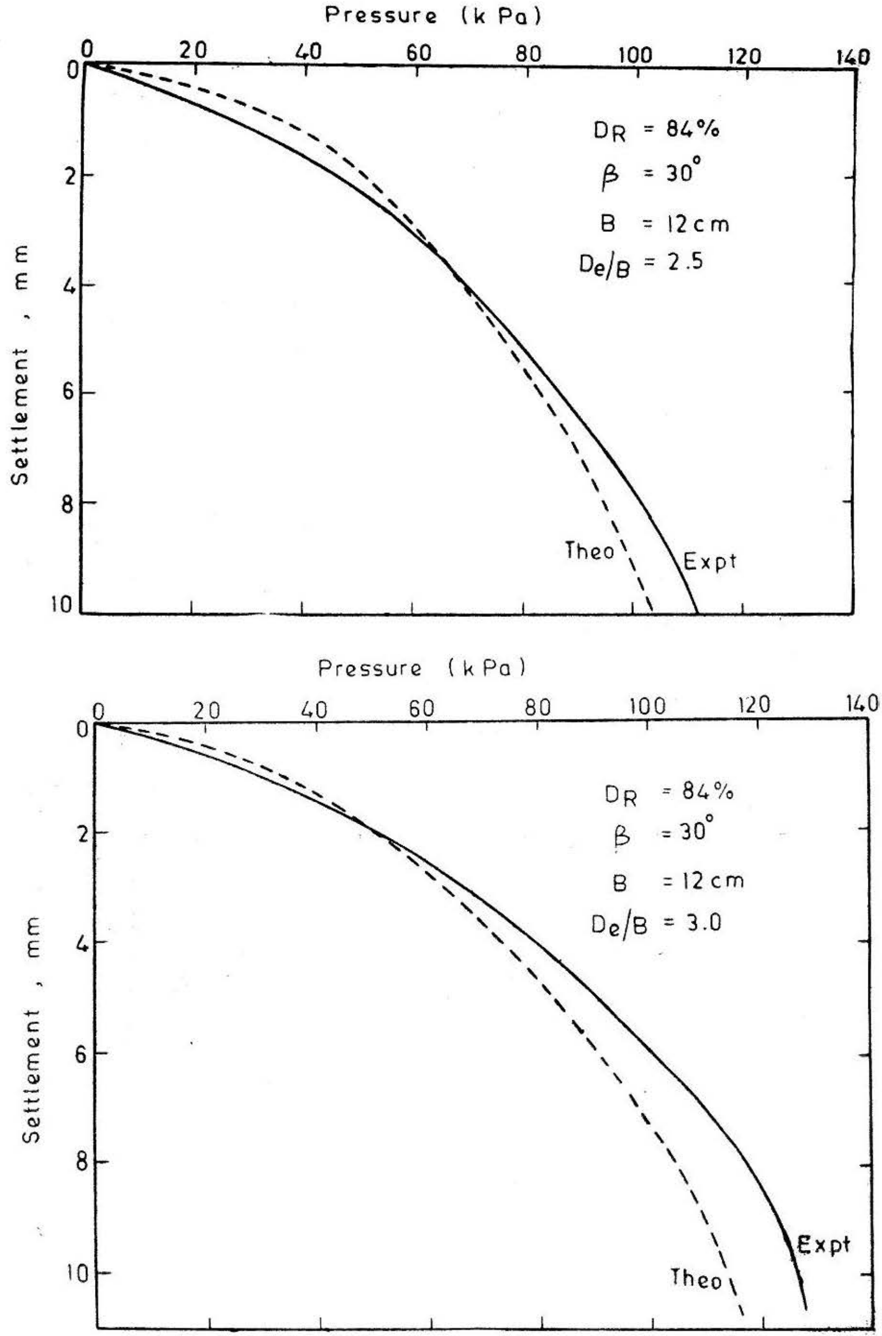

FIGURE 11 Comparison of Computed and Experimental Pressure Settlement Curves

28

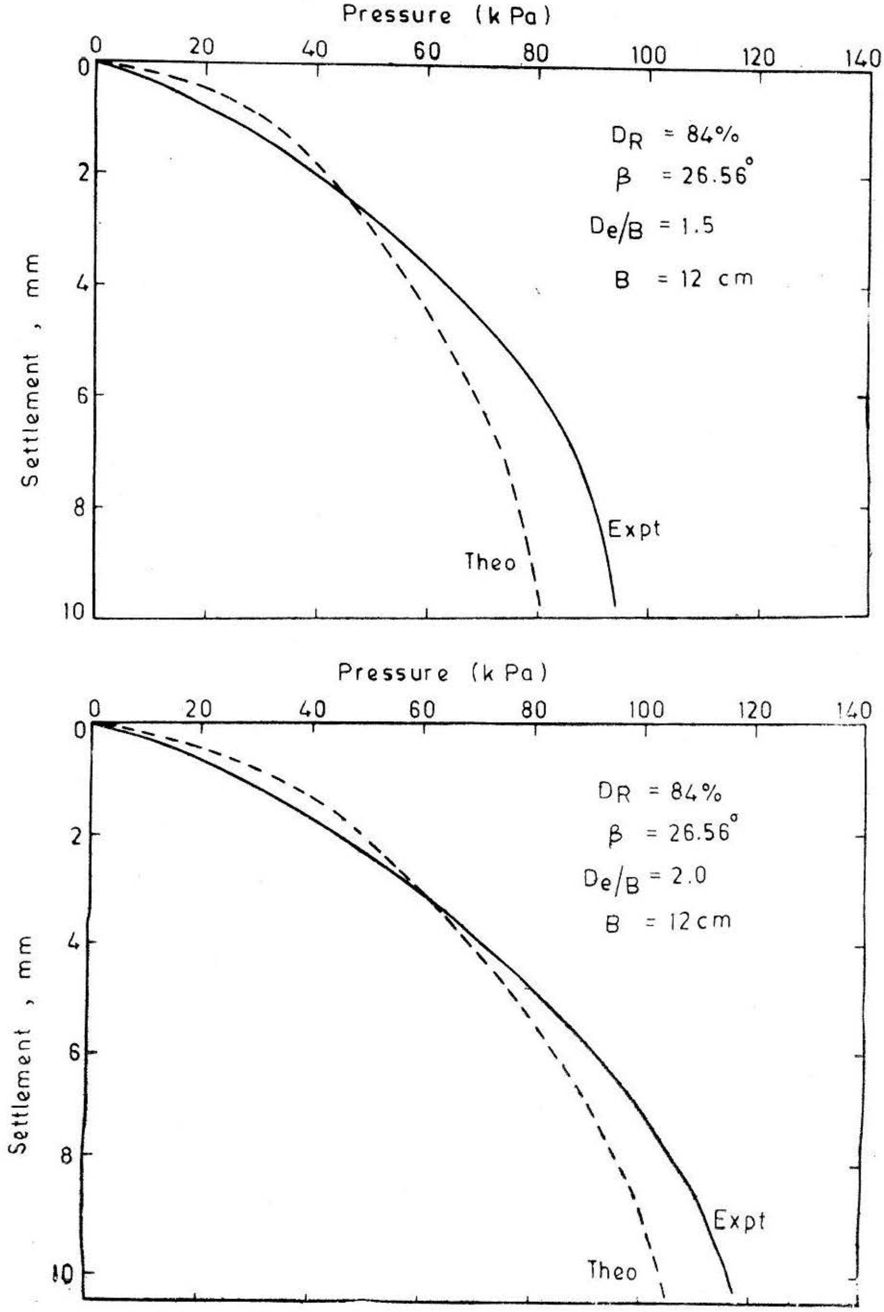

FIGURE 12 Comparison of Computed and Experimental Pressure Settlement Curves

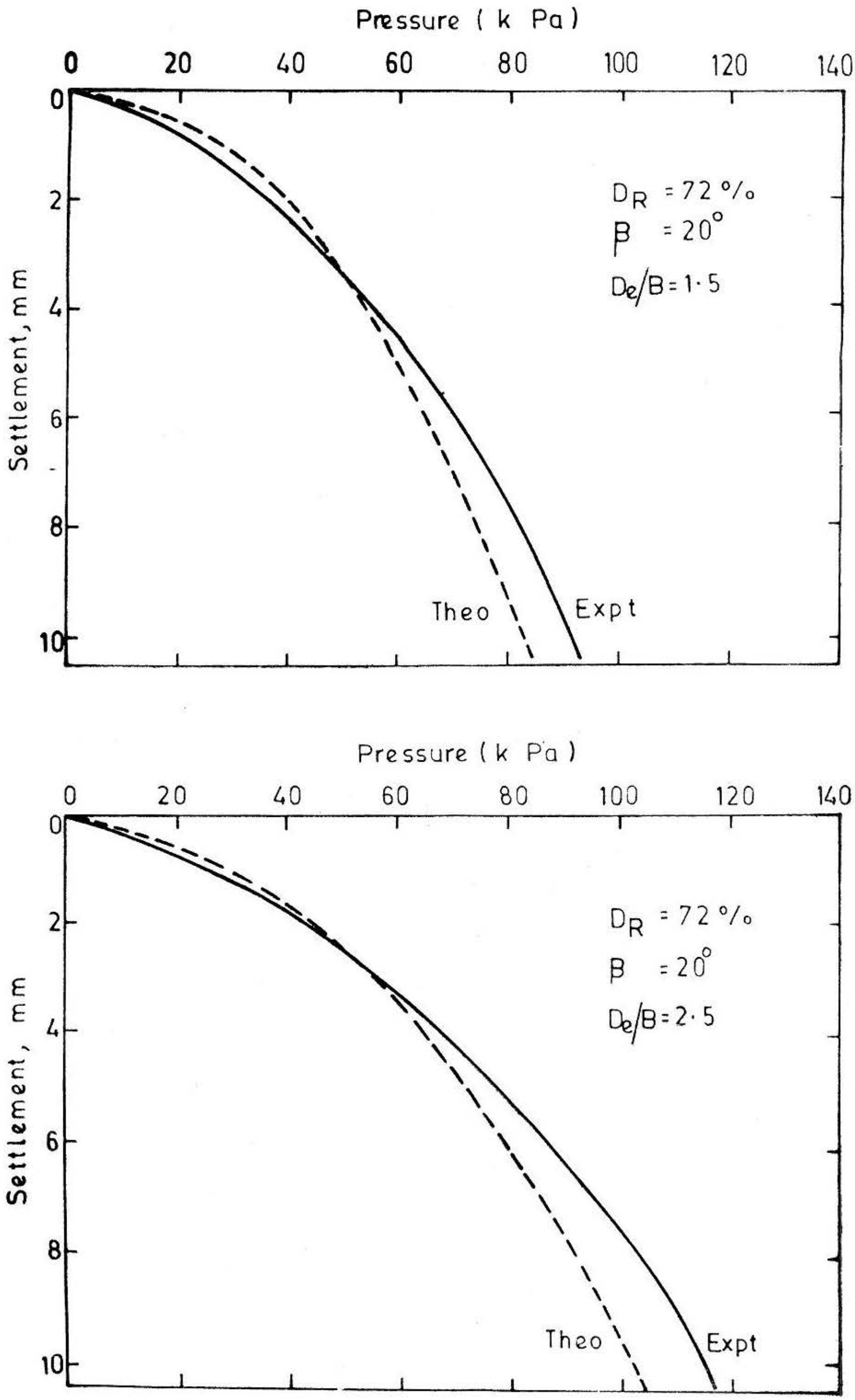

FIGURE 13 Comparison of Computed and Experimental Settlement Curves

Conclusions

A semi-empirical procedure is developed to predict the settlement characteristics of actual footings resting on c—ϕ soils adjacent to slopes using non-linear constitutive laws of soils. Triaxial tests were used to establish the non-linear constitutive laws of sands. The settlements obtained by the proposed procedure have been compared with the model test data and found to be in good agreement.

References

Bowles, E.J., (1977), "Foundation Analysis and Design" McGraw Hill Kogakusha Ltd., : 131-133.

CORNFORTH, D.H., (1964), "Some Experiments on the influence of strain conditions on the strength of sands," *Geotechnique*, Vol. 14, No. 2 : 143-167.

KONDNER, R.L. and ZELASKO, S., (1963), "A Hyperbolic Stress-Strain Formulation for Sands," *Procs. of 2nd pan American Conf. on Soil Mech. and Foundation Engg.*, Brazil, Vol I, : 289-324.

KUSAKABE, O., et al., (1981), "Bearing Capacity of Slopes under Strip Loads on the Top Surfaces", *Japanese Society of Soil Mech. and Foundation Engg.*, 21 : 4.

MEYERHOF, G.G., (1957), "The Ultimate Bearing Capacity of Foundations on Slopes"; *Proc. 4th Int. Conf. on soil Mech. and Foundation Engg.*, Vol. I; : 384-386.

MIZUNO, T., YASHIHARU, T. and HIROSHI, K., (1960), "On the Bearing Capacity of a Slope on Cohesionless Soils", Japanese Society of Soil Mech. and Eoundation Engg., Vol I, No, 2 : 30-37.

SHARAN, U.N., (1977), "Pressure Settlement Characteristics of Surface Footing from constitutive laws', *Ph.D. Thesis*, University of Roorkee, Roorkee.

SOKOLOVSKI, V.V., (1960), "Statics of Granular Media", 2nd Ed., Butter worths Scientific Publication, London.

SUD, V.K. (1984), "Behaviour of Shallow Foundations Adjacent to Slopes", *PhD. Thesis*, University of Roorkee ,Roorkee.

Notations

Symbols		Description	Unit
A_1, A_2	=	Constants	
a	=	Hyperbolic constant	
B	=	Width of footing	cm, m
b	=	Hyperbolic constant	
c	=	Unit cohesion	kN/m^2
c_a	=	Adhesion between the side wall and soil	kN/m^2

c_{mi}	= Mobilized cohesion at i^{th} strip	kN/m^2
D_e	= Distance of the edge of the foundation from the slope shoulder	m
D_f	= Depth of foundation	m
E	= Elastic modulus	kN/m^2
E_{pi}	= Passive resistance at the i^{th} strip	kN
F	= A factor	
H	= Depth of soil mass under the footing at which stresses become zero	m
H_s	= Height of slope	m
K_o	= Coefficient of earth pressure at rest	
K_1, K_2	= Constants	
$M_\gamma\ (i)$	= Moment of resistance due to weight only c considering the height of face equal to $[i(\Delta H)+D_f]$	$kN\text{-}m$
$M_\gamma\ (i\text{-}1)$	= Moment of resistance due to weight only considering the height of face equal to $[(i\text{-}1)(\Delta H)+D_f]$	$kN\text{-}m$
$M_c(i)$	= Moment of resistance due to cohesion only considering the height of face equal to $[i(\Delta H)+D_f]$	$kN\text{-}m$
$M_c(i\text{-}1)$	= Moment of resistance due to cohesion only considering the height of face equal to $[(i-1)\Delta H+D_f]$	$kN\text{-}m$
q	= Load intensity	kPa
q_f	= Friction force	N
q_u	= Ultimate bearing pressure	kPa
q_v	= Vertical stress intensity on any strip	kPa
R_o	= Initial radius of log spiral	m
S	= Total settlement	mm
β	= Angle which the slope makes with the horizontal	
γ	= Unit weight of soil	kN/m^3
δ	= Angle of wall friction	
ϕ	= Angle of shearing resistance	
$\phi_m i$	= Mobilized angle of shearing resistance at i^{th} strip	
σ_u	= Ultimate stress	kPa
σ_x	= Stress in x-direction	kPa
σ_z	= Stress in z-direction	kPa
τ_{xz}	= Shear stress	kPa

σ_1	=	Major principal stress	kPa
σ_2	=	Intermediate principal stress	kPa
σ_3	=	Minor principal stress	kPa
θ	=	Log spiral angle on the slope side	
θ_1	=	Inclination of major principal stress w.r.t. vertical axis	
θ_3	=	Inclination of minor principal stress w.r.t. vertical axis	
ϵ_{zi}	=	Strain in the vertical direction at i^{th} strip	
ϵ_1	=	Major principal strain	
ϵ_2	=	Intermediate principal strain	
ϵ_3	=	Minor principal strain	