CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Studies on the behaviour of anchors against uplift force were started way back in the 1960s. Initially the pullout capacity of anchors was predicted from the test results on anchor for transmission line towers (Giffels et al 1960; Ireland 1963; Adams and Hayes 1967). Later on the prediction were based on the results on 1g model and centrifuge model studies. Over a period of the last four decades research work in this area was carried out extensively, in order to understand the real mechanism of failure of anchors against uplift force. Numerous theories were developed, mostly based on limit equilibrium method by using assumed / observed failure surfaces. With the vast knowledge on the uplift capacity of anchor, researchers have concentrated mainly on improving the uplift capacity of the anchors. Although several attempts have been made for evolving a method for improving uplift capacity of the anchor, the use of geosynthetics material in the field of geotechnical engineering showed a new approach in enhancing the uplift capacity. However the works reported on improving the uplift capacity of the anchors using reinforcement in soil bed are limited and little is known about their failure mechanism.

In this chapter, the literature is reviewed in the following sequence.

Studies on plate anchors embedded in sand bed

Studies on plate anchors embedded in reinforced sand bed
Studies on plate anchors embedded in submerged sand bed

Studies on behaviour of anchor under cyclic load

2.2 STUDIES ON PLATE ANCHORS EMBEDDED IN SAND BED

Varieties of anchors are developed and deployed in the field to meet the needs of growing anchorage problems. Scanning through the various types of anchors and the field applications, the present study focuses on plate anchors, which are considered to be one of the most popular varieties of anchors, which are extensively used in various types of onshore and offshore construction and maintenance works. They represent an economically better alternative to gravity and other embedded anchors for resisting uplift forces (Bouazza and Finlay 1990). Many researchers have worked with the anchors subjected to uplift of plate anchors buried both in cohesionless and cohesive soils. The works carried out to predict the uplift behaviour of anchors in cohesionless soils under monotonic and cyclic loading conditions are being reviewed.

Uplift theories are generally based on assumed failure surfaces. Marston (1930) developed the first rational approach of loads on buried conduits by assuming a vertical slip surface having a width equal to that of the diameter of the pipe embedded. The pullout capacity was computed by considering the frictional resistance along this surface and the weight of soil bounded within the failure surfaces. However, his analysis exhibited the behaviour entirely different from the experimental results as well as from the other investigators (ex: Trautmann et al 1985).
Majer (1955) used a simple failure mechanism to estimate the uplift capacity of footing. In this method, a cylindrical failure surface (Figure 2.1) having a diameter equal to that of the anchor width was assumed and the pullout capacity was computed by considering the weight of soil within the failure zone and the friction developed along the failure surface.

\[ P = \frac{1}{2}(K\gamma H^2) \]

\[ \gamma \]

\[ P_u \]

\[ D \]

\[ H \]

\[ W \]

\[ P = \frac{1}{2} (K \gamma H^2) \]

\[ \gamma \]

Figure 2.1 Assumed failure mechanism by Majer (1955)

Mors (1959) assumed a truncated cone extending from the base of the anchor up to ground level. The pullout capacity of anchor was assumed to be equal to the weight of the soil mass within the truncated cone (Figure 2.2). Both the above methods did not differentiate between shallow and deep anchor failures. The cone method suggested by Mors was found to be conservative for shallow anchors but quite opposite at larger depths (Turner 1962).
Balla (1961) developed an analysis to estimate the breakout resistance of foundation for pylons supporting electrical power lines. From the laboratory model tests, the author found that the failure surface resembles an arc of a circle having radius \( r = (H-h') / \sin (45 - \phi/2) \), with its tangent making 90º with the horizontal at the base and (45-\( \phi \)/2) at the ground level as shown in Figure 2.3, where \( H \) is the depth of embedment and \( h' \) is the thickness of anchor. Baker and Kondner (1966) based on their experiments on circular plate anchors in dense (\( \phi = 42º \)) Ottawa sand concluded that for \( H/D<6 \) (\( D = \) diameter of anchor), the appearance of circular failure surface is distinct. For deep anchors (\( H/D>6 \)), the failure surface is different and the analysis of Balla (1961) could be used for anchors of \( H/D \) ratio less than 6. Sutherland (1965) found that the mode of failure varied also with the density of sand and showed that Balla’s analytical approach estimated pullout capacity reasonably well in sands of loose to medium density.
Meyerhof and Adams (1968) conducted several laboratory tests to study the uplift behavior of circular and strip plate anchors in loose and dense sand beds. It has been emphasized that there exists a critical embedment depth after which the breakout factor remains constant. The behaviour of both dense and loose uniform sand was observed in semi-spatial using time exposure photographs. In dense sand at shallow depth, a distinct slip surface occurs which extends in a shallow arc from the anchor edge to the surface. At greater depth the failure surface is less distinct being initially curved and then essentially vertical and extending to the surface. In the loose sand, at larger depth, the failure surface was essentially vertical and limited to a short distance above the anchor whereas at shallow depth the failure surface was again essentially vertical but extending to the ground surface. Figure 2.4 illustrates the two distinct modes of failure depending on the depth of embedment, namely shallow and deep embedment. For shallow embedment the average angle of failure surface with the vertical, in sands, for strip anchor varies between about $\phi/3$ and $2\phi/3$. Equations were proposed to estimate the total uplift capacity of anchors in c-$\phi$ soil for shallow and deep embedment.
Figure 2.4 Shallow and Deep anchor under uplift load assumed by Meyerhof and Adams (1968)

For shallow embedment (where the failure surface reaches ground surface)

\[ P_{pu} = 2cH + \gamma H^2 K_p \tan \delta + W \]  \hspace{1cm} (2.1)

For deep embedment (where the failure surface does not reach ground surface)

\[ P_{pu} = 2cH_c + \gamma (2H - H_c)H_c K_u \tan \phi + W \]  \hspace{1cm} (2.2)

where  
- \( P_{pu} \) - ultimate uplift capacity  
- \( H_c \) - critical embedded depth  
- \( H \) - total embedded depth  
- \( c \) - cohesion along the vertical plane through footing edge  
- \( \gamma \) - density of back fill  
- \( K_p \) - coefficient of passive earth pressure  
- \( \delta \) - angle of total passive earth pressure component in horizontal
W - weight of lifted soil mass
\( \phi \) - angle of internal friction
\( K_u \) - nominal uplift coefficient of earth pressure on vertical plane through footing edge

Vesic (1971) computed the uplift capacity of embedded objects using the solution of expansion of cavities of a semi-infinite rigid plastic body. The uplift capacity of the embedded objects was equated to the ultimate radial pressure, \( q_o \) needed to break out a cylindrical or a spherical cavity. Figure 2.5 shows the assumed failure surface considered in the study.

\[
q_o = c \bar{F}_c + \gamma H \bar{F}_q
\]  

(2.3)

where
\( \gamma \) - unit weight of soil medium
\( H \) - depth of embedment
\( c \) - cohesion
\( \bar{F}_c \) and \( \bar{F}_q \) - cavity breakout factors (depends on shape, relative density and the angle of shearing resistance of soil)

Figure 2.5 Expanding spherical cavity assumed by Vesic (1971)
For deep anchors Vesic (1971) equated limiting cavity pressure to the uplift resistance and proposed the following equation.

\[ p_{pu} = cF_c + \gamma HF_q \]  

(2.4)

where \( p_{pu} \) - ultimate pressure for expansion of a deep cylindrical cavity

\( F_c \) and \( F_q \) - cylindrical cavity factors

Vesic also compared his theoretical solutions with the experimental results of various researchers and found that the breakout factors increased with depth only at shallow depths. For each soil there was a critical relative depth beyond which the anchor plate behaved as a deep anchor.

Das and Seeley (1975a, b) conducted laboratory model tests on loose sand bed (\( \gamma=15.1 \text{ kN/m}^3 \) and \( \phi=31^\circ \)) using aluminum plate anchors for various length to width ratios. Variation of breakout factor and critical embedment depth with length to width ratio of plates were studied. The critical embedment ratio of rectangular plate was higher than that of square plate and also it increased with the aspect ratio of the plate.

Rowe and Davis (1982) studied the behaviour of anchor plates using an elasto-plastic finite element analysis, where the influence of soil dilatancy (\( R_q \)), initial stress state (\( R_k \)) and anchor roughness (\( R_R \)) were studied. Effect of anchor embedment, angle of shearing resistance and the roughness of anchor base were also investigated in this study. The anchor plate was assumed to be perfectly rigid and weightless. The soil was assumed to satisfy the Mohr-Coulomb failure criterion and either an associated flow rule or a non-associated flow rule. Analysis involving straight rupture
surfaces at an inclination to the vertical equal to the dilatancy angle ($\psi$) was proposed by Vermeer and Sutjiadi (1985).

Trautmann et al (1985) conducted tests on steel pipe having an outside diameter of 102 mm and a length of 1.2 m embedded in sand bed, with a view to study

(i) maximum uplift resistance as a function of pipe depth and soil density
(ii) variation of load – displacement behaviour in terms of density and embedment ratio.
(iii) the force-displacement behaviour as a mathematical function
(iv) the comparison of observed results with the previously published research works.

Based on the experimental findings, they concluded that the force-displacement relationship can be modeled by a rectangular hyperbola as examined by Das and Seeley (1975b). The hyperbolic relationship is given by

$$F^* = \frac{Z^*}{(0.07+0.93Z^*)}$$

where $F^*$ - normalized force ($P_u/\gamma \text{ HDL})/N_q$

$N_q$ - breakout factor ($P_{pu}/\gamma \text{ HDL}$)

$Z^*$ - normalised displacement = ($\delta$/D)/($\delta^*/D$)

$\delta$ - displacement

$\delta^*$ - displacement at peak load.

$P_u$ - force measured at each increment of displacement

$P_{pu}$ - peak load

$\gamma$ - unit weight of soil

$H$ - depth of embedment
D - diameter of pipe
L - length of pipe

Using this relationship and appropriate values of $N_q$ and $(\delta^*/D)$, a force displacement relationship can be constructed for any combination of pipe diameter, depth and density.

Murray and Geddes (1987) examined the pullout resistance of anchor plates in dense and medium dense, dry cohesionless sands through laboratory tests on 50.8 mm wide rectangular anchor with $L/B$ ratio between 1 and 10 and circular anchors of 50.8 and 76.2 mm diameter. The effect of size, shape, depth of embedment and roughness of plate and the density of sand were studied. The results of laboratory tests were compared with theoretical solutions developed based on limit equilibrium for strip and circular anchors. Figure 2.6a illustrates the failure mechanism considered for the limit equilibrium analysis and Figure 2.6b describes the mechanism considered in limit analysis. On the basis of limit equilibrium approach, they developed the following equations for strip and circular anchors.

\[ W_{bd} \quad \text{weight of soil vertically above the anchor plate} \]
\[ W_w \quad \text{weight of soil contained in wedge } abc \]

**Figure 2.6a Definition of parameters in limit equilibrium analysis**

*(Murray and Geddes 1987)*
Figure 2.6b  Limit analysis solution (Murray and Geddes 1987)

For strip anchor,

$$\frac{P_{pu}}{\gamma BH} = 1 + \frac{H}{B} (\sin \phi + \sin(\phi/2))$$

(2.6)

For Circular anchor,

$$\frac{P_{pu}}{\gamma AH} = 1 + \frac{2H}{D} \left( \sin \phi + \sin \left( \frac{\phi}{2} \right) \right) \left( 1 + \frac{2}{3} \frac{H}{D} \tan \left( \frac{\phi}{2} \right) \right) \left( 2 - \sin \phi \right)$$

(2.7)

where  \( P_{pu} \) - ultimate uplift resistance of circular anchor or ultimate uplift resistance per unit length of strip anchor

\( A \) - plan area of circular anchor
\( B \) - width of rectangular anchor
\( D \) - diameter of circular anchor
\( H \) - depth of embedment
\( \phi \) - angle of shearing resistance
\( \gamma \) - unit weight of soil.

The ultimate uplift resistance of a strip anchor based on limit analysis was represented by the following non-dimensional relation.
\[
\frac{P_{pu}}{\gamma BH} = 1 + \frac{H}{B} \frac{\tan \theta \tan \delta_w}{\tan \delta_w - \tan(\phi - \alpha)}
\] (2.8)

where \( \phi \geq \delta_w \geq (\phi - \alpha) \geq 0 \)

- \( P_{pu} \) - the ultimate uplift resistance per unit length of plate
- \( \gamma \) - unit weight of soil bed
- \( B \) - breadth of anchor plate
- \( H \) - depth of embedment
- \( \phi \) - angle made by the force \( F \), to the normal of the failure plane
- \( \alpha \) - angle made by the tangent to the vertical
- \( \delta_w \) - inclination of the resultant forces \( W_w \) and \( F \) to the horizontal in Figure 2.6a

In practice, average value of \( \theta \) and \( \alpha \) taken as \( \phi/2 \) and \( \delta_w \) as \( 3\phi/4 \) and can be rewritten as

\[
\frac{P_{pu}}{\gamma BH} = 1 + \frac{H}{B} \left( \sin \phi + \sin \frac{\phi}{2} \right)
\] (2.9)

Lower bound solution fetched a poor lower bound value, since the yield condition cannot be violated anywhere and equilibrium should be maintained throughout the soil mass. Equating the work done by the external forces to the dissipation of energy, for cohesionless media, the following upper bound solution was obtained

\[
\frac{P_{pu}}{\gamma BH} = 1 + \frac{H}{B} \left( \frac{\tan(\phi + \omega)\tan(\beta - \omega) - \tan(\beta - \omega - \phi)}{\tan(\phi + \omega) + \tan(\beta - \omega - \phi)} \right)
\] (2.10)
The theoretical solution for strip anchors yields greater value with the experimental results, due to its inability to describe the stress history or degree of over consolidation during sand packing.

Tagaya et al (1988) determined the uplift capacity of plate anchor in medium to dense sand bed based on theory of plasticity and compared with the existing theories. The ultimate pullout resistance on a two-dimensional (strip) anchor buried at shallow depths in sand at any angle was expressed as

\[ P_{pu} = \frac{1}{2} \gamma H^2 \frac{K_b A}{B} + (W_s + W_A) \cos \alpha \]  \hspace{1cm} (2.11)

where
- \( P_{pu} \) - ultimate pullout resistance of shallow anchor
- \( \gamma \) - effective unit weight of soil
- \( H \) - maximum depth of anchor
- \( B \) - width of anchor
- \( K_b \) - uplift coefficient of shallow anchor
- \( A \) - area of anchor
- \( W_s \) - effective weight of soil above anchor
- \( W_A \) - effective weight of anchor
- \( \alpha \) - inclined angle of anchor

The failure mechanism of a deep horizontal anchor was very similar to that of the point bearing capacity of a pile, as shown in Figure 2.7. Concept of cavity expansion was used to analyse the point bearing of the anchor. The ultimate pullout resistance was determined as:

\[ P_{pu} = \sigma' \tan^2 \left( \frac{\pi}{4} + \frac{\phi}{2} \right) \exp \left[ \left( \frac{\pi}{2} - \theta \right) \tan \phi \right] \]  \hspace{1cm} (2.12)

where
- \( P_{pu} \) - peak pullout resistance of deep anchor
- \( \sigma' = \frac{q F_q}{4} \)
$\bar{q}$ - mean effective ground stress of the plastic zone III

$F_{q}$ - cavity expansion factor (Vesic 1971)

$\phi$ - angle of shearing resistance

It was found that the value of breakout factor was increasing linearly in the case of shallow embedment and embedment depth more than the critical embedment depth, the breakout factor was almost constant. The shape effect of the rectangular anchor was established, to arrive at its capacity from the capacity of strip anchor.

Figure 2.7 Physical plane of a rupture of deep anchor (Tagaya et al 1988)

Dickin (1988) investigated the uplift capacity of a one metre wide strip anchor in cohesionless soil using centrifuge at the Liverpool University on 25mm model anchor plates with aspect ratios $L/D = 1, 2, 5$ and 8 at embedment ratios upto 8 in both loose and dense sands by subjecting the model to centrifugal acceleration of 40g, where $g =$ acceleration due to gravity, enabling the behaviour of one metre wide prototype anchor to be
investigated. On the basis of the experimental results, Dickin (1988) concluded that,

i. the breakout factor increases significantly with anchor embedment depth and soil density but decreases with increased aspect ratios.

ii. displacement corresponding to failure also increases with embedment but reduces with increased relative density.

iii. the shape factor is relatively insensitive to anchor size but increases with both embedment ratio and soil density.

Frydman and Shaham (1989) conducted experiments to find out the uplift capacity of horizontal rectangular slab anchor in sand and proposed the following equations.

For dense sand,

$$P_{pu} = \left(1 + \frac{H}{B} \tan \phi \right) \left(1 + \frac{(B/L)-0.15}{(1-0.15)} \right) \left(0.51 + 2.35 \log \left(\frac{H}{B}\right)\right)$$  \hspace{1cm} (2.13)

For loose sand

$$P_{pu} = \left(1 + \frac{H}{B} \tan \phi \right) \left(1 + 0.5 \left(\frac{(B/L)-0.15}{(1-0.15)} \right)\right)$$  \hspace{1cm} (2.14)

where

\begin{align*}
P_{pu} & \quad \text{uplift capacity} \\
\gamma & \quad \text{unit weight of soil} \\
H & \quad \text{depth of embedment} \\
B & \quad \text{width of the slab} \\
L & \quad \text{length of the slab} \\
\phi & \quad \text{angle of shearing resistance}
\end{align*}
Ilamparuthi (1991) carried out detailed experimental investigation on plate anchor of three different shapes embedded in cohesionless soil. Based on the failure mechanism observed from experiments conducted on both shallow and deep anchors, the following equations to compute the pullout capacities were proposed.

**Shallow anchor**

\[
P_{pu} = \frac{\pi \gamma H}{12} \left[ D^2 + D(D + 2H \tan \theta) + (D + 2H \tan \theta)^2 \right] + \frac{\pi \gamma H^2}{2} (D + \frac{2H}{\tan \theta}) (\sin \theta + K \cos \theta) \tan \phi \]

(2.15)

where  
- \(P_{pu}\) - uplift capacity  
- \(\theta\) - \(\phi/2\)  
- \(K\) - \(1/\tan(3\phi/2)\)  
- \(\gamma\) - unit weight of soil  
- \(H\) - depth of embedment  
- \(D\) - diameter of circular anchor  
- \(\phi\) - angle of shearing resistance.

**Deep anchor**

\[
P_{pu} = \frac{\pi \gamma x D}{12} \left[ D^2 + D(D + 2x \tan \theta) + (D + 2x \tan \theta)^2 \right] + \frac{\pi}{4} (D + 2x \tan \theta)^2 \gamma (H - xD) \]

\[+ \pi \gamma \left( H \frac{x D}{2} \right) x D \left( D + \frac{2}{3} x \tan \theta \right) (\sin \theta + K_p \cos \theta) \tan \phi \]

(2.16)

where  
- \(\theta\) - 0.8\(\phi\)  
- \(K_p\) - passive earth pressure co-efficient  
- \(x\) - value depends on the density and depth of embedment of anchor.
Subbarao and Jyanth Kumar (1994) developed a theory using the method of characteristics coupled with a log-spiral failure surface for determining the vertical uplift capacity of shallow strip anchors in c-\(\phi\) soil. Figure 2.8 demonstrates the failure mechanism during uplift as assumed by Subbarao and Jyanth Kumar (1994). Uplift capacity factors \(F_c\), \(F_q\) and \(F_\gamma\) have been established as a function of embedment ratio and angle of internal friction. The theory proposed was found to predict pullout capacity accurately in loose and medium dense sand, but it was found to be conservative in case of dense and very dense sand.

The ultimate pullout load, \(P_{pu}\) per unit length of the strip anchor is written as

\[
P_{pu} \cdot W_R = c F_c B + q F_q B + 0.5 \gamma B^2 F_\gamma
\]

(2.17)

where

- \(P_u\) - ultimate pullout resistance of anchor
- \(c\) - cohesion of soil mass
- \(W_R\) - weight of rectangular portion of the soil mass over the anchor plate
- \(\gamma\) - unit weight of soil mass
B - width of anchor

$F_c, F_q, F_\gamma$ - uplift capacity factors corresponding to cohesion, surcharge and unit weight respectively

Jyanth Kumar (2004) proposed solution for finding the uplift capacity of anchors embedded in associated and non-associated flow mass. The analysis involved the calculation of uplift resistance from the static equilibrium and theory of energy balance. It was found that both the static equilibrium and energy balance were found to give same results. The limitation of the theory is that it is valid only for shallow anchors, where the rupture surface extends upto the ground surface.

Subbarao and Manjunatha (2004) proposed a theory based on method of characteristics for estimating the uplift capacity of circular and strip anchors. The theory was derived based on the assumption that 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. It was assumed that the soil at each and every point on the failure surface follows Mohr-Coulomb failure criteria. The proposed theory was validated with other available theories and it was found that the prediction from the present theory matched well with earlier methods for higher embedment ratios, while it underestimated the pullout capacity for lower embedment ratios.

Dickin and Laman (2007) studied the physical and computational investigation on uplift response of strip anchors in sand. In this study the anchors used for centrifuge tests were 25mm wide, 3mm thick stainless steel model anchor plates with aspect ratios, $L/B = 1, 2, 5$ and $8$ at embedment ratios $(H/B)$ from $1$ to $8$ in both dense and loose sand beds. The model anchor plates were subjected to acceleration of $40 \text{ g}$ in flight, and therefore
experienced stress levels similar to those around 1m wide prototype anchors. Separate tests were carried out at each embedment ratio to determine the pullout load of the tie rod alone to made correction in the total uplift load. However the correction due to tie rod was generally less than 5% of the total uplift load. Numerical analysis was performed on centrifuge model tests conducted using PLAXIS finite element code and the failure mechanism of both shallow and deep anchors was compared with the failure pattern observed by Ilamparuthi et al (2002).

From the study the authors concluded the following:

Both the physical and computational studies show that breakout factors for 1 m wide strip anchors increase with anchor embedment ratio and sand packing.

Agreement between breakout factors from the centrifuge tests and PLAXIS results based on a computed displacement of 0.2 m is excellent for anchors up to embedment ratio of 6. Some divergence occurs for deeper cases.

To facilitate the determination of uplift capacity simple expressions are presented by various investigators which are summarized in Table 2.1.

Scanning through the literature review it is observed that attempts were made by several researchers to understand the behaviour of plate anchors and failure surfaces in the determination of uplift capacities. In a similar way the peak pullout load is expressed in terms of a dimensionless parameter called the breakout factor. The methodology adopted to formulate the equation for determination of the breakout factor by various researchers is summarised in Table 2.1.
Table 2.1 Summary of Design Methods proposed by various Researchers

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Method of analysis</th>
<th>Formulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Majer (1955)</td>
<td>Vertical slip surface</td>
<td>$N_u = 1 + 2K (H/D) \tan \phi$</td>
</tr>
<tr>
<td>Balla (1961)</td>
<td>Tangential-curve slip surface</td>
<td>$N_u = (F_1 + F_3) (4/\pi) (H/D)^2$ where $F_1$ and $F_3$ are dependent on $\phi$ and $\gamma$ and obtained from chart provided by the author</td>
</tr>
<tr>
<td>Meyerhof and Adams (1968)</td>
<td>Pyramidal – shaped slip surface</td>
<td>$N_u = 2 (H/D) K_u^1 \tan \phi S_f + 1$ where $K_u^1 = 0.95$ for $30^\circ &lt; \phi &lt; 45^\circ$ and $S_f$ is the shape factor dependent on $\phi$.</td>
</tr>
<tr>
<td>Vesic (1971)</td>
<td>Cavity-expansion model</td>
<td>Chart of $N_u$ in terms of $\phi$ and H/D is provided in Das and Yang (1987)</td>
</tr>
<tr>
<td>Clemence and Veesaert (1977)</td>
<td>Inverted cone slip surface; cone angle with vertical $= \phi / 2$</td>
<td>$N_u = [1 + (H/D)\tan(\phi / 2)]^2 + 4K \tan \phi \cos^2(\phi / 2) [1/2(H/D)^2 + 1/3 (H/D)^2 \tan (\phi/2)]$</td>
</tr>
<tr>
<td>Andreadis et al (1981)</td>
<td>Derived from model and field tests on anchor</td>
<td>Chart of $N_u$ in terms of $\phi$ and H/D provided by the authors</td>
</tr>
<tr>
<td>Rowe and Davis (1982)</td>
<td>Finite element analysis giving uplift capacity of strip anchors</td>
<td>$N_u = F_\gamma R_\psi R_k R_R$ where $F_\gamma$, $R_\psi$ is a function of $\psi$ and $H/D$. $R_k$ and $R_R$ may be taken from charts provided by the authors; $N_u$ for a circular anchor may be obtained by applying suitable shape factor $S_f$ (eg. Dickin 1983)</td>
</tr>
<tr>
<td>Murray and Geddes (1987)</td>
<td>Inverted cone slip surface; cone angle $= \phi$</td>
<td>Strip anchor $N_u = 1 + (H/B) \tan \phi$</td>
</tr>
<tr>
<td>Researcher</td>
<td>Method of analysis</td>
<td>Formulation</td>
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</table>
| Ghaly et al (1991a)| Inverted cone slip surface | \[ N_u = \frac{4}{3} D^2 \left[ \left( D+H \tan \theta \right)^2 \right] + \frac{2HK_1^1 / D^2}{(D+L \tan \theta / \cos \theta) \times \tan \delta^1}, \]  
|                    |                        | where \( K_1^1 = \frac{(1+\sin \delta^1)}{1-\sin \delta^1} \)  
|                    |                        | \( \delta^1 = \text{Average mobilized angle of shearing resistance} \)  
|                    |                        | \( k_1^1 = \text{modified earth pressure coefficient} \)  
| Ilamparuthi (1991) | Truncated frustum of cone | For shallow anchor  
|                    |                        | \[ N_u = \frac{1}{3} \left[ 1+(1+\frac{2H}{D} \tan \theta)+(1+\frac{2H}{D} \tan \theta)^3 \right] + \frac{2H}{D} (1+\frac{2H}{3D} \tan \theta) (\sin \theta + K \cos \theta) \tan \phi \]  
|                    |                        | where \( \theta = \phi / 2 \)  
|                    |                        | \( K = 1 / \tan(3\phi/2) \)  

where \( N_u \) - breakout factor; \( H \) - embedment depth  
\( D \) - diameter of anchor  
\( \phi \) - peak friction angle  
\( \psi \) - dilatancy angle  
\( \gamma \) - unit weight of soil  
\( K \) - coefficient of lateral earth pressure  
\( I_D \) - relative density

### 2.3 STUDIES ON PLATE ANCHORS EMBEDDED IN REINFORCED SAND BED

The capacity of anchors embedded in soil against uplift is equal to the sum of the weight of soil within the failure zone and frictional resistance along the failure surface. In some cases the capacity of anchor of given size is inadequate to develop required capacity. Increase in size and depths of embedment of anchor are possible alternative methods to improve the uplift capacity but the capacity is not in proportion to the increased size of anchor. The increasing use of anchors to resist uplift forces of higher magnitude
necessitated the development of suitable and economical alternative techniques. However reported research work on the enhancement of pullout capacity of buried anchors is very limited. In the last few years, some researchers have attempted to enhance the pullout capacity of anchors by the inclusion of geosynthetic reinforcement in sand (Subbarao et al 1988; Selvadurai 1989, 1993; Krishnaswamy and Parashar 1991, 1994; Ilamparuthi and Dickin 2001a,b and Swami Saran and Rao 2002). These works are reviewed in this section.

Subbarao et al (1988) reported the improvement in pullout capacity by using geotextiles as ties to anchor embedded in sand. Tests were conducted on reinforced concrete model anchors of cylindrical and belled shapes. Polypropylene ties of width 55 mm and of thickness 0.72 mm were used. Ties for cylindrical and belled anchors were 650 mm and 350 mm in length respectively, and were connected at several levels using steel clamps. At each level six strips were connected to the model and spread radially with an angle of 60° between them. Results indicated that the anchors with geotextile ties offered greater uplift resistance than those without ties. Furthermore, the provision of a single layer of ties close to the anchor was reported to be more effective than the use of multiple layers.

Selvadurai (1989) studied the performance of uplift resistance of buried pipelines by the use of geogrid in sand bed. Experiments were made on 150 mm diameter and 850 mm length pipe section. The rate of loading was maintained approximately at 0.02 mm/sec. It was shown that the judicious incorporation of geogrids can lead to a substantial increase in uplift capacity of the buried pipeline section. The increase in uplift capacity of the reinforced system can be of the order of 100% when considering the peak loads and nearly 250% on ultimate load capacity at large displacement. Further, it is emphasised that the ductility of the system can also be increased. It is
foreseen that as the diameter of the pipe increases, the contribution from the geogrid will represent a smaller fraction of the uplift capacity of the reinforced system, the maximum capacity of the geogrid will be the governing factor. In order to determine the contribution towards the uplift resistance due to the inclusion of geogrid, the author developed an equation using the theory of Meyerhof and Adams (1968) as modified by Das and Seeley (1975a,b). The author considered that pipeline uplift capacity comprised two additive components, the ultimate capacity of the pipe section in unreinforced soil, and the limiting resistance generated by the geogrid reinforcement against pullout at the soil/geogrid interface. Assuming no construction effects, these components were defined by the following equations:

\[ P_{pr} = \gamma' D_p d_p L_p \left\{ \left( D_p/d_p \right) K_u \tan \phi \left[ \left( 2m \left( D_p/d_p \right) + 1 \right) \left( d_p/L_p \right) + 1 \right] + 1 \right\} \]  \hspace{1cm} (2.18)

\[ P_r = \gamma' L_p \left( H^2 - D_o^2 \right) \left[ 2 - \sin \phi \left( 1 - \cos^2 \alpha \right) \right] \tan \delta' \]  \hspace{1cm} (2.19)

where

- \( P_{pr} \) - ultimate capacity of pipe section in reinforced soil
- \( P_r \) - limiting resistance generated by the reinforcement
- \( D_p \) - depth of embedment to the top level of the pipe
- \( d_p \) - pipe diameter
- \( L_p \) - length of pipe section
- \( K_u \) - uplift coefficient
- \( m \) - Meyerhof and Adams’ coefficient
- \( H \) - maximum depth of geogrid,
- \( D_o \) - the depth to the underside of the pipe
- \( \alpha \) - geogrid inclination to the vertical
- \( \gamma' \) - effective unit weight of soil
- \( \delta' \) - effective angle of friction at the soil/geogrid interface.
Even though equation 2.19 is very approximate and thus has limitations in predicting the uplift contribution of the geogrid inclusion, the mathematical approach adopted provides a basis upon which the resistance due to reinforcement might be determined. However, in subsequent investigations, (Selvadurai 1993), involving the uplift behaviour of CONWED STRATAGRID anchored pipelines in granular soil, Equation 2.19 was not validated against experimental findings.

Selvadurai (1993) extended the research on anchored pipeline by using conwed strata grid as reinforcement in granular soil bed. The pipe used in those series of tests was 215 mm in diameter and 1610 mm in length. The controlled movement was achieved by a servo-controlled hydraulic actuator which exerts a controlled rate of uplift movement. The increase in the peak uplift loads was of the order of 80% in both fine grained and coarse grained granular soils. One of the most important observations reported based on the experiments was that the reinforcement maintained increased pullout load with increase in displacement which was in contrast to the performance of unreinforced system, wherein significant reduction was observed beyond the peak load.

Krishnaswamy and Parashar (1994) studied the uplift behaviour of plate anchors embedded in cohesive and cohesionless soil medium, with and without geosynthetics. The type of geosynthetics, the ratio of the area of geosynthetics inclusion to the area of the plate anchor, the depth of embedment, the type of soil, the strain rate and the position of the water table were found to have significant effects on the uplift behaviour of the plate anchors. Four different configurations of the geosynthetics inclusion were conducted as shown in Figure 2.9 and placing the geosynthetics directly on the top (configuration C) of the metallic anchor was proved to be beneficial in achieving maximum increase in the uplift capacity. It has been further
documented that use of wider mesh opening did not result in substantial increase in the uplift capacity of anchor since effective surface contact area between geogrid and sand media was lesser. Incase of the deep plate anchors the geosynthetics inclusion changes the failure pattern from local failure to general failure and the failure surface reaches the ground surface. Also, geosynthetics layer increases the effective area of anchorage.

**Figure 2.9 Different configuration of the geosynthetics inclusion (Krishnaswamy and Parashar 1994)**

Ilamparuthi and Dickin (2001a) studied the behaviour of belled pile to the pullout load in the reinforced sand bed. Tests were conducted in a square tank with inner dimensions 760 mm × 760 mm × 900 mm. Model piles with diameters 60, 80 and 100 mm were reinforced with the NETLON geogrid (CE-121). The tests were displacement controlled at a constant rate of 1.25mm/min by placing the base of the pile in the geogrid reinforced gravel layer. Five different arrangement of reinforcement as shown in the Figure 2.10 were tested by the authors.
The variation of pullout load with displacement for the various arrangements of reinforcement are compared with the test results for models in unreinforced sand in Figure 2.11. The load displacement behaviour of the model embedded in a gravel layer within the sand was similar to that for one embedded in homogeneous sand. The inclusion of gravel layer increased the uplift resistance due to the higher interlocking friction and less rolling and sliding friction in gravel than in sand in the prepeak and residual phases. The load-displacement relationship of belled piles in reinforced sand at shallow depth was characterized by three-phase behaviour, namely

- Prepeak behaviour, which shows rapid increase in resistance.
- Post peak behaviour, which shows a slight gradual decrease in resistance with further displacement similar to strain softening behaviour.
- Post peak behaviour beyond a lower peak where either the resistance increases with displacement (Similar to strain hardening behaviour in mild steel) or is maintained without any appreciable reduction (as in ductile materials).
The higher uplift resistance obtained in the geogrid cell reinforcement was attributed to a combination of the following:

i. the larger contact area between the geogrid cell and the soil.

ii. the soil mass mobilized to withstand pullout is much higher than in the unreinforced condition.

iii. the frictional resistance along the periphery of geogrid cell.

iv. the dilation of the gravel particles within the geogrid cell during uplift will tend to increase the lateral confinement and hence the uplift capacity of the pile.

Moreover the computation of lateral extent of surface heave by considering the rupture surface is a plane surface and emerges from the pile base at angle of φ/2 is in agreement with the surface heave observed by visual judgment in the reinforced case. Further more, it was observed that the uplift capacity of the pile increased more than 7 times the unreinforced condition in shallow depth in loose sand and 25% in case of deeper belled piles in dense

**Figure 2.11 Comparison of uplift behaviour for various arrangements of reinforcement (Ilamparuthi and Dickin 2001b)**
sand. This can be made use of in foundations, which are designed to withstand large displacements, and those designed for the repetitive loading environment.

A more extensive study was conducted by Ilamparuthi and Dickin (2001b) and the breakout factors from a series of tests on model belled piles on various model geometries and sand densities were examined. Breakout factor, \( N_r \), for piles embedded in reinforced sand beds was initially computed using the well established equation

\[
N_r = \frac{P_{pr}}{\gamma AH} \quad (2.20)
\]

where
- \( N_r \) - breakout factor in reinforced sand bed
- \( P_{pr} \) - peak pullout load in reinforced sand bed
- \( \gamma \) - unit weight of sand bed
- \( A \) - area of anchor
- \( H \) - depth of embedment

However the breakout factors obtained from the above equation (2.20) for pile/geogrid reinforcement system decreases with an increase in the pile bell diameter irrespective of the sand packing. In view of complexity of the soil-geogrid cell interaction, breakout factors were computed from Majer’s simple theory. It was found that assuming a lateral earth pressure coefficient, \( K = 0.8(K_p)^{1/2} \), where \( K_p \) is the Rankine earth pressure coefficient, gave a good overall fit to the experimental data. Also the relationship between the pullout load and the displacement was modeled using hyperbolic transformation as,

\[
\frac{P}{P_{pr}} = \frac{\delta/\delta_t}{c_Q + m_Q(\delta/\delta_t)} \quad (2.21)
\]
where \( P \) - pull out load at displacement \( \delta \)
\( P_{pr} \) - peak pullout load at displacement \( \delta_r \)
\( c_Q \) - 0.09
\( m_Q \) - 0.91

This relationship could be used to obtain soil stiffness for nonlinear analyses of the uplift behaviour of geogrid-cell reinforced piles in sand using a piecewise linear technique.

Swami Saran and Rao (2002) performed tests on 100mm x 100mm square, and 100 mm dia circular anchor by introducing single and multiple layers of reinforcement at spacings of 0.25 \( B \) and 0.5 \( B \) at a constant reinforcement width of 3\( B \) and at embedment ratios of 2, 3 and 4. The reinforcement used was Geogrid (CE-121 HDPE) with a tensile strength of 7.7 kN/m. All the tests conducted were stress controlled. Based on the results, it was reported that the pullout capacity of anchor increased predominantly with the reinforcement placed just above the anchor plate. In the multi-layer reinforcement, the double layer system was efficient in increasing the pullout capacity of anchor. However, there was not a predominant increase in case of 3 layers of reinforcement. It was observed that the pullout capacity of the square anchor was higher than that of circular anchor, owing to the higher area contribution to resist the uplift in square plate than circular plate.

The literature reviewed here has indicated that research on uplift behaviour of anchors in reinforced sand bed has invited the attention of geotechnical engineers in the late ’90’s. However, the technique of enhancing pullout capacity by reinforcing the soil has not made expected breakthrough as that of bearing capacity in reinforced sand. The basic reason for this may be attributed to the complex interaction between the soil and reinforcement under tensile (uplift) load. Therefore, the mechanism of strip anchor in sand
reinforced with geogrids is studied through model tests and a relevant theory is derived in this work.

2.4 STUDIES ON ANCHORS IN SUBMERGED SAND

Limited work was reported on the effect of submergence under uplift loading. Vesic (1971), Andreadis et al (1981), Ghaly et al (1991b), Krishnaswamy and Parashar (1991), Ilamaparuthi (2004) are a few researchers who have contributed to this field. Krishnaswamy and Parashar (1994) performed the test in reinforced sand bed and others in unreinforced conditions.

Andreadis et al (1981) carried out tests on cylindrical, conical and plate anchors to determine the influence of anchor configuration and other factors involved on the ultimate static uplift capacity and response to different levels of static loading in Bushfarm sand and Borough green sand. A pullout rate of 0.5 mm/min was adopted in this study, which permitted free drainage around the anchor body during testing.

Selected testing rates were chosen to study the influence of pore water pressures under uplift condition. If the displacement of anchor is rapid in dense cohesionless beds, only a limited dissipation of induced negative pore pressure will take place resulting in partial drained conditions and therefore increased initial holding capacity. Maximum snap loads attained by the tested anchors were more than double their static drained capacities. As medium dense cohesionless soils dilate markedly less than the denser soils when subjected to shear stresses, increase in undrained holding capacity decreases with decreasing soil relative density. Loose and very loose sands develop positive pore pressures and the undrained holding capacities are reduced in comparison with the drained case as expected.
Haroon et al (1983) conducted uplift tests on model circular plate anchors of 60 mm, 80 mm and 100 mm diameter in sand having angle of shearing resistance equal to 30 degrees. The tests were conducted in both dry and submerged conditions for embedment ratios up to 4. They observed that the rupture surface starts from the top edge of the anchor and extends vertically up for the major portion of the depth of embedment, however, it gets curved near the surface of the sand mass. Based on these observations, they approximated the failure surface to a cylinder and proposed the following expression for pullout capacity in submerged sand.

\[
P'_{pu} = \left( \frac{\pi}{2} \right) \gamma'DH^2K_0 \tan\phi + \left( \frac{\pi}{4} \right) \gamma HD^2
\]

(2.22)

where

- \( P'_{pu} \) - peak pullout load in submerged condition
- \( \gamma' \) - submerged unit weight of soil
- \( \gamma_s \) - saturated unit weight of soil
- \( K_0 \) - co-efficient of earth pressure at rest (\( = 1 - \sin\phi \))
- \( D \) - diameter of anchor
- \( H \) - depth of embedment

Comparing with experimental results, they concluded that their theoretical estimates were lower than experimental pullout capacities and the maximum variation is 50%.

Ilamparuthi (1991) carried out model tests with circular plate anchors in submerged sand beds to study the effect of submergence. The equation proposed for estimating the pullout capacity is

\[
P'_{pu} = P_{pu} R_\gamma (\gamma'/\gamma)
\]

(2.23)
where \( P'_{pu} \) - Pullout capacity corresponding to submerged sand condition

\( P_{pu} \) - pullout capacity corresponding to dry sand condition

\( R_w \) - submergence factor

\( \gamma' \) - submerged unit weight

For \( \phi = 33.5^\circ \)

\[ R_w = 1.0 \text{ for } H/D \leq 4.8 \]

\[ R_w = \frac{21}{N_u} + 0.473 \left( 1 - \frac{21}{N_u} \right) \text{ for } 4.8 < H/D \leq 9.6 \]

For \( \phi = 43^\circ \)

\[ R_w = 1.127 \text{ for } H/D \leq 6.75 \]

\[ R_w = \frac{60}{N_u} + 0.473 \left( 1 - \frac{53.25}{N_u} \right) \text{ for } 6.75 < H/D \leq 9.6 \]

where \( N_u \) is break out factor in dry sand condition.

Based on the observation made by the researcher, it was reported that in the submerged condition, eventhough the angle of shearing resistance is essentially the same as for dry sand, the normal stress will be less due to the submerged unit weight of soil. Therefore, the shearing resistance, which is not only a function of \( \phi \), but also a function of the normal stress, will be less in the case of submerged sand bed. Thus, not only the effective weight of the soil inside the failure zone, but also the shearing resistance along the surface will be less in submerged condition. However, since the experimental results show breakout factors in submerged condition to be close to those in dry condition, the apparent gain of strength may be due to suction developed. As
the soil dilates with anchor movement, negative pore water pressure is developed, which creates suction force at the base of the anchor.

Ghaly et al (1991b) presented the behaviour of model single-pitch screw anchor installed in dense sand subjected to hydrostatic pressure or upward seepage flow. The test program included 20 experiments conducted on both shallow and deep anchors using screw anchors. For shallow anchors, the mode of failure propagates to the sand surface, and the weight of sand involved within the shape of failure represents a considerable portion of the total resistance against uplift. This weight of sand was strongly altered by the sand submersion, which results in a reduction of ultimate pullout load. In the case of deep anchors, the failure mechanism was of a local nature and the sand weight is a relatively small component in the total value of the uplift resistance, while the shearing resistance of the sand represents the major portion contributing to the total pullout load.

Taking into account that the shear resistance depends on the angle of shearing resistance of the sand and that the latter was only slightly altered by the submersion process, it was evident that the pullout load-upward displacement relationship for both dry and submerged sands is almost the same for deep anchors as reported by Vesic (1971). For a given installed depth and sand properties, the ultimate pullout load of screw anchors installed in dry sand was higher than that for anchors installed in submerged sand in the case of anchors embedded at shallow depths. For deep anchors, they are almost the same.

Krishnaswamy and Parashar (1991) studied the effect of rate of pullout and submergence in reinforced soil bed. Increase in the rate of pullout increases the uplift resistance of plate anchors. Reduction in the rate of pullout from 1.27 mm/min to 0.127 mm/min results in a reduction of ultimate
uplift resistance of about 16 to 18%. However, the percentage variations in the uplift capacity of plate anchors without geosynthetics as well as with geosynthetics are nearly the same over a given range of strain rate.

The submergence of soil anchor system results in a reduction of uplift capacity by approximately 30–40%. But, the ratio of the ultimate uplift capacity of anchors with geosynthetics inclusions to the ultimate uplift capacity of anchors without geosynthetics inclusions remains nearly the same for dry and submerged conditions. The effect of submergence on uplift capacity of plate anchors was reported in detail by Krishnaswamy and Parashar (1991).

Ilamparuthi and Muthukrishnaiah (2001) studied the behaviour of anchor subjected to snap loading in submerged sand bed. It was well known that under very fast loading, there may be apparent strength increase due to negative pore water pressure, but very little is known in a quantitative sense about the response of anchors embedded in submerged sand to the rate of loading under pullout. Tests were performed on rigid steel circular anchors of diameter 300 mm with a plate thickness of 8 mm under two rates of pull of 0.5 mm/sec and 1.5 mm/sec (snap loading). Variation in pore water pressure was measured at the base of anchor and other locations. The pore water pressures recorded at the base of anchor were negative for the rates of pull applied. The magnitude of negative pore water pressure was higher for higher rate of pull and maximum at the base of anchor for the depth of embedments investigated. Moreover the peak pullout loads were also higher for higher rate of pull. The magnitude of pore water pressure in high rate of pull (1.5 mm/sec) is about five times the slow rate of pull (0.5 mm/sec) for an embedment ratio of unity. The negative pore water pressure was quantified in terms of peak pullout load and was termed as force ratio. For a given rate of
pull the force ratio decreases with increase in embedment ratio which was found to be independent of density of sand bed.

2.5 STUDIES ON ANCHOR BEHAVIOUR UNDER CYCLIC LOAD

Literature reviewed shows that there is a dearth of published articles that describing the effect of cyclic loading in reinforced sand bed for uplift capacity of anchor. However limited studies on anchors subjected to cyclic loads reported in literature have been reviewed here.

Bemben et al (1973) and Bemben and Kupferman (1975) have presented results on the long term behaviour of fluke anchors in sandy and clayey soils subjected to cyclic loading. Fluke anchors of 76 mm and 152 mm diameter with projected plan area of 3950 mm$^2$ and 15730 mm$^2$ respectively were used in the investigation. Sinusoidal loading cycles of 8 second period with varying load amplitudes were used. Based on the model tests they suggested that fluke anchors to resist cyclic load should be designed at 40% of their static holding capacity.

Ponnaiah and Finlay (1983) reported the long-term behaviour of circular plate anchors subjected to sinusoidal loading cycles of 10 second period. All the tests were conducted with circular plate anchors of 50 mm diameter buried in normally consolidated clay at an embedment ratio of 4.5. Based on the test results it was reported that the anchors did not fail when load cycle upto $50 \pm 20\%$ of the drained ultimate pullout capacity. With recycling, it was found that the failure load increased to $70 \pm 20\%$ of the drained capacity of anchors. It was further observed that the anchor displacements were directly associated with the build-up and dissipation of pore water pressure during loading cycles.
Datta et al (1990) reported the short term cyclic behaviour of plate anchors in soft cohesive soil. These investigations were conducted with circular plate anchors of 50 mm diameter buried at a depth of 300 mm and subjected to a rectangular cyclic loading of 15 seconds time period for a maximum of 500 loading cycles. The principal parameters studied in this testing programme were the mean load and the cyclic amplitude and their influence on anchor movement and post-cyclic static pullout capacity. From the test results, it was concluded that the plate anchors should be designed for a load of one-third of its static pullout capacity to take into account the effects of cyclic loadings. Further it was pointed out that the movement of anchor under cyclic loading can be prevented if the maximum cyclic load level is kept below 25% of the static pullout capacity.

Byrne (2000) investigated the behaviour of suction anchor embedded in dense sand under monotonic and cyclic loads. It was reported that at small displacements, monotonic and cyclic load results showed little effect on loading rate. However at large displacement, particularly under tension the rate of loading affected the stiffness of the response and was dependent on pore fluid drainage. Further Byrne et al (2002) formulated a theoretical model namely, continuous hyper-plasticity and demonstrated its applicability to reproduce experimental data on suction anchor subjected to both monotonic and cyclic loads.

Singh and Ramaswamy (2002) studied the behaviour of plate anchors in soft saturated clay under monotonic and cyclic loads. Behaviour of plate anchors of 80mm dia in clay bed was carried out at an average testing water content of 57.3 % (I_c = 0.4) for various cyclic load ratios. Different combinations of sustained and cyclic loads were studied to determine the influence of anchor movement and post-cyclic monotonic pullout capacity.
The anchors were subjected to a maximum of 1000 cycles or till the movement of anchor was 80 mm.

Effect of preloading and the period of loading cycles on movement of anchor, and the period of loading cycles on post-cyclic pullout have been investigated. From the cyclic loading tests conducted, it was recommended that the design of plate anchors subjected to cyclic loading should be based on the allowable movement of structures rather than the breakout capacity of anchor. To prevent any substantial movement, the amplitude of cyclic loading should be kept below 30 % of the static anchor capacity.

From the literature reviewed on anchors against uplift load, it was found that the works reported on improving the uplift capacity of the anchors using reinforcement in sand bed are limited and the interaction behaviour between reinforcement and sand is not addressed adequately. In addition reported work on the behaviour of anchors in reinforced submerged sand bed under both monotonic and cyclic loading conditions is limited. Very few researchers have attempted to understand the behaviour of plate anchors under cyclic loading that too in cohesive soil. Hence a detailed study is essential in order to understand the failure mechanism of anchors embedded in reinforced sand bed, under monotonic and cyclic loads.