Uplift Capacity of Circular and Strip Anchors

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Introduction

Estimation of uplift capacity of anchors has attracted considerable research in recent years. Depending upon the depth of embedment of the plate in the soil, the plate anchors are classified as shallow or deep anchors. In the case of shallow anchors the failure surface reaches the ground surface and a small increase in embedment depth results in a considerable increase in breakout load whereas in deep anchors, due to limiting settlement consideration, the failure surface does not reach the ground. A number of theoretical analyses and model tests, both conventional and centrifugal, have been presented (Balla, 1961; Matsuo, 1967 and 1968; Meyerhof and Adams, 1968; Vesic, 1971; Murray and Geddes, 1987; Rowe and Davis, 1982a and b; Saeedy, 1987; Subba Rao and Jyant Kumar, 1994; Ghaly and Hanna, 1994, etc.) for homogeneous soils. Balla (1961), Matsuo (1967 and 1968), Saeedy (1987) and Ghaly and Hanna (1994) use Kotter's equation for pre-fixed failure surfaces to obtain shear stresses on the failure surface. The theory of Meyerhof and Adams (1968) is based on Caquot and Kerisel's earth pressure coefficients (Kerisel and Absi, 1990) for a curved rupture surface coupled with an assumption of mobilised friction angle on the cylindrical surface. The theory of Vesic (1971) is based on cavity expansion. Rowe and Davis (1982a and b) have used a FEM approach in obtaining the uplift capacity. Murray and Geddes (1987) follow a limit analysis approach. Nayak et al. (2003) proposed a method for the determination of uplift capacity of horizontal strip anchors in layered sands based on an assumed distribution of interslice friction angle across the slices and satisfying the equilibrium conditions.

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Meyerhof and Adams (1968) and Dickin (1988) have suggested empirical shape factors to obtain uplift capacity of circular anchors from the results of strip anchors. Model tests conducted by Das and Seeléy (1975) and Frydman and Shaham (1989) have led to varying values of shape factors.

Subba Rao and Jyant Kumar (1994) proposed a theory in terms of uplift capacity factors for shallow strip anchors by using the method of characteristics coupled with a log-spiral failure surface in the lower region and a Rankine passive zone at the ground surface. A modified theory is presented in the present paper for both circular and strip anchors without the Rankine passive zone at the ground surface. The log-spiral failure surface is assumed for the entire depth of embedment. This change was needed in the light of the earlier theory (Subba Rao and Jyant Kumar, 1994) over predicting the extent of failure zone at the ground surface. The critical failure surface is obtained by satisfying vertical equilibrium as before but a new methodology in search of the focus for the critical failure surface has been adopted. The results are expressed in terms of uplift capacity factors F_c, F_q and F_g. The values of shape factors for cohesion, surcharge and unit weight have also been presented. Finally, the predictions from the theory have been compared with the available results.

Method and Analysis

Circular Anchor

Figure 1a is the schematic diagram showing shallow horizontal circular anchor of diameter B subjected to vertical uplift in homogeneous soil. The following assumptions have been made in the analysis.

(i) Anchor plate is smooth.

(ii) Anchor tie rod has no influence on failure load or on pattern of failure.

(iii) Soil is at failure at each and every point on the failure surface and follows Mohr- Coulomb failure criterion.

(iv) The failure surface from the edge of the anchor plate is an arc of log-spiral and its tangent at the ground surface is inclined at an angle of $45° - \phi/2$ with the horizontal (Fig.1a).

(v) The suction force under the base of the anchor is ignored.

A system of co-ordinate axes with the center of the anchor plate as the origin is shown in Fig.1a. Let L be the distance between the point G on the edge of the anchor plate and the focus F of the log-spiral, which lies on the

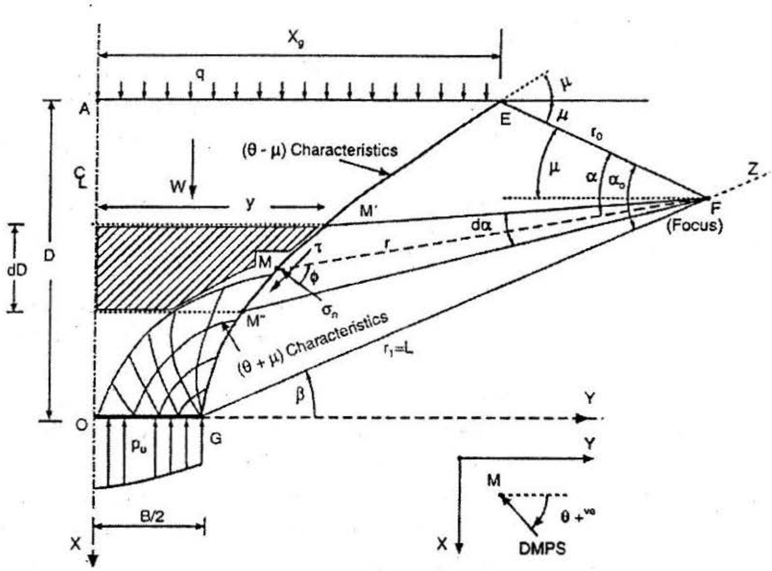

FIGURE 1a : Failure Pattern and System of Forces for Circular Anchor

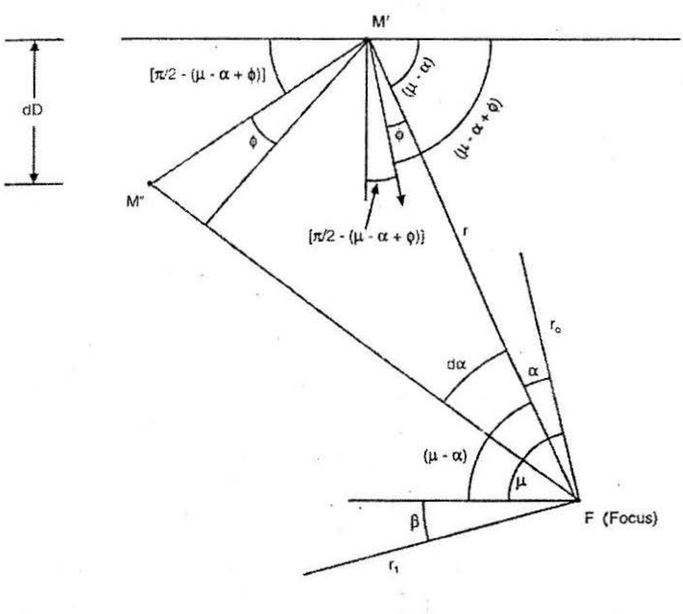

FIGURE 1b : Magnified Segmental Element M'M"

line GZ which is inclined at an angle β with the horizontal. To start with, for a trial value of β, it is assumed that the failure surface starting from the edge G of the anchor plate meets at an angle of $45° - \phi/2$ with the horizontal at the ground surface. This condition can be established by the following steps. From Fig.1a,

$$D = r_1 \sin\beta + r_0 \sin\mu \tag{1}$$

But,

$$\alpha_o = (\mu+\beta) \text{ and } r_1 = r_0\, e^{(\alpha_0 \tan\phi)}$$

hence,

$$r_1 = \frac{D}{\sin\beta + \dfrac{\sin\mu}{e^{(\alpha_0 \tan\phi)}}} \tag{2}$$

As the ground surface is horizontal and the surcharge is uniform, the direction of major principal stress (DMPS) is horizontal at point E. Hence the state of stress is known at point E. Referring to Fig.1a, θ is the angle which the direction of major principal stress makes with the horizontal Y-axis and clockwise direction is taken as positive. From the definition of characteristics, the failure surface GE will become one of $(\theta - \mu)$ characteristics and, accordingly, by making use of equations valid along $(\theta - \mu)$ characteristics, the state of stress at any point along the failure surface is determined as below.

For any point M on the failure surface (Fig.2), let a be the angle which the radius vector to this point makes with the initial radius vector EF. It can be seen that co-ordinates of point M(x, y) are:

$$x = -\left[D - r_0 \sin\mu - r\sin(\alpha-\mu)\right] \tag{3}$$

$$y = \frac{B}{2} + r_1 \cos(\alpha_0 - \mu) - r\cos(\alpha-\mu) \tag{4}$$

The value of θ at point M (Fig.2), can be obtained from the expression,

$$\theta = -\left[\frac{\pi}{2} - (\phi - \alpha + 2\mu)\right] \tag{5}$$

The equations valid along $(\theta - \mu)$ characteristics can be used to determine the state of stress at point M. Along $(\theta - \mu)$ characteristics,

$$\frac{dx}{dy} = \tan(\theta-\mu) \tag{6a}$$

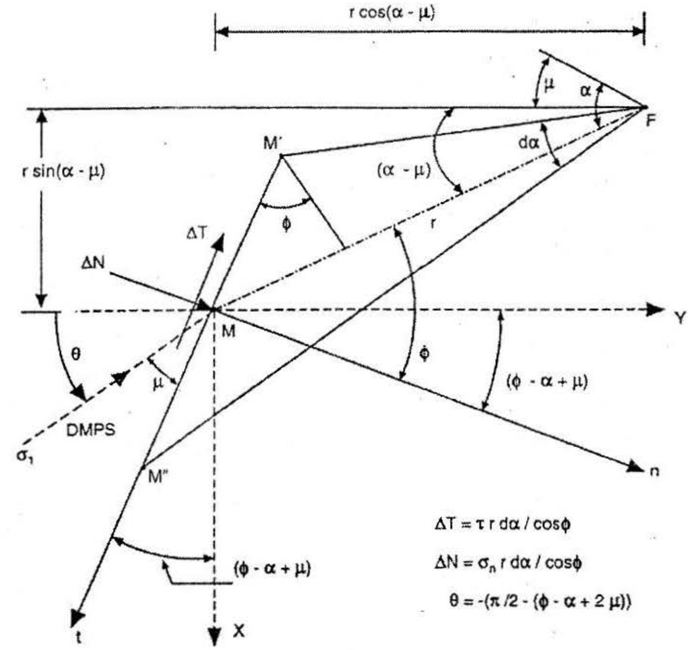

FIGURE 2 : Forces Acting over an Element M′M″ on Curved Failure Surface for Circular Anchor

and

$$d\sigma - 2\sigma \tan\phi d\theta - \gamma(dx - \tan\phi dy) + \frac{\sigma}{y}\tan\phi\left[\cos\phi dy - (1-\sin\phi)dx\right] = 0 \tag{6b}$$

Expressing the above equations in finite difference form, the state of stress at any point along EG (Fig.1a) is obtained. Again, starting from the edge G of the anchor plate which is a singular point, and making use of the condition that shear stress along GO is zero, the base pressure distribution along the anchor plate is obtained. In the analysis the equations used along $(\theta + \mu)$ characteristics are,

$$\frac{dx}{dy} = \tan(\theta + \mu) \tag{6c}$$

and

$$d\sigma + 2\sigma \tan\phi d\theta - \gamma(dx + \tan\phi dy) + \frac{\sigma}{y}\tan\phi\left[\cos\phi dy + (1-\sin\phi)dx\right] = 0 \tag{6d}$$

The above analysis has been carried out for a number of failure surfaces by varying the angle β (Fig.1a). It may be mentioned here that fixed boundaries at both top and bottom have been considered in the analysis of Balla (1961), Matsuo (1967 and 1968), Khadilkar et al. (1971), Saeedy (1987) and Ghaly and Hanna (1994). While the top boundary of the failure surface was taken at $(45^\circ - \phi/2)$ to the ground surface, various bottom boundary surfaces were considered by various authors. The analysis of Meyerhof and Adams (1968) is different from these and considers passive earth pressure on a vertical surface. The analysis that is pursued in the present paper has the top boundary surface fixed at $(45^\circ - \phi/2)$ to the ground surface, but the bottom boundary surface varies.

By taking into account the axisymmetry of the problem the overall vertical equilibrium of the soil mass is examined to establish the correct failure surface. For all except the correct failure surface this vertical equilibrium condition will not be satisfied. For the correct failure surface,

$$P_u = \left(Q + V_s + W\right) \tag{7}$$

in which, P_u = total ultimate vertical uplift load

$Q = q\pi X_g^2$

where, q = surcharge pressure

X_g = extent of failure at ground from the center line of the anchor plate, i.e.,

$$X_g = \frac{B}{2} + r_1 \cos\beta - r_0 \cos\mu \tag{7a}$$

V_s = total vertical downward component of resultant force on the failure surface EG

W = weight of breaking out soil mass bounded by the failure surface GE all along the circumference of the circular anchor, which is given by:

$W = V\gamma$

in which, γ = unit weight of soil mass, and

V = volume of breaking out soil mass and it can be calculated by integrating an elemental circular area of radius of revolution, y, (Figs.1a and 1b) on the total height, D, of the log-spiral.

$$V = \int_0^D \pi y^2 dD \tag{8}$$

From Figs.1a and 1b,

$$Y = \left[\frac{B}{2} + r_1 \cos(\alpha_0 - \mu) - r\cos(\alpha - \mu)\right]$$

and

$$dD = \frac{r_0 e^{(\alpha \tan\phi)} d\alpha}{\cos\phi} \sin\left[\left(\frac{\pi}{4} - \frac{\phi}{2}\right) + \alpha\right]$$

The state of stress at any point is obtained in terms of four variables x, y, σ and θ. Here x and y are the co-ordinates of any point considered. σ = stress difference between the center of Mohr circle and the point where Mohr-Coulomb's failure envelope meets the normal stress axis.

Calculation of P_u and V_s

P_u is determined by integrating numerically the vertical stresses acting at various points along the anchor plate OG and it is given by,

$$P_u = \sum_{i=1}^{n} \left(\sigma_{x_{i-1}} y_{i-1} + \sigma_{x_i} y_i\right) \pi \left(y_{i-1} - y_i\right) \tag{9}$$

where, n = no. of points considered along the anchor plate up to the point O .

σ_x = vertical normal stress at any point considered along the anchor plate

y = horizontal coordinate of the point considered

The total vertical downward component of resultant force on the failure surface EG is given by

$$V_s = \int_{\alpha=0}^{\alpha_0} \Delta V_s \tag{10}$$

In which ΔV_s is the vertical component of resultant force due to shear and normal stresses acting on the circumferential area of the element M′M″ (Fig.2). Also,

$$\Delta V_s = \left[\Delta T \cos(\phi - \alpha + \mu) - \Delta N \sin(\phi - \alpha + \mu)\right] 2\pi y_m \tag{11}$$

here y_m = y coordinate of point M,

ΔT = shear force on the circumferential area of the element M′M″

$$= \tau \cdot r \cdot \frac{d\alpha}{\cos\phi} \tag{12}$$

ΔN = normal force on the circumferential area of the element M′M″

$$= \sigma_n \cdot r \cdot \frac{d\alpha}{\cos\phi} \tag{13}$$

where shear stress τ and normal stress σ_n are given by:

$$\tau = -\sigma \sin\phi \left[\sin 2\{\theta - (\phi - \alpha + \mu)\}\right] \tag{14}$$

$$\sigma_n = \sigma\left[1 + \sin\phi \cos 2\{\theta - (\phi - \alpha + \mu)\}\right] - c\cot\phi \tag{15}$$

in which, c and ϕ = shear parameters for the soil

σ and θ = appropriate values corresponding to the point M, and

$$r = r_0\, e^{(\alpha \tan\phi)} \tag{16}$$

By substituting the values of various variables from the above equations value of V_s, (as per Eqn.10) has been calculated by numerical integration.

Uplift Capacity Equation

The average ultimate uplift pressure, p_u, is expressed as,

$$p_u = \frac{4 P_u}{\pi B^2} \tag{17}$$

where, P_u = ultimate uplift load.

Writing in a manner somewhat similar to the conventional bearing capacity equation of foundation under compression, net average ultimate uplift capacity, $p_{u,\,net}$, is written as,

$$p_{u,net} = p_u - \gamma D = c F_c + q F_q + 0.5 \gamma B F_\gamma \tag{18}$$

where, D = embedment depth

γ = unit weight of soil

B = diameter of the anchor

F_c, F_q and F_g are the uplift capacity factors corresponding to cohesion, surcharge and unit weight, respectively, and they are functions of ϕ and embedment depth ratio λ or D/B ratio (i.e., λ = D/B).

In the bearing capacity problem, both the overburden pressure and the foundation pressure are compressive in nature and so the standard bearing capacity equation stands in terms of gross pressure. In the uplift capacity problem, the overburden pressure being compressive, the standard uplift capacity equation has been expressed in terms of net uplift pressure. However it is to be noted that net and gross pressures are easily convertible.

The uplift capacity factors have been determined separately as explained below.

$$F_c = \frac{P_{u,cq\gamma} - P_{u,q\gamma}}{c} \tag{18a}$$

$$F_q = \frac{P_{u,cq\gamma} - P_{u,c\gamma}}{q} \tag{18b}$$

$$F_\gamma = \frac{P_{u,q\gamma} - q F_q}{0.5 \gamma B} \tag{18c}$$

where, $p_{u,cq\gamma}$ = net average ultimate uplift capacity due to cohesion, surcharge and unit weight for the given values of ϕ and λ.

$p_{u,q\gamma}$ = net average ultimate uplift capacity due to surcharge and unit weight without considering cohesion i.e., c = 0, for the same values of ϕ and λ.

$p_{u,c\gamma}$ = net average ultimate uplift capacity due to cohesion and unit weight without considering surcharge i.e., q = 0, for the same values of ϕ and λ.

Using the Eqns.18a, 18b and 18c, the uplift capacity factors F_c, F_q and

F_γ are obtained by actually running the program. These equations have been developed from the Eqn.18 by considering different cases for the same values of ϕ and λ, i.e.,

Case (i) : $c \neq 0$, $q \neq 0$ and $\gamma \neq 0$

Case (ii) : $c = 0$, $q \neq 0$ and $\gamma \neq 0$

Case (iii) : $c \neq 0$, $q = 0$ and $\gamma \neq 0$

Strip Anchor

Consider a strip anchor having a width B and length L being very large, theoretically infinite. Practically, if L/B exceeds about 8 (Dickin, 1988) then the anchor behaves like a strip anchor. The failure surfaces can be considered in the same manner as shown in Fig.1a with the only difference that B now stands for width of the strip anchor. The assumptions and the procedure are the same as those used for circular anchor. Using the following expressions valid along the characteristics (Sokolovski, 1960) the base pressure distribution along the strip anchor plate for the statically correct failure surface is obtained.

Along $(\theta + \mu)$ characteristics,

$$\frac{dx}{dy} = \tan(\theta + \mu) \text{ and } \frac{d\xi}{dy} = b \tag{19a}$$

Along $(\theta - \mu)$ characteristics,

$$\frac{dx}{dy} = \tan(\theta - \mu) \text{ and } \frac{d\eta}{dy} = a \tag{19b}$$

in which, $\xi = \chi + \theta,$

$$\eta = \chi - \theta$$

$$\chi = \frac{\cot\phi}{2}\ln\frac{\sigma}{\sigma_0}$$

σ_0 = characteristic stress,

$$\mu = \pi/4 - \phi/2 .$$

Further,

$$a = \frac{d\eta}{dy} = \frac{d\left(\frac{\cot\phi}{2}\ln\frac{\sigma}{\sigma_0} - \theta\right)}{dy} = \frac{-\gamma\cos(\theta+\mu)}{2\sigma\sin\phi\cos(\theta-\mu)} \tag{20a}$$

$$b = \frac{d\xi}{dy} = \frac{d\left(\frac{\cot\phi}{2}\ln\frac{\sigma}{\sigma_0} + \theta\right)}{dy} = \frac{\gamma\cos(\theta-\mu)}{2\sigma\sin\phi\cos(\theta+\mu)} \tag{20b}$$

ξ, η, χ, a and b are the standard variables used in Sokolovski's (1960) approach for two dimensional analysis in the method of characteristics. The average net ultimate uplift pressure for strip anchor is expressed in terms of uplift capacity factors F_c, F_q and F_γ in the same way as expressed for circular anchor and the results are obtained for different values of ϕ and λ. The expression for extent of failure zone at ground surface is the same as that for circular anchor as given in Eqn.7a.

Determination of Ultimate Uplift Capacity of Circular Anchor using Shape Factors

The average ultimate uplift pressure p_u for circular anchor from the present theories is expressed as

$$p_{u,cir} = c\,F_{c,cir} + q\,F_{q,cir} + \left[0.5\gamma B F_{\gamma,cir} + \gamma D\right] \tag{21}$$

where, $F_{c,cir}$, $F_{q,cir}$ and $F_{\gamma,cir}$ are the uplift capacity factors for circular anchor for given ϕ and λ. The above equation can also be expressed using shape factors as

$$p_{u,cir} = S_{uc}cF_{c,str} + S_{uq}qF_{q,str} + S_{u\gamma}\left[0.5\gamma B F_{\gamma,str} + \gamma D\right] \tag{22}$$

where, $F_{c,str}$, $F_{q,str}$ and $F_{\gamma,str}$ are the uplift capacity factors for strip anchor with the same ϕ and λ. B is the width of strip anchor which is the same as the diameter of circular anchor. S_{uc}, S_{uq} and $S_{u\gamma}$ are shape factors for cohesion, surcharge and unit weight, respectively, for the circular anchor. These shape factors are also readily seen to be the ratios of the respective breakout factors for cohesion, surcharge and unit weight. The overall breakout factor is defined as $P_u/\gamma AD$. Here A is the area of anchor plate.

S_{uc} is the ratio of breakout factors of circular anchor ($N_{uc,cir}$) and strip anchor ($N_{uc,str}$) for cohesion for the same ϕ and λ.

Hence

$$S_{uc} = \frac{N_{uc,cir}}{N_{uc,str}} = \frac{F_{c,cir}}{F_{c,str}} \tag{23}$$

Similarly S_{uq} which is the ratio of breakout factors of circular anchor ($N_{uq,cir}$) and strip anchor ($N_{uq,str}$) for surcharge for the same ϕ and λ is expressed as

$$S_{uq} = \frac{N_{uq,cir}}{N_{uq,str}} = \frac{F_{q,cir}}{F_{q,str}} \tag{24}$$

$S_{u\gamma}$ is the ratio of breakout factors of circular anchor ($N_{u\gamma,cir}$) and strip anchor ($N_{u\gamma,str}$) for unit weight for the same ϕ and λ.

Hence

$$S_{u\gamma} = \frac{N_{u\gamma,cir}}{N_{u\gamma,str}} = \frac{\left[0.5F_{\gamma,cir} + \lambda\right]}{\left[0.5F_{\gamma,str} + \lambda\right]} \tag{25}$$

Results and Discussion

The results obtained from the present theory are compared with the available experimental and theoretical results. Figure 3 shows the variation of

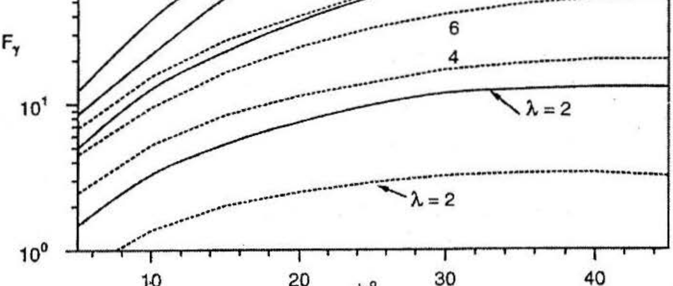

FIGURE 3 : Variation of Uplift Capacity Factor, F_c

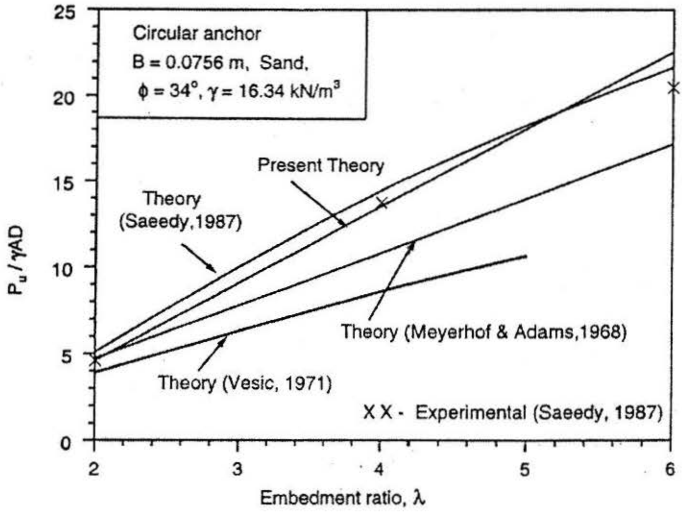

FIGURE 4 : Variation of Uplift Capacity Factor, F_q

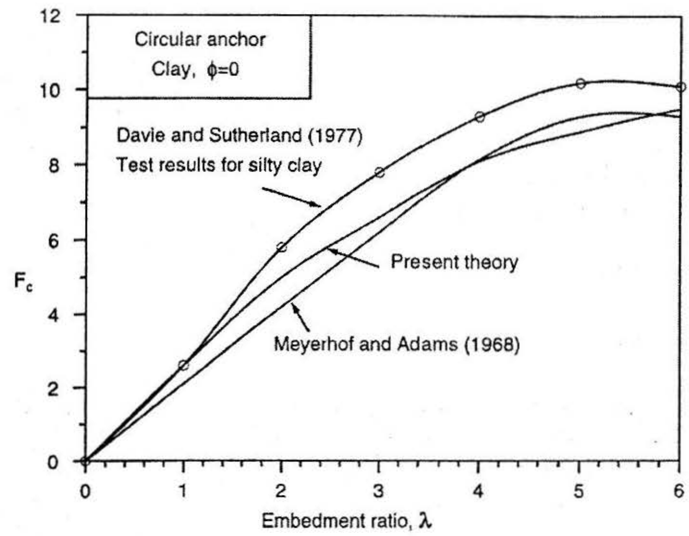

FIGURE 5 : Variation of Uplift Capacity Factor, F_γ

uplift capacity factor for cohesion F_c as a function of ϕ and λ for both circular and strip anchors. The increase in F_c for circular anchor is more than the corresponding value for strip anchor for an increase in ϕ or λ. It is also observed that for any value of λ, the ratio of F_c for circular anchor to the corresponding F_c for strip anchor is increasing as ϕ increases. Figures 4

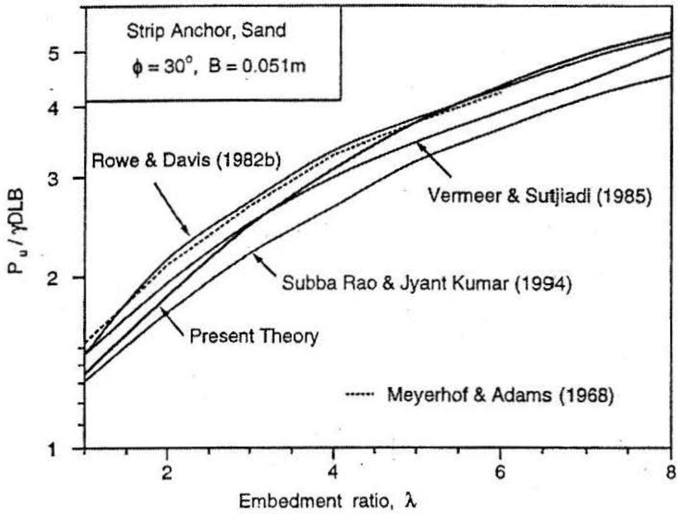

FIGURE 6 : Comparison of Results for Circular Anchor in Sand

and 5 show the variation of uplift capacity factors for surcharge and unit weight, F_q and F_g respectively, as functions of ϕ and λ for both circular and strip anchors. The nature of variation of uplift capacity factors F_q and F_γ is seen to be similar to that of F_c. As of now for a general $(c - \phi)$ soil the information on critical embedment ratio (λ_{cr}) or D/B ratio is not available and hence values of uplift capacity factors have been shown up to $\lambda_{cr} = 8$. Critical embedment ratio (λ_{cr}) is that embedment ratio up to which anchor behaves as shallow.

Circular Anchor

Figure 6 shows the comparisons of different theories with experimental results of Saeedy (1987) for circular anchors in Ottawa sand. The present theory compares reasonably well with the experimental results. The theory of Vesic (1971) gives lower values of P_u, for all values of when compared to the presented theoretical and experimental values. The breakout factor, $P_u/\gamma AD$, values predicted by this study are compared with those from other theories in Table 1 and shows the closeness to the mean values performed on two types of soils (ϕ = 25° and ϕ = 45°). This table is specially constructed to illustrate the range of variation for a wide variation in ϕ and for a typical λ. The present theoretical results compare favourably with the other theoretical and experimental results as indicated in Tables 2 and 3 for circular anchor in sand with varying parameters.

Comparison with experimental results of Davie and Sutherland (1977)

TABLE 1 : Comparison of Breakout Factor ($P_u/\gamma AD$) from various Theories for Circular Anchor in Sand

$\lambda = 3, \gamma = 16.3$ kN/m^3, B = 0.0756 m	($P_u/\gamma AD$)							
ϕ in Deg.	Balla (1961)	Mariupol' skii (1965)	Vesic (1965)	Matsuo (1967)	Meyerhof & Adams (1968)	Saeedy (1987)	Mean Values	Present Theory
25	8.0	3.3	4.3	9.2	3.3	4.0	5.3	5.6
45	10.3	9.1	7.0	15.0	14.3	12.4	11.3	9.2

TABLE 2 : Comparison of Field Tests and Theoretical Results for Circular Anchor in Sand

$\gamma' = 10.37$ (kN/m^3)	Embedment Ratio (λ)	Dia. of Anchor, B (m)	$P_u/\gamma AD$ from field test Sutherland et al. (1982)	Theoretical values $P_u/\gamma AD$				
ϕ in Deg.				Balla (1961)	Vesic (1965)	Meyerhof & Adams (1968)	Saeedy (1987)	Present Theory
42	1.91	2.39	7.55	6.26	4.51	6.94	8.39	5.46
42	2.17	2.39	9.35	7.20	5.0	8.16	9.59	7.94
35	2.17	2.95	4.50	6.10	4.1	5.44	5.4	5.1
35	2.94	2.39	7.84	10.5	7.6	7.80	8.1	8.1

TABLE 3 : Comparison of Results for Circular Anchor in Sand

$\phi = 42°, \gamma = 17.9$ kN/m^3		$P_u / \gamma AD$				
Embedment Ratio, λ	Dia. of Anchor, B (m)	Exptl., Baker & Kondner (1966)	Vesic's Theory (1965)	Meyerhof & Adams (1968)	Saeedy's Theory (1987)	Present Theory
7	0.0756	40.2	23.9	47.9	41.7	42.1
6	0.0756	33.2	18.4	37.8	32.4	32.5
5	0.0756	24.8	15.3	28.4	25.9	24.0
7.5	0.0504	45.8	27.2	55.2	47.4	45.5
6	0.0504	32.6	18.0	38.3	32.0	31.5
8	0.0378	50.9	28.2	62.0	54.3	52.7

TABLE 4 : Comparison of Results for Circular Anchor in $c - \phi$ Soil

$\phi = 34°, \gamma = 17$ kN/m^3, c =4.4 kN/m^2		$P_u / \gamma AD$		
Embedment Ratio, λ	Dia. of Anchor, B (m)	Experimental Khadilkar et al. (1971)	Meyerhof & Adams' Theory (1968)	Present Theory
3	0.1016	21.5	17.6	26.1
4	0.0762	31.4	24.1	41.5
6	0.0508	59.0	40.0	76.3
6	0.0762	61.2	32.8	59.3

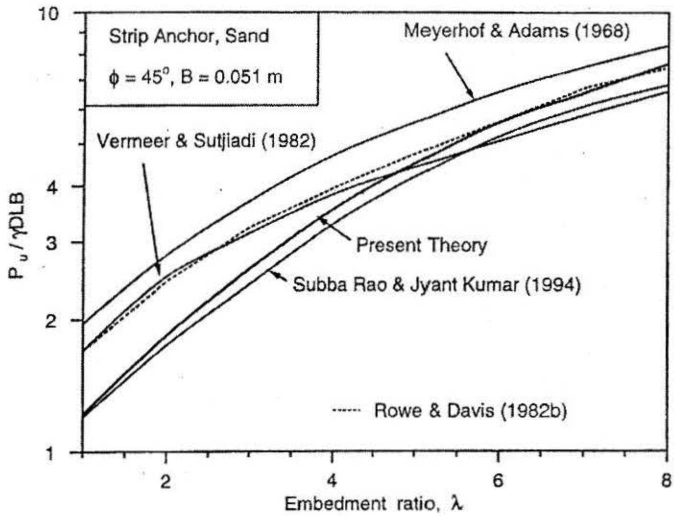

FIGURE 7 : Comparison of F_c for Circular Anchor in Clay

for $\phi = 0$ case is shown in Fig.7. Experimental results were only up to $\lambda = 6$. Uplift capacity factor F_c obtained from the present theory, for $\phi = 0$, is seen to compare well with the experimental data. The theory of Meyerhof and Adams (1968) is seen to underestimate the uplift capacities. Table 4 shows the comparison of present theoretical results with the experimental results of Khadilkar et al. (1971), for circular anchor in $(c - \phi)$ soil. The theory of Meyerhof and Adams (1968) gives lower values of P_u when compared to experimental (Khadilkar et al., 1971) results.

Strip Anchor

The results from the present theory for strip anchor have been compared with other available theories. Figures 8 and 9 show the comparisons by different theories for two different values of ϕ, one representing loose sand and the other representing dense sand. For this purpose the theories of Meyerhof and Adams (1968), Rowe and Davis (1982 b), Vermeer and Sutjiadi (1985) and Subba Rao and Jyant Kumar (1994) have been chosen and shown in Figs.8 and 9. For Loose sand ($\phi = 30°$) from Fig.8, it is observed that the present theory compares reasonably well with the other theories. For dense sand ($\phi = 45°$) the predictions from the present theory match well with the other theories for higher embedment ratios (Fig.9). For lower embedment ratios, the pullout estimates from the present theory are conservative when compared to the theories other than theory of Subba Rao and Jyant Kumar (1994). It is to be observed that among the theories considered, the theory of Meyerhof and Adams (1968) results in the maximum pullout capacities.

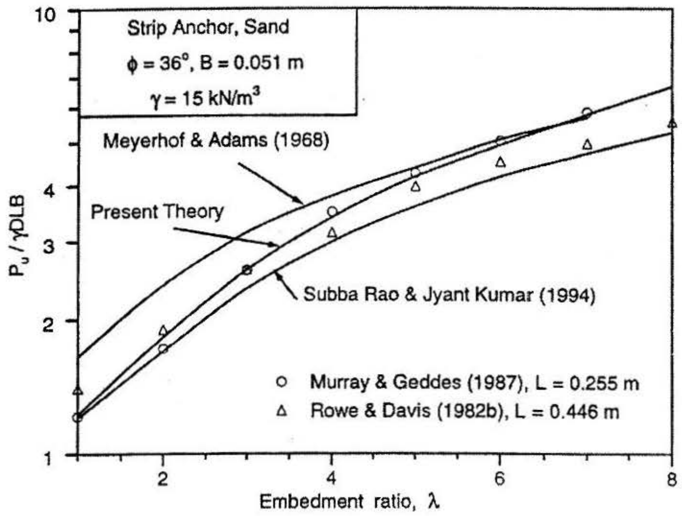

FIGURE 8 : Comparison of Theories for Strip Anchor in Loose Sand, ϕ = 30°

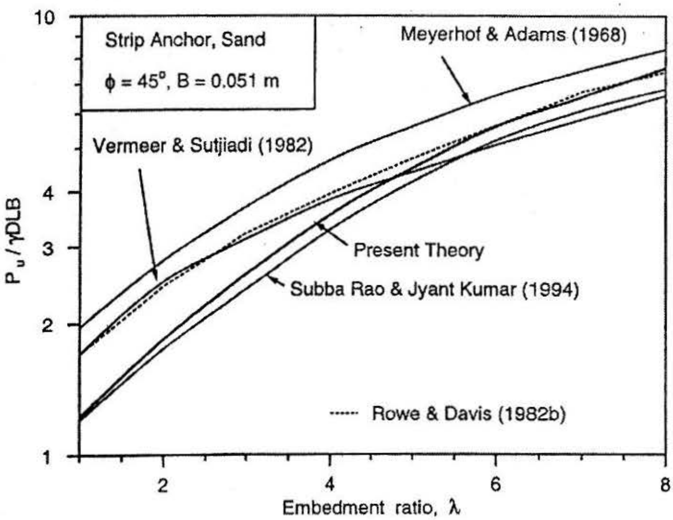

FIGURE 9 : Comparison of Results for Strip Anchor in Dense Sand, ϕ = 45°

Figure 10 shows the comparisons for strip anchor in medium dense sand considering the conventional model test results of Murray and Geddes (1987) and Rowe and Davis (1982 b) for ϕ = 36°. The predictions of the proposed theory are seen to be close to the observed values. Figure 11

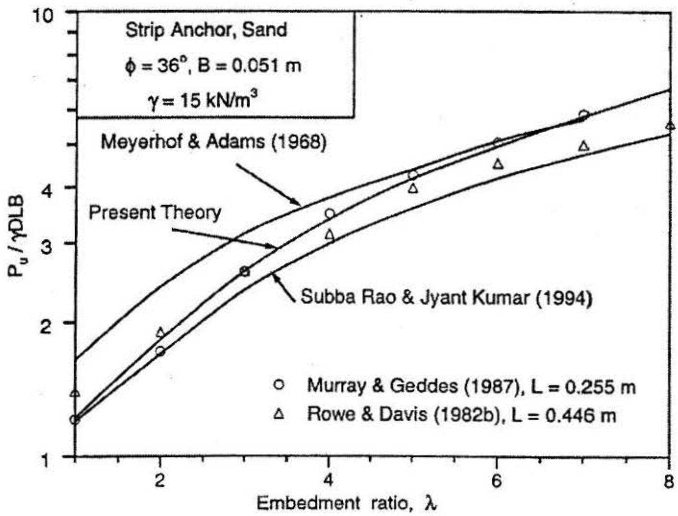

FIGURE 10 : Comparison of Results for Strip Anchor in Medium Dense Sand

shows the comparisons of the predictions made by the available theories and the present theory in terms of F_c for strip anchor in clay. The theory of Vermeer and Sutjiadi (1985) which is based on the assumption of linear failure surface which is assumed to be inclined at dilatancy angle with the

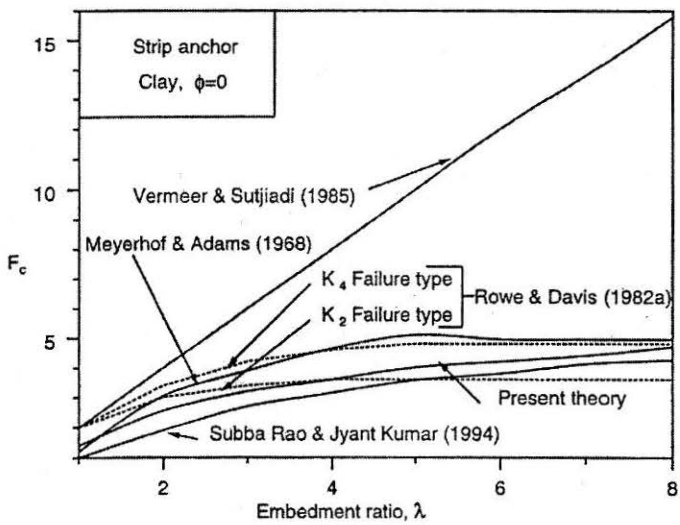

FIGURE 11 : Comparison of Theories in Clay for Strip Anchor

vertical, is seen to result in very high pull out values in comparison with the other theories in clays. The predictions of both the present theory and the theory of Rowe and Davis (1982a) are close. A series of pull-out tests in compacted $(c - \phi)$ soil were performed by Ranganath (1993) on strip anchor. The test results along with the predictions from different theories have been reported in Table 5. The present theory shows a reasonable agreement with the experimental values while the theory of Meyerhof and Adams (1968) shows an overestimation.

From the above discussion for circular and strip anchors it is observed that the results from the theory of Meyerhof and Adams (1968) overestimate $P_u/\gamma AD$ for strip anchors and underestimates the same for circular anchors. This is due to the fact that for strip anchor, the theory of Meyerhof and Adams (1968) was developed by using available earth pressure coefficients for curved failure surfaces (Kerisel and Absi, 1990) and the theory has been extended to circular anchor using shape factor governing the passive earth pressure on a convex cylindrical wall in the corresponding term of the general expression for strip anchor.

Extent of Failure Surface at the Ground

Figure 12a shows the extent of failure surface (X_g) at the ground level from the centre of the anchor plate in terms of (X_g/D) ratio as affected by ϕ and λ for circular anchors. Figure 12b shows the comparison of X_g/D obtained by the present theory with the results of Subba Rao and Jyant Kumar (1994) for strip anchor. The present theory predicts X_g/D smaller than that obtained from the theory of Subba Rao and Jyant Kumar (1994) for strip anchor. This can be substantiated by considering the reported value of 0.5 for λ = 3 in dense sand by Dickin (1988). Table 6 shows the comparison of X_g/D obtained from the present theory with the available

TABLE 5 : Comparison of Results for Strip Anchor in $c - \phi$ Soil

ϕ =17.5°, c = 8kN/m^2, γ = 12.4 kN/m^3, L = 0.35 m, B = 0.05 m	$P_u/\gamma DLB$			
Embedment Ratio, λ	Exptl., Ranganath (1993)	Meyerhof & Adams' Theory (1968)	Subba Rao & Jyant Kumar's Theory (1994)	Present theory
4	12.8	27.5	14.7	17.6
5	16.7	27.6	14.8	17.8
6	16.8	28.0	14.1	17.3

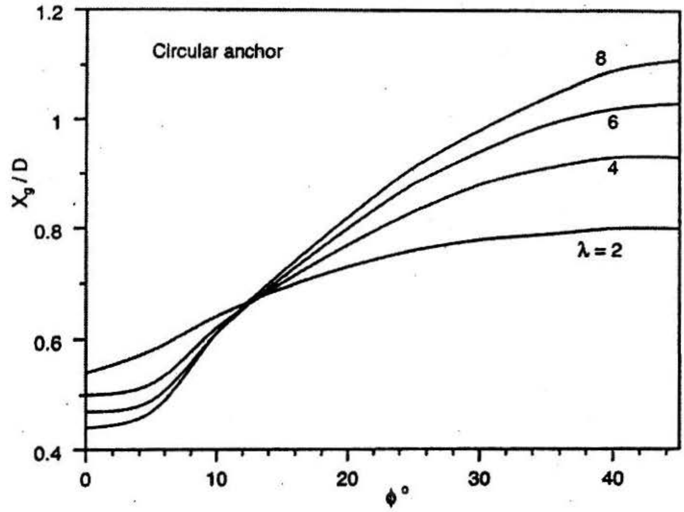

FIGURE 12a : Extent of Failure Surface at Ground Level for Circular Anchor

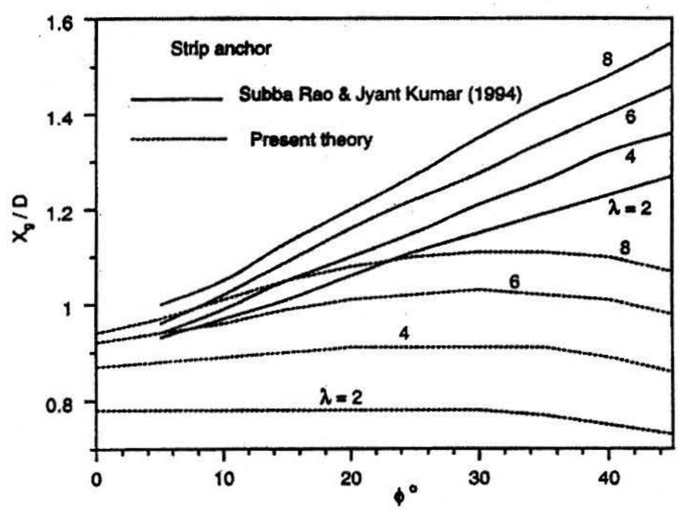

FIGURE 12b : Comparison of X_g/D for Strip Anchor

experimental and theoretical values for circular anchor when $\phi = 34°$. It shows that present theoretical values are comparing favourably with the experimental values of Khadilkar et al. (1971). This is also supported by the experimental results of Janardan Jha (1965) for $\lambda = 5$ for circular anchor in sand which showed $X_g/D = 0.75$.

TABLE 6 : Comparison of (X_g/D) for Circular Anchor

$\phi = 34°$, $c = 4.4$ kN/m^2 $\gamma = 17$ kN/ m^3	X_g / D			
Embedment Ratio, λ	Exptl., Khadilkar et al. (1971)	Balla's theory (1961)	Khadilkar et al. theory (1971)	Present theory
2	0.97	0.85	0.94	0.80
3	0.85	0.77	0.86	0.83
4	0.77	0.72	0.82	0.89
6	0.78	0.68	0.78	0.98

Shape Factors

Figures 13a, 13b and 13c show the variation of shape factors for cohesion (S_{uc}), surcharge (S_{uq}) and unit weight ($S_{u\gamma}$), respectively, for circular anchor when λ varies from 2 to 8. The expressions in Eqns.23, 24 and 25 for S_{uc}, S_{uq} and $S_{u\gamma}$ show how the valves for the plots in Figs.13a, 13b and 13c have been calculated. Referring to Fig.13a for $\phi = 0$, S_{uc} varies from 1.9 to 2.4 when λ varies from 2 to 8 but the suggested value of S_{uc} for $\phi = 0$ by Meyerhof and Adams (1968) is 2.0 for all λ. When $\phi = 0$, both S_{uq} and $S_{u\gamma}$ are equal to 1. All the shape factors increase as ϕ and λ increase. Using the shape factors for cohesion, surcharge and unit weight for circular anchor presented in this paper it is possible to obtain the uplift capacity for circular anchor from the results of the strip anchor for the same ϕ, λ and the width of anchor.

Conclusions

The proposed uplift capacity theory based on method of characteristics assumes the failure surface to be a log-spiral for the entire depth of embedment. The uplift capacity factors F_c, F_q and F_γ have been obtained for both strip and circular anchors in a general $(c - \phi)$ soil as functions of ϕ and λ.

The shape factors for circular anchors associated with cohesion, surcharge and unit weight have been obtained as functions of ϕ and λ.

The proposed theory predicts the uplift capacity for shallow horizontal circular and strip anchors in general $(c - \phi)$ soils, satisfactorily. The theory also predicts satisfactorily the extent of failure surface at the ground.

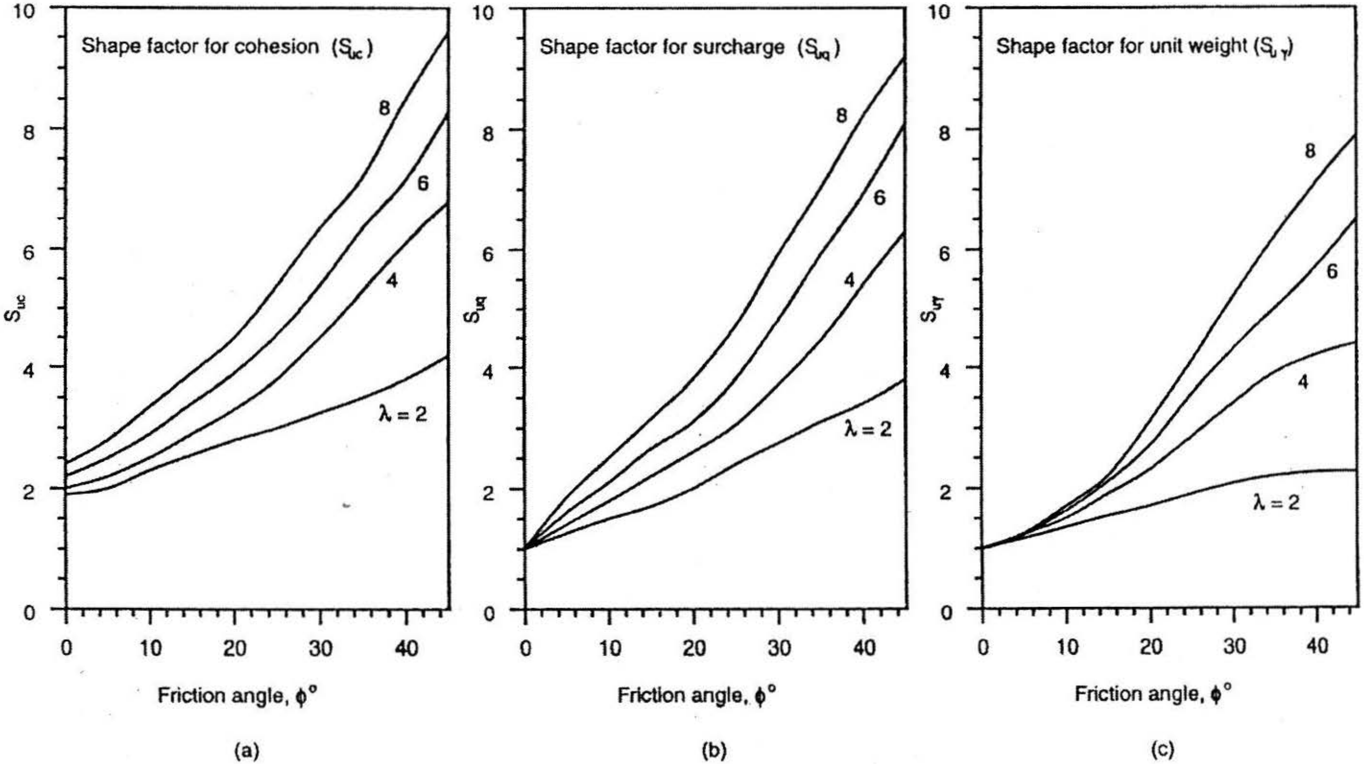

FIGURE 13 : Variation of Shape Factors S_{uc}, S_{uq} and $S_{u\gamma}$

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Notations

A = area of anchor plate.

B = width of strip anchor or diameter of circular anchor.

c = cohesion of soil.

D = depth of anchor plate from ground.

F_c = uplift capacity factor for cohesion.

F_q = uplift capacity factor for surcharge.

F_γ = uplift capacity factor for unit weight.

L = length of anchor.

N_{uc} = breakout factor for cohesion.

N_{uq} = breakout factor for surcharge.

$N_{u\gamma}$	=	breakout factor for unit weight.
P_u	=	ultimate uplift load.
p_u	=	average ultimate uplift pressure.
q	=	surcharge pressure at ground level.
S_{uc}	=	shape factor for cohesion.
S_{uq}	=	shape factor for surcharge.
$S_{u\gamma}$	=	shape factor for unit weight.
V	=	volume of breaking out soil mass.
V_s	=	total vertical downward component of resultant force over the failure surface.
X_g	=	extent of failure surface at ground from centre of anchor.
x	=	x-coordinate.
y	=	y-coordinate.
γ	=	unit weight of soil.
γ'	=	submerged unit weight of soil
θ	=	angle which the direction of major principal stress makes with reference axis.
λ	=	embedment ratio.
μ	=	$45° - \phi/2$.
σ	=	stress difference between the centre of Mohr circle and origin of failure envelope.
σ_n	=	normal stress on the failure surface.
τ	=	shear stress on the failure surface.
ϕ	=	angle of internal friction of soil.