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Pullout capacity of vertical plate anchors in cohesion-less soil

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Abstract. In this paper, the ultimate pullout capacity of a vertical plate strip anchors in cohesion-less soil is analyzed with the consideration of active and passive state of equilibrium in the soil. Kötter's equation is used to compute the active and passive thrusts (along with their point of application) which are subsequently used in the analysis in which, all the equation of equilibrium are properly interpreted. A comparison of the results with the experimental results vis-à-vis available theoretical/empirical solutions shows that, the proposed analysis provides a better estimate of the pullout capacity.

Keywords: vertical anchor plates; cohesion-less soil; pullout capacity; kötter's (1903) equation.

1. Introduction

Generally, earth anchors are used to transmit tensile forces from a structure to the soil. Their pullout capacity is obtained through the shear strength and dead weight of the surrounding soil. Plate anchors may be made of a steel plate and precast or cast in situ concrete slab. These anchors can be installed by excavating the ground to the required depth followed by back filling and compacting with a good quality soil.

The analysis of ultimate pullout capacity of vertical anchor plates is quite similar to that for shallow and deep horizontal anchors. In case of shallow anchors, the embedment ratio is such that, failure surface reaches the ground surface at limit equilibrium; whereas in case of deep anchors, the embedment ratio is such that, failure surface does not reach the ground surface at limit equilibrium (Das 1990).

The proposed analysis is confined to shallow laid anchors in cohesionless soil.

2. Experimental Investigations

Buchholz (1930) observed that, critical embedment ratio was the ratio beyond which, the failure surface did not reach the soil surface. He defined this ratio as H/h = 7. Hueckel (1957) has reported

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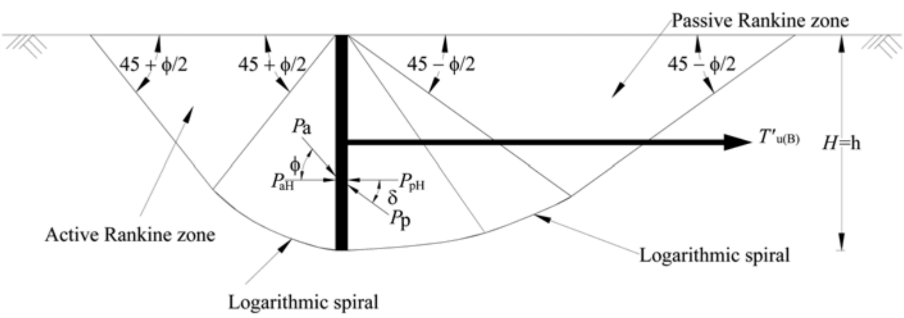

Fig. 1 Basic case - failure surface of a vertical anchor plate in cohesionless soil (Ovesen and Stromann 1972)

that, ultimate resistance of inclined plates with the pull in horizontal direction was smaller than that of the vertical one, regardless of their angle of inclination.

Ovesen and Stromann (1964, 1972) used the failure mechanism proposed by Hansen (1953) to estimate the earth pressures for the case of a continuous shallow plate anchor flushing with the cohesion-less ground surface, termed as the basic case ($H/h = 1.0$). The failure mechanism consists of Rankine (1857) and logarithmic spiral zones (Terzaghi 1943) as shown in Fig. 1.

Based on the above failure mechanism and laboratory model tests, the ultimate pullout capacity, per unit width of a strip anchor in cohesionless soil was estimated as

$$T_u = R_{0v} T_{u(B)} \tag{1}$$

Where $T_{u(B)}$ is the ultimate holding capacity as estimated by the following expression from Fig. 1 with horizontal force equilibrium.

$$T_{u(B)} = P_{PH} - P_{aH} \tag{2}$$

In the above expression, P_{pH} and P_{aH} are the horizontal components of the passive and active thrusts, which can be estimated using the earth pressure coefficients reported by Caquot and Kerisel (1949). The parameter, R_{0v} in Eq. (1) is given as (Dickin and Leung 1985)

$$R_{0v} = \frac{C_{0v} + 1}{C_{0v} + H/h} \tag{3}$$

where, $C_{0v} = 19$ for dense sand and 14 for loose sands.

Neely *et al.* (1973) performed laboratory tests on anchor plates in dry sand and ultimate resistances of these plates were examined using both limit analysis and the method of stress characteristics. Results of tests on rigid anchor plates in terms of $M_{\gamma q}$, a dimensionless force coefficient, were expressed as $M_{\gamma q} = T_u/\gamma B h^2$. $M_{\gamma q}$ varied strongly with geometry and for this, a dimensionless parameter known as shape factor (S_f) was introduced that depended on B/h and H/h ratios.

Das and Seeley (1975) conducted several laboratory model tests to determine the ultimate pullout resistance of shallow vertical anchors and suggested a simple semi-empirical relation for the pullout resistance in a non-dimensional form as the ratio of $T_u/\gamma B h^2$ for square and rectangular anchors. Ultimate pull out capacity for a single anchor was expressed by the following relation.

$$T_u = (4.59 \times 10^{-5}) S \phi^{3.22} (H/h)^n \gamma B h^2 \tag{4}$$

where, S is the shape factor which is a function of H/h and ϕ is angle of soil friction in degrees.

The value of n varies linearly from 1.8, for $B/h = 1$ to about 1.68 for $B/h = 5$.

The capacity of deeper vertical anchors in medium dense sand was investigated by Akinmusuru (1978) for square, circular and rectangular anchors. On the basis of experimental findings, the variation of $T_u/\gamma h^3$, a non-dimensional anchor load at ultimate failure with λ, a non-dimensional embedment coefficient ($\lambda = H/h$) was presented in the form of a chart, where T_u is ultimate pullout load for an anchor length of 10h. Akinmusuru (1978) clearly defined the critical embedment depth as the one corresponding to $\lambda = 6.5$.

Dickin and Leung (1983) conducted both centrifuge and conventional chamber tests and reported very thorough investigations on the behavior of vertical square and rectangular anchors in dense sand. The variations of breakout factor $N_{\gamma q}$, and the force coefficient $M_{\gamma q}$ with embedment ratio were separately reported in the form of a chart with $N_{\gamma q} = T_u/\gamma BhH$ and $M_{\gamma q} = T_u/\gamma Bh^2$. The results obtained by them suggested potentially serious over predictions of pull-out resistance and underestimations of the failure displacements. Such errors arose due to the characteristic stress-dependent behavior of dense soils.

Hoshiya and Mandal (1984) investigated the capacity of square and rectangular anchors in loose sand. The size of box used for testing was very small, which facilitated the testing of only 300 mm wide and 400 mm long plates. This was likely to introduce edge effects into the results. They concluded that, anchor break-out factor increased with depth up to a certain embedment ratio before reaching a constant value thereafter.

Naser (2006) carried out theoretical as well as experimental studies on the ultimate pullout capacity of a block anchor of concrete embedded in sand and observed that, anchor thickness contributed to the pullout capacity through base friction forces. This contribution was not significant as compared to the passive resistance. Uplifting and tilting of the block was also observed.

3. Theoretical investigations

Terzaghi (1943) evaluated the resistance of vertical strip anchor plates assuming Rankine (1857) states of passive and active pressures. This approach was adopted in the British civil engineering code of practice. The net resistance of a vertical anchor, T_u is given as $(P_p - P_a)$, where P_p and P_a are the passive and active thrusts (kN/m) acting on the anchor plate.

Teng (1962) estimated holding capacity of a vertical (strip) plate anchor embedded in granular soil at relatively shallow depth ($h/H \leq 1/3$ to $1/2$), based on Rankine's (1857) theory of lateral earth pressures. He obtained the expression for ultimate holding capacity as, $T_u' = P_p - P_a$ where P_p and P_a are the passive and active pressure thrust (kN/m) acting on anchor plate. For anchors with limited width B, the frictional resistance developed along the vertical faces of the failure surface was also taken into account and the expression for ultimate holding capacity (T_u) was reported as

$$T_u = T_u'B + \frac{1}{3}K_0\gamma[\sqrt{K_p} + H\sqrt{K_a}]H^3 \tan\phi \tag{5}$$

where, K_0 is the coefficient of earth pressure at rest, K_a is the Rankine's active earth pressure coefficient and K_p is the Rankine's passive earth pressure coefficient.

In case of shallow strip anchors, Meyerhof (1968, 1973) used the passive and active coefficients of earth pressure proposed by Caqout and Kerisel (1949) and Sokolovskii (1965) and proposed the following simple relationship for ultimate holding capacity per unit width of a continuous (strip)

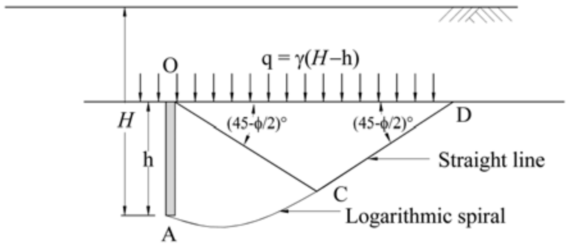

Fig. 2 Surcharge method (Neely *et al.* 1973)

vertical plate anchor.

$$T_u = 1/2\gamma H^2 K_b \tag{6}$$

where, K_b is the pullout coefficient that can be obtained from a graph using soil friction angle.

Neely *et al.* (1973) determined the theoretical resistance of continuous (strip) vertical anchor plates in cohesionless soils by two methods. In the first method, failure surface was assumed to be consisting of a logarithmic spirals and its tangent inclined at (45°-ϕ/2) to the horizontal as shown in Fig. 2. Soil above top of the anchor was considered to act as a simple surcharge, $q\{\gamma(H\text{-}h)\}$ and therefore, the method was termed as surcharge method.

Shearing resistance of the soil above the anchor top is ignored when H/h is small; therefore the method was subsequently modified by considering shear strength above the top of anchor plate when H/h is considerable and was defined as the Equivalent Free Surface method. The assumed failure surface in soil (Fig. 3) is an arc of logarithmic spiral with pole at top of the wall. OB is a straight line which is an equivalent free surface. The shearing resistance of upper layers of soil was included in the calculation by making use of equivalent free surface concept proposed by Meyerhof (1951) in connection with bearing capacity of shallow foundations. The normal and shear stresses along OB (p_0 and s_0, respectively) were calculated using Rankine (1857) active stresses on the vertical surface, OA above the top of the anchor plate as shown in Fig. 3.

The above analysis is based on the method of stress characteristics and represents a more refined analytical and numerical attempt to predict the ultimate capacity of the vertical plate anchors but it ignores the active earth pressure distribution behind the anchor plate and the kinematic behaviour of the material.

Rowe and Davis (1982b) reported a two-dimensional finite element analysis incorporating an elasto-plastic soil model. For a continuous vertical plate anchor assumed to be thin and perfectly rigid, the resistance is as given by the following expression.

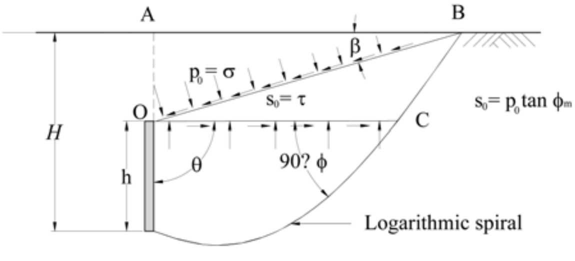

Fig. 3 Equivalent free surface analysis (Neely *et al.* 1973)

$$M_{\gamma q} = F_{\gamma} R_{\psi} R_R R_K \tag{7}$$

where, F_{γ} is the capacity factor of a smooth anchor resting on soil which deforms plastically at a constant volume ($\psi = 0°$), with coefficient of earth pressure at rest, $K_0 = 1$ and R_{ψ}, R_R and R_K are correction factors for the effects of sand dilatancy, anchor plate roughness and initial stress state respectively. The theoretical data was presented in the form of design charts.

A comparative study of the force coefficient, $M_{\gamma q}$ as obtained from experimental investigations and theoretical methods proposed by Ovesen and Stromann (1972), Neely and Stuart, (1973) was carried out by Dickin (1983). Significant disparity was observed in the results because they were based on two-dimensional analysis and their application to single anchors required a suitable shape factor. Dickin and Leung (1985) observed that, effect of anchor shape on dimensionless coefficients was due to side shear resistance. They observed failure planes radiating outward involving a soil mass wider than a single anchor itself in the failed body. A dimensionless shape factor, S_f to account for the influence of anchor geometry on the ultimate resistance was introduced by them.

Finite element method is also used by various researchers such as Vemeer and Sutjiadi (1985), Tagaya *et al.* (1983, 1988), Dickin and King (1997) and Sakai and Tanaka (1998). Unfortunately, only limited results are available from these studies. Tagaya *et al.* (1983, 1988) reported two-dimensional plane strain and axi-symmetric finite element analyses using the constitutive law of Lade and Duncan (1975).

Upper and lower bound limit analyses are also reported by Murray and Geddes (1987, 1989), Basudhar and Singh (1994) and Smith (1998) to estimate the capacity of vertical strip anchor plates. Basudhar and Singh (1994) obtained estimates with a generalized lower bound procedure based on finite element method and non-linear programming similar to that of Sloan (1988). The solutions proposed by Murray and Geddes (1987, 1989) are based on kinematically admissible failure mechanisms (upper bound).

Kumar and Rao (2004) has extended the concept of the equivalent free surface to determine the seismic horizontal pullout capacity of shallow vertical strip plate anchors buried in sand using the method of stress characteristics.

Merifield *et al.* (2006) presented the results of a rigorous numerical work (finite elements coupled with upper and lower bound limit analyses) to estimate the ultimate pullout capacity T_u, for vertical anchor plate in the cohesionless material. For comparison purposes, numerical and theoretical results of the break-out factor were presented in the form analogous to Terzaghi's (1943) equation of the bearing capacity of shallow foundations.

$$T_u = \gamma H N_{\gamma} \tag{8}$$

where, N_{γ} is the anchor break-out factor that can be obtained from a graph using soil friction angle.

The failure mode (Fig. 4) reported by Merifield *et al.* (2006) for vertical anchors indicates that the

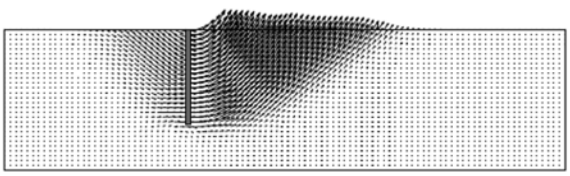

Fig. 4 Failure modes and zones of plastic yielding for rough vertical anchors in cohesionless soils ($\phi = 20°$), Merifield *et al.* (2006)

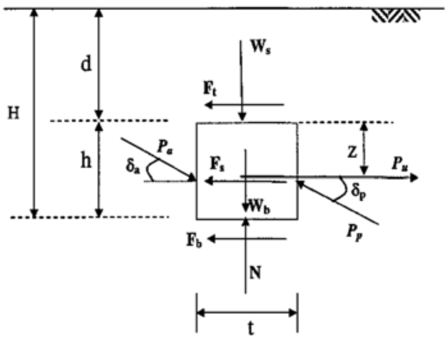

Fig. 5 Free body diagram of block anchor (Naser 2006)

soil retained behind the anchor can significantly affect the estimated capacity of shallow anchors. This is particularly the case for loose sands, where the development of a significant active zone behind the anchor is observed. Changing the interface roughness from perfectly rough to perfectly smooth can lead to a reduction in the anchor capacity by as much as 65%.

Naser (2006) analyzed pullout capacity of an anchor block using limit equilibrium approach (Fig. 5).

The ultimate pullout capacity of block anchor (T_u) was obtained from the equilibrium of forces acting on the block by summing them along the horizontal direction and multiplying the lateral earth pressures (passive and active) by the 3-D corrections factor M, to yield the following equation.

$$T_u = M(P_{ph} - P_{ah}) + F_t + F_s + F_b \quad (9)$$

where, F_t, F_b and F_s are the effective friction forces at the top, bottom and at two side of the block, N is the normal force, P_{ph} is the effective horizontal passive thrust and P_{ah} is the effective horizontal active thrust. For Coulomb (1776) and log spiral theories, $F_b = 0$ (as $N = 0$). Pullout capacity of block anchor with Rankine's theory (1857), corrected for the 3-D effect with the contribution of friction, showed a close agreement with experimental results.

In the present analysis, for a vertical strip plate anchor (basic case, $H/h = 1.0$) a total of seven experimental studies namely, Ovesen and Stromann (1964, 1972), Neely *et al.* (1973), Das and Seeley (1975), Akinmusuru (1978), Dickin and Leung (1983, 1985), Hoshiya and Mandal (1984) and Murray and Geddes (1987) are referred for comparison with the proposed solution.

4. Proposed method

In the proposed analysis of the estimation of pullout capacity of a strip anchor in cohessionless soil, all the three equation of equilibrium are utilized to obtain the required solution. Both passive and active states of equilibrium on the two sides of anchors are considered in the analysis. The active/passive thrusts along with their points of application are evaluated using Kötter's (1903) equation. This equation has been used by other researchers such as Dewaikar and Mohapatro (2003) for computation of bearing capacity factor, N_γ, Deshmukh *et al.* (2010) for the estimation of breakout capacity of horizontal rectangular/square anchors in cohesionless soils and Kame *et al.* (2010a) for estimation of active thrust on a vertical retaining wall with horizontal cohesionless backfill.

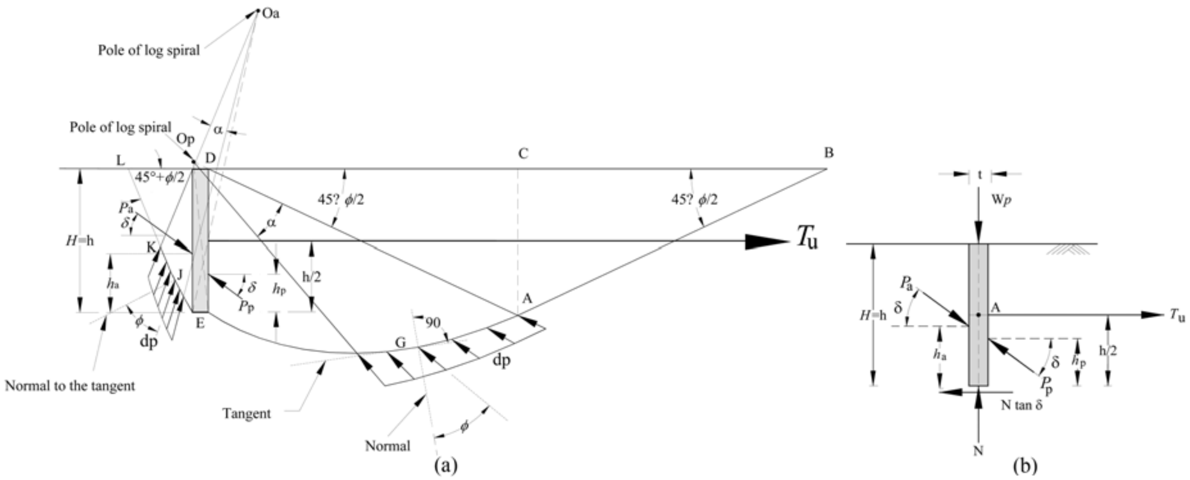

Fig. 6 (a) Failure mechanism at ultimate load for continuous (strip) vertical plate anchor in cohesion-less soil and (b) Free body diagram of the anchor plate

4.1 Geometry of the problem

In Fig. 6(a), the failure mechanism adopted in the analysis is shown. There are passive and active states of equilibrium in which the failure surface consists of a log spiral followed by its tangent that meets the ground surface. The anchor is flushing with the ground surface (basic case, H/h = 1.0).

In Fig. 6(b), free body diagram of the strip anchor is shown from which, the following information is generated.

P_P = resultant passive thrust
P_a = resultant active thrust
δ = angle of friction between soil and the plate anchor
ϕ = angle of soil friction
h_p = distance of point of application of passive thrust, P_p from the anchor base
h_a = distance of point of application of active thrust, P_a from the anchor base
W_p = weight of the anchor plate per meter
t = thickness of the anchor plate
N = upward soil reaction

The parameters P_p, P_a, h_p and h_a are computed using Kötter's (1903) equation.

4.2 Kötter's (1903) equation

In a cohesionless soil medium with passive and active states of equilibrium under plane strain condition, Kötter's (1903) equation is given as

$$\frac{dp}{ds} + 2p\tan\phi\frac{d\alpha}{ds} = \gamma\sin(\alpha + \phi) \quad \text{for the passive states (Fig. 7(a))} \tag{10a}$$

And

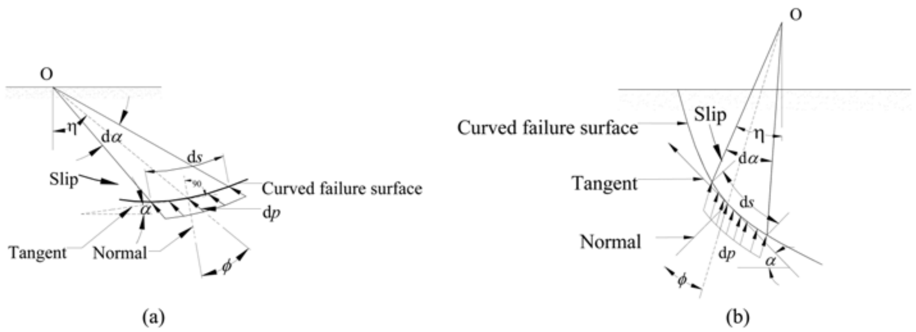

Fig. 7 (a) Reactive pressure distribution on the failure surface for passive case (Kame *et al.* 2010b), (b) Reactive pressure distribution on the failure surface for active case (Kame *et al.* 2010a)

$$\frac{dp}{ds} - 2p\tan\phi\frac{d\alpha}{ds} = \gamma\sin(\alpha - \phi) \text{ for the active states (Fig. 7(b))} \tag{10b}$$

In the above equations,

dp = differential reactive pressure on the failure surface
ds = differential length of arc of failure surface
ϕ = angle of soil internal friction
$d\alpha$ = differential angle and,
α = inclination of the tangent at the point of interest with the horizontal

Kame *et al.* (2010a or 2010b) have reported a method based on the application of Kötter's (1903) equation for the estimation of active and passive thrusts on a vertical wall retaining horizontal cohesionless backfill. The unique failure surface consisting of a log spiral and its tangent is identified on the basis of force equilibrium conditions and the point of application of active/passive thrust is computed using moment equilibrium. In the proposed analysis, this procedure is adopted to compute the values of P_p, P_a, h_p and h_a. The final expression for the reactive passive pressure distribution at any point on the curved failure surface using Kötter's (1903) equation is obtained with the following expression.

$$p = \begin{bmatrix} \gamma r_0 \mathrm{K}\sin\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\tan\phi(3\theta_m - 2\theta)} \\ + \frac{\gamma r_0 \sec\phi e^{\theta\tan\phi}}{(1 + 9\tan^2\phi)} \begin{Bmatrix} 3\tan\phi\sin(\theta - \theta_L + \phi) \\ -\cos(\theta - \theta_L + \phi) \end{Bmatrix} \\ - \frac{\gamma r_0 \sec\phi e^{\tan\phi(3\theta_m - 2\theta)}}{(1 + 9\tan^2\phi)} \begin{Bmatrix} 3\tan\phi\sin(\theta_m - \theta_L + \phi) \\ -\cos(\theta_m - \theta_L + \phi) \end{Bmatrix} \end{bmatrix} \tag{11}$$

Where, K is the parameter indicating location of the pole of the log spiral along line AO in terms of starting radius of log spiral r_0 as measured from point D (Fig. 8).

θ = spiral angle measured from the starting radius
r_0 = starting radius of the log spiral at the wall base (at $\theta = 0$)
θ_m = maximum spiral angle
θ_V = angle between vertical face of the wall and the starting radius r_0 and

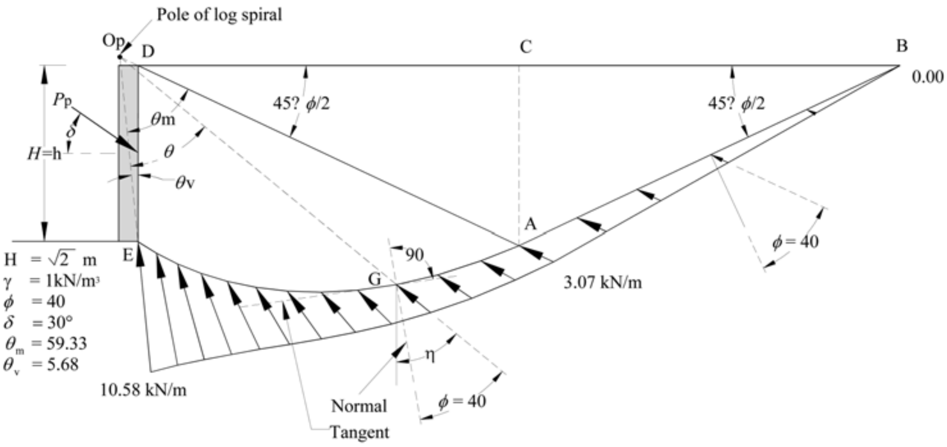

Fig. 8 Reactive pressure distribution on the failure surface for the passive case using Kötter's (1903) equation (Kame *et al*. 2010b)

$\theta_L = (90\text{-}\theta_V)$

Similarly, the final expression for reactive active pressure distribution at any point on the curved failure surface based on the application of Kötter's (1903) equation is given by the following expression.

$$p = \begin{bmatrix} \gamma \cdot r_0 \sec\phi e^{\tan\phi\theta}\left\{\dfrac{3\tan\phi\sin(\theta_L-\theta)+\cos(\theta_L-\theta)}{1+9\tan^2\phi}\right\} \\ -\gamma \cdot r_0 \sec\phi e^{-\tan\phi 2\theta}\left\{\dfrac{3\tan\phi\sin\theta_L+\cos\theta_L}{1+9\tan^2\phi}\right\}+\gamma\cdot e^{-\tan\phi 2\theta}\{\sin\theta_L \mathrm{AD}\} \end{bmatrix} \tag{12}$$

where, $\theta_L = (45\text{-}\phi/2)$ and AD = length of line AD (Fig. 9)

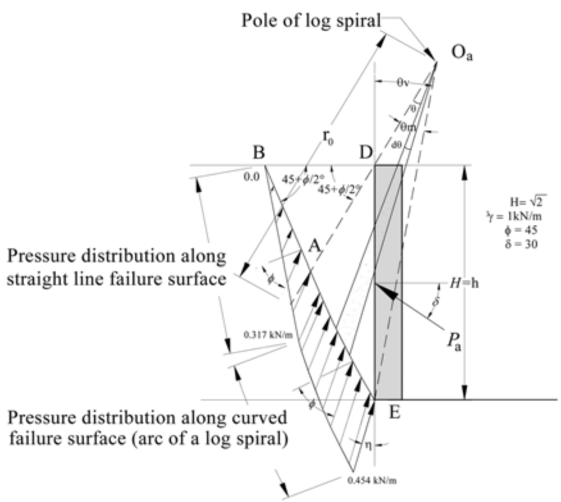

Fig. 9 Reactive pressure distribution on the failure surface for the active case using Kötter's (1903) equation (Kame *et al.* 2010a)

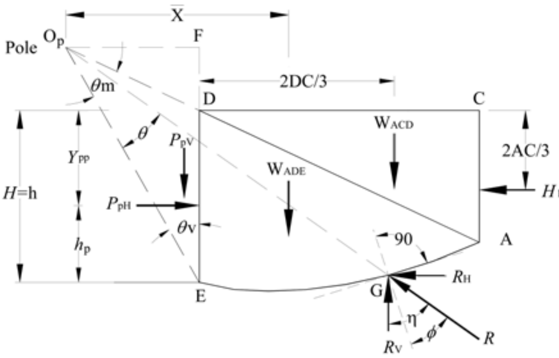

Fig. 10 Free body diagram of failure wedge EACD (passive state)

The distribution of reactive pressures on the failure surface in both cases using Kötter's (1903) equation are as shown in Figs. 8 and 9. The magnitudes of passive and active thrusts on the vertical plate and their points of application are thus obtained using Kötter's (1903) equation.

The resultant soil reaction, R (Fig. 10) on the failure surface is obtained as

$$R = \int p.ds \tag{13}$$

The vertical and horizontal components, R_V and R_H of resultant reaction are obtained as

$$R_V = \int_0^{\theta_m} p\cos\eta.ds \tag{14}$$

$$R_H = \int_0^{\theta_m} p\sin\eta.ds \tag{15}$$

where, ds is the length of failure surface and η is the varying angle of inclination of reactive pressure with vertical (Figs. 7(a) and (b)). The detailed calculations for estimation of active thrust are reported in the paper "Active thrust on a vertical retaining wall with cohesion-less backfill" published in the Electronic Journal of Geotechnical Engineering, (EJGE), U.S.A. (Vol. 15(Q), Page 1848-1863, Kame *et al.* 2010a) and the detailed calculations for estimation of the passive thrust are reported in the paper "Passive thrust on a vertical retaining wall with horizontal cohesion-less backfill" communicated to Soils and Rocks, An International Journal of Geotechnical and Geo-environmental Engineering, Brazil. (Kame *et al.* 2010b)

4.3 Magnitude of passive thrust

In Fig. 10, which shows free body diagram of failure wedge EACD the passive Rankine thrust H_1 acts at a distance 2/3. AC from point, C. Static equilibrium of wedge, EACD is then considered.

Vertical force equilibrium condition gives

$$P_{pV} = P_p\sin\,\delta = R_v - W_{ACD} - W_{ADE} \tag{16}$$

From which, P_p is obtained as,

$$P_p = \frac{R_v - W_{ACD} - W_{ADE}}{\sin\delta} \tag{17}$$

Horizontal force equilibrium condition gives

$$P_{pH} = P_p \cos\delta = R_H + H_1 \tag{18}$$

From which, P_p is obtained as

$$P_p = \frac{R_H + H_1}{\cos\delta} \tag{19}$$

where,

P_{pH}, P_{pV} = horizontal and vertical components of resultant passive thrust, P_p

R_H, R_V = horizontal and vertical components of resultant soil reaction acting on the curved part of the failure surface

H_1 = passive thrust exerted by the backfill on the Rankine wall AC

W_{ACD} = weight of soil in the failure wedge, forming a part of the Rankine zone

W_{ADE} = weight of soil in the zone, EAD of the failure wedge, EABCD

Y_{pp} = the distance of point of application of P_p from the wall top

$\bar{X}$ = the distance from pole O to the centroid of log spiral

h_p = distance of point of application of passive thrust, P_p from the anchor base

η = varying angle of inclination of reactive pressure with vertical

It may be noted that, both Eqs. (17) and (19) give the magnitude of unknown thrust, P_p. These two equations will yield the same and unique value of P_p only when the equilibrium conditions correspond to those at failure, which are uniquely defined by a characteristic value of θ_V and this value can be determined by trial and error procedure.

4.4 Trial and error procedure

In this procedure, first a trial value of θ_V is assumed and corresponding weight of trial failure wedge, EACD (Fig. 10) is computed. Using Eqs. (14) and (15), magnitudes of vertical and horizontal components of soil reaction (R_V and R_H) are computed and from Eqs. (17) and (19), values of P_p are determined. If the trial value of θ_V is equal to its characteristic value corresponding

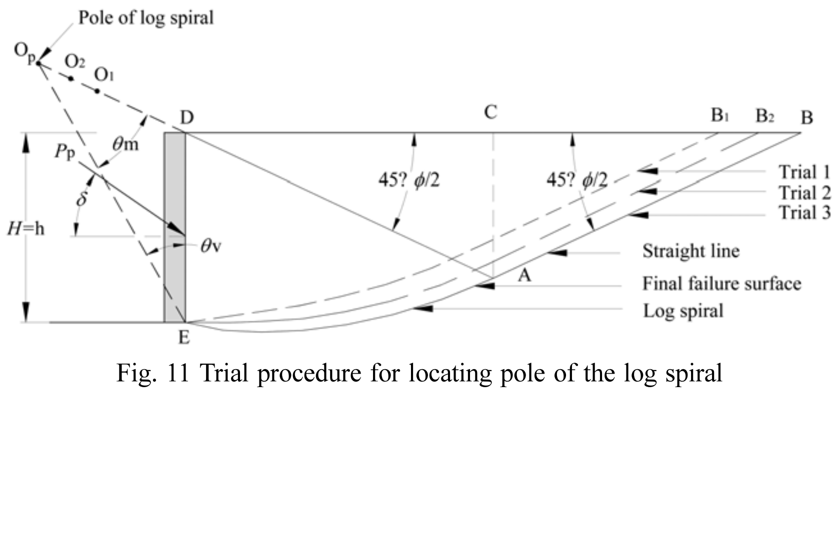

Fig. 11 Trial procedure for locating pole of the log spiral

Table 1 Earth pressure coefficients using proposed method (Kame *et al.* 2010a and 2010b)

Angle of friction		Earth pressure coefficients	
Soil, ϕ (degrees)	Wall, δ (degrees)	K_a (Active state)	K_p (Passive state)
20		0.430	3.086
25		0.349	4.097
30	2/3ϕ	0.282	5.606
35		0.227	7.983
40		0.180	11.995
20		0.410	3.290
25		0.329	4.560
30	ϕ	0.263	6.572
35		0.209	10.018
40		0.165	16.464

to the failure condition, the two computed values of P_p will be the same; otherwise, they will be different. For various trial values of θ_V, computations are carried out till the convergence is reached to a specified (third) decimal accuracy. Thus, in this method of analysis, the unique failure surface (Fig. 11) is identified by locating the pole of log spiral in such a manner that, force equilibrium condition of failure wedge, EACD is satisfied. This approach is different from other analyses in which, P_p is obtained from the consideration of its minimum value.

Values of the passive thrust are obtained for different values of angles of soil friction, ϕ and wall friction, δ. In Table 1 some of the computed values of passive earth pressure co-efficient, K_p $(2P_p/\gamma H^2)$ are reported.

Similar procedure (Kame *et al.* 2010a) is adopted for determination of active thrust for different values of angles of soil friction, ϕ and wall friction, δ and in Table 1 some of the computed values of active earth pressure co-efficient, K_a $(2P_a/\gamma H^2)$ are reported.

5. Analysis of anchor plate

Referring to Fig. 6(b), all the three equilibrium conditions are examined.

5.1 Vertical equilibrium

The forces involved are $P_p\sin\delta$ and N in the vertically upward direction and $P_a\sin\delta$ and W_p in the vertically downward direction. Since $P_p > P_a$ and weight, W_p of the plate is small enough, there is no equilibrium of the forces in the vertical direction. The reaction, N is zero and the plate accelerates in the vertically upward direction. This agrees well with the experimental observation (Naser 2006).

5.2 Moment equilibrium

The forces N and $N\tan\delta$ now can be considered to be zero and the moment equilibrium is considered about the point, A as shown in Fig. 6(b). The only forces that contribute to the moment equilibrium are P_a and P_p and since $P_p > P_a$, clearly moment equilibrium is also not satisfied. The plate rotates at limit equilibrium and this agrees with the experimental observation (Naser 2006).

5.3 Horizontal equilibrium

Summing up the forces in horizontal direction the following expression is obtained.

$$T_u = P_p \cos\delta - P_a \cos\delta \tag{20}$$

As stated earlier, P_p and P_a are evaluated using Kötter's (1903) equation and then T_u is determined from the horizontal equilibrium condition.

The values of T_u are computed using the available experimental data and comparisons with the available theoretical solutions are made.

6. Discussion

In Table 2, the data of tests conducted by various researchers on the vertical plate anchors in cohesionless soil for the basic case (H/h = 1.0) is reported.

In Table 3, the experimental values of T_u (kN/m) are reported in column 2 of the table. In the same table, the values of T_u as computed using various theoretical/empirical solutions are reported along with the results obtained with the proposed method.

From Table 3, it is seen that, the proposed method provides a better estimate of the pullout capacity as compared to the other methods. For example, when compared to experimental value reported by Neely (1973), the proposed method gives an error of +8.9% while the errors in respect to the methods proposed by Ovesen (1973), Das (1975), Akinmusuru (1978), Dickin and Leung (1983, 1985), Hoshiya and Mandal (1984) and Murry & Geddes (1989) are −44.3%, 16.6%, −22.8%, 11.2%, −34.8% and 36.5% respectively. This observation is further substantiated by the data generated in Table 4, which gives cumulative frequency distribution of errors. While generating this data only absolute value of the error is considered.

The proposed method gives absolute error in the range, 0 to 25% in 4 out of 7 cases and in remaining cases the range is 25% to 100%.

In 1 out of 6 cases Ovesen's (1972) method gives absolute errors in the range, 1% to 25% and in

Table 2 Anchor and soil parameters

Author	ϕ (degrees)	δ (degrees)	t (m)	γ (kN/m^3)	H (m)	B (m)	γ_p (kN/m^3)	Material of the plate
Ovesen and Stromann (1972)	42.0	38.7	?	16.770	0.250	1.000	?	Plexiglas
Neely (1973)	38.5	21.0	0.006	15.900	0.051	0.255	77.0	Steel
Das (1975)	34.0	34.0	0.003	15.920	0.038	0.191	28.0	Aluminum
Akinmusuru (1978)	35.0	29.0	0.003	15.550	0.038	0.380	77.0	Steel
Dickin and Leung (1983, 1985)	41.0	29.0	0.003	16.000	0.050	0.250	28.0	Aluminum
Hoshiya and Mandal (1984)	29.5	29.5	0.005	14.120	0.025	0.152	28.0	Aluminum
Murry and Geddes (1989)	43.6	10.6	0.006	16.500	0.051	0.508	77.0	Steel

Table 3 Comparison of pullout capacity – experimental results and semi-empirical methods for vertical (strip) plate anchors

Author	Experimental	Proposed method		Ovesen and Stromann (1972)		Neely (1973)		Das (1975)		Akinmusuru (1978)		Dickin and Leung (1983, 1985)		Hoshiya and Mandal (1984)		Murry and Geddes (1989)	
	T_u (N/m) 1	T_u (N/m) 2	% Error 3	T_u (N/m) 4	% Error 5	T_u (N/m) 6	% Error 7	T_u (N/m) 8	% Error 9	T_u (N/m) 10	% Error 11	T_u (N/m) 12	% Error 13	T_u (N/m) 14	% Error 15	T_u (N/m) 16	% Error 17
Ovesen (1972)	2545	4377	72.0	2545	0	3925	54.2	4577	79.8	2915	14.5	4200	65	2559	0.5	5156	102
Neely (1973)	41.7	45.4	8.9	22.3	-44.3	41.7	0.0	48.6	16.6	30.9	-22.8	44.6	11.2	27.2	-34.8	54.7	36.5
Das (1975)	20.3	16.3	-19.4	7.7	-63.2	17.4	-14.2	20.3	0.0	12.9	-38.2	18.6	-11	11.3	-44.1	22.8	9.3
Akinmusuru (1978)	25.6	32.6	27.3	15.8	-38.4	34.5	34.6	40.2	57.0	25.6	0	36.9	44.1	22.5	-12.2	45.3	76.9
Dickin and Leung (1983, 1985)	42.0	60.3	43.6	23.8	-43.4	39.3	-6.5	45.8	9.0	29.2	-30.6	42	0	25.6	-39.1	51.6	22.8
Hoshiya and Mandal (1984)	4.0	3.7	-9.2	3.8	-10.9	6.2	53.4	7.2	78.8	4.6	8.3	6.6	56	4.0	0.0	8.1	91.6
Murry and Geddes (1989)	108.2	99.7	-7.8	62.5	-42.3	82.3	-23.9	96.0	-11.2	61.2	-43.5	88.1	-18.6	53.7	-50.4	108.2	0

Table 4 Cumulative frequency distribution of errors for vertical plate anchors

Absolute % error	Proposed method		Ovesen and Stromann (1972)		Neely (1973)		Das (1975)		Akinmusuru (1978)		Dickin and Leung (1983, 1985)		Hoshiya and Mandal (1984)		Murry and Geddes (1989)	
	Frequency	Cum. Frequency	Frequency	Cum. Frequency	Frequency	Cum. Frequency	Frequency	Cum. Frequency	Frequency	Cum. Frequency	Frequency	Cum. Frequency	Frequency	Cum. Frequency	Frequency	Cum. Frequency
0-10	3	3	1	1	2	2	2	2	2	2	1	1	2	2	2	2
10-20	1	4	1	2	1	3	2	4	1	3	3	4	1	3	0	2
20-30	1	5	0	2	1	4	0	4	2	5	0	4	0	3	1	3
30-40	0	5	1	3	1	5	0	4	1	6	0	4	2	5	1	4
40-50	1	6	3	6	0	5	0	4	1	7	1	5	1	6	0	4
>50	1	7	1	7	2	7	3	7	0	7	2	7	1	7	3	7

remaining 5 cases the errors are as high as 25% to 100%.

Neely's (1973) method gives absolute errors in the range 1% to 25% in three out of 6 cases and in remaining 3 cases the range is 25% to 100%.

The methods proposed by Das (1975), Akinmusuru (1978) and Dickin and Leung (1983, 1985) give absolute errors in the range, 1% to 25% in 3 out of 6 cases and in remaining cases, the errors range is 25% to 100%.

Similarly, the methods proposed by Hoshiya & Mandal (1984) and Murry & Geddes (1989) are giving absolute errors in the range, 1% to 25% in 2 cases and in remaining cases, the range is 25% to 100%.

The capability of the proposed method is related to the passive and active earth pressure coefficient values which are unique for the failure mechanism consisting of log spiral and its tangent.

7. Conclusions

The active and passive earth pressures are computed using Kötter's (1903) equation which lends itself as a powerful tool and facilitates the identification of a unique failure surface on the basis of force equilibrium conditions. The moment equilibrium condition is effectively used for the computation of points of application of the active and passive thrusts. For the first time, the evaluation of distribution of soil reactions on the failure surface has been possible through the proposed analysis.

The pullout capacity of the vertical anchor plate in cohesion-less soil for the basic case is estimated with the interpretation of all the equilibrium equations and a closer agreement with the available experimental results vis-à-vis existing theoretical/empirical solutions is obtained.

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Notations

B	= anchor length
H	= depth of anchor embedment
h	= height of anchor plate
T_u	= ultimate pullout capacity of vertical plate (strip) anchor per unit width
γ	= unit weight of soil
H/h	= embedment ratio
γ_p	= unit weight of plate material