Uplift Capacity of Enlarged Base Piles in Sand

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ABSTRACT

Pile foundations, also known as substructure, are used to carry and transfer the load of the structure to the bearing ground located at some depth below ground surface. The main components of the foundation are the pile cap and the piles. Piles are long and slender members which transfer the load through the shallow soil of lower bearing capacity to deeper soil or rock of high bearing capacity. They are also used in normal soil condition to resist uplift forces or in poor soil condition to resist lateral forces. Most of the piles are made from three main types of material which are wood, steel and concrete. Piles are normally driven, drilled or jacked into the ground and connected to pile caps. Based on a few number of laboratory model results many investigators reported the uplift response of piles in cohesionless soil, a review of related previous works shows that not much research has been done to define the uplift capacity in cohesionless soil, a problem that is often encountered in field. The paper observed that the ultimate uplift capacity is dependent on the relative undrained/drained shear strength of cohesionless soil, the depth ratio of embedment and soil thickness ratio.

KEYWORDS: Piles, Driven, Drilled, Jacked, Sand, Uplift response, Failure mechanism

INTRODUCTION

Pile foundations are generally used to support compressive loads from superstructure. Some structures such as tall chimneys, transmission towers and jetty structures are subjected to overturning loads imposed by wind. In such cases, piles are required to resist uplift forces which are much greater than the weight of the structure itself. In addition to wind, marine structures and transmission line towers are hit by wave forces and line tension respectively. Straight shafted piles obtain their resistance to uplift through friction between the pile shaft and the surrounding soil. It may be increased by under-reaming or belling out the bottom of the piles.

The uplift capacity of a buried enlarged base or belled pile essentially come from the weight of soil within the failure zone above the base, the frictional resistance along the failure surface and the self-weight of the foundation. The required pullout resistance can be achieved by increasing sand density, base diameter of pile and the depth of embedment. The influence of these parameters on the uplift capacity in sand has been investigated by many researchers. However, studies regarding the influence of base angle on the uplift behaviour are limited. According to Joseph, E. Bowles, the ratio of the base diameter to the shaft diameter (D_b/D_s) should be no greater than 3.

Enlarged base piles are proven to be economical foundation type to provide resistance to both compressive and uplift load. The cost advantages of enlarged base piles are due to the reduced pile shaft diameter which results in less concrete needed to replace the excavated material. However, the construction of enlarged base pile exposes the construction workers to the danger of collapse of the excavation.

Contractors build enlarged base or bell out the bottom of the pile by using special belling buckets. The enlarged base is usually cut at 45 or 60 degree angle with maximum diameter of being not more than three times the diameter of the shaft.

PREVIOUS RESEARCH STUDIES CARRIED OUT ON UPLIFT BEHAVIOUR OF PILE

Large-scale field tests on anchors for transmission line towers conducted by Giffels et al. (1960), Ireland (1963) and Adams and Hayes (1967) were early investigations regarding the pullout resistance. In contrast, Majer (1955), Balla (1961), Downs and Chieurzzi (1966), Baker and Kondner (1966), Meyerhof and Adams (1968), Hanna and Carr (1971), Hanna and Sparks (1973), Das and Seeley (1975a, b), Clemence and Veesaert (1977), Andreadis et al. (1981), Sutherland et al. (1982), Murray and Geddes (1987), Ghaly et al. (1991a, b) employed 'Conventional' laboratory model studies at unit gravity to gain a better understanding of their behaviour. On the other hand, Ovesen (1981), Tagaya et al. (1983, 1988), Dickin (1988), Dickin and Leung (1990,1992) managed to provide data on anchors and belled piles subjected to stress levels experienced by full scale prototypes based on the development of the centrifugal modelling technique. Theoretical analyses include those of Vesic (1971) based on cavity expansion theory, limit equilibrium analyses of Chattopadhyay and Pise (1986), Saran et al. (1986) and elasto-plastic finite element analyses of Rowe and Davis (1982). Inevitably, these researches have resulted in many different design methods. Andreadis and Harvey (1981), who conducted a series of laboratory model and field tests on anchors, developed design charts to determine the uplift resistance of anchors. Meanwhile, Ovesen (1981) suggest a relationship for the uplift capacity of plate anchors based on centrifugal model tests. A series of laboratory model tests on anchors were conducted by Sutherland. From these tests, he concluded that the angle of inclination of the inverted cone slip surface with the vertical is a function of soil friction angle and relative density. By using finite-element analysis and elastic-plastic model, Rowe and Davis (1982) managed to determine the uplift capacity of horizontal strip anchors. Vermeer and Sutjiadi (1985) noticed that the angle of the inverted cone slip surface with the vertical is equal to the dilatancy angle ψ of the soil. In contrast of that, Murray and Geddes (1987) stated that the angle is equal to the soil friction angle. According to Dickin and Leung (1990,1992), increase in the angle of base and in diameter ratio result in a decrease in net uplift capacity and failure displacement.

Failure Mechanism

Many of the existing design methods of uplift capacity of enlarged base pile are based on results obtained from small laboratory model test on anchor foundation. Dickin (1988) have summarized a number of existing design method giving the uplift capacity of horizontal anchors in sand. He classified the assumed failure mechanism of most design methods into three categories which are vertical slip-surface model, inverted truncated cone model and curved slip-surface model. These three categories are shown in Figure 1. Majer (1955), one of the earliest researchers, assumed the failure mechanism is a vertical slip surface above the anchor. According to him, the uplift capacity is basically the total of weight of soil above the anchor and shear resistance along the perimeter of the vertical slip surface.

After that, Balla (1961) come up with different assumption that the slip surface above small model anchors was a tangential curve. According to Balla, the failure surface for shallow footings embedded in dense sand was nearly circular in elevation and the tangent to the surface of ground contact was at an angle of approximately $45^0 - \phi/2$ to the horizontal. He obtained a reasonable correlation between theory and the results of full scale tests on shallow footings by assuming a circular failure path.

Mac Donald (1963), who had done model test in sand, showed that the failure surface was approximately parabolic for shallow depths and the failure plane was approximately vertical for greater depths. The diameter of the cylinder formed was about 1.75 times the base diameter of the footing. The theory developed by Mac Donald (1963) assumed failure for the shallow case is conical with angle of inclination equal to half the angle of internal friction. Meanwhile, failure for the deep case was assumed to be cylindrical with a cylinder diameter of 1.75 times the base diameter.

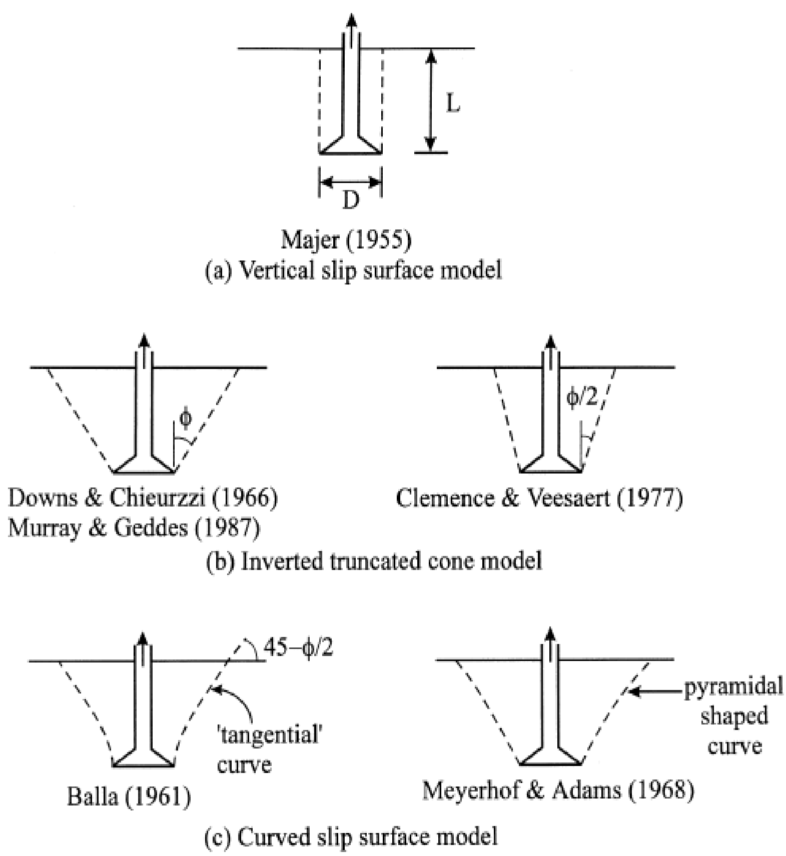


Figure 1: Assumed failure mechanisms for belled piers subject to uplift loads classified by Dickin (1988)

A relationship between the ratio of unit uplift resistance to overburden pressure and the ratio of footing depth to width was demonstrated by Sutherland (1965). This dimensionless empirical relation helped Sutherland to predict full scale behaviour. Spence, B.E. (1965) examined a theory in which shear was mobilized on a cylindrical surface extending only partially to ground level. He realized that the ratio of the cylinder height to base diameter was constant with the ratio of the depth to base diameter by taking into account the full suction and soil weight.

Downs and Chieurzzi (1966), who had done field test observations on belled piers, hold on to the concept that the uplift capacity is derived from the weight of the soil in an inverted cone above the bell plus the self weight of the pier. Then, Meyerhof and Adams (1968) suggested a pyramidal shaped slip surface above the anchor based on his observation in laboratory model tests. According to Clemence and Veesaert (1977), the angle θ in the inverted cone slip-surface model is equal to ϕ / 2 to the vertical, where ϕ is the angle of friction.

Lateral Earth Pressure Coefficients

The lateral earth pressure coefficients in uplift recommended by various investigators are presented in Table 1. Due to the various assumptions made about the failure mechanisms, the coefficients values differ in case such theories are compared and shear stresses acting in the soil.

Table 1: Assumption made in design methods summarized by Ilamparuthi and Dickin (2001)

Source	Recommended Coefficient K	ϕ (Degree)	Value of K	Remarks
Meyerhof and Adams (1968)	$K_u = 0.9$	34.5 39.5 43	1.1 - 1.8 1.2 - 2.1 1.3 - 2.5	K varies linearly with L/D ratio
Clemence and Veesaert (1977)	K_o	34.5 39.5 43	0.43 0.36 0.32	K = 1 used in calculations
Surterland *et al*. (1982)	K_o	34.5 39.5 43	0.43 0.36 0.32	-
Kulhawy (1985)	K_a to K_o K_o to 1 1 to K_p	Loose Medium Dense	- - -	Used stress modification factor to adjust for construction influences
Chattopadhyay and Pise (1986)	K_a	34.5 39.5 43	0.43 0.36 0.32	-
Bobbitt and Clemence (1987)	$S_f K_u$	34.5 39.5 43	0.66 - 1.1 0.72 - 1.3 0.78 - 1.5	60% of Meyerhof and Adams (1968) values
Ghaly et al. (1991)	$K_p = \frac{(1+\sin\delta)}{(1-\sin\delta)}$	34.5 39.5 43	2.2 2.5 2.8	δ = 0.6-0.7 ϕ for shallow anchors

Half-cut model tests (e.g. Dickin and Leung, Ilamparuthi) show strong evidence that failure surfaces above shallow anchors or belled piles in dense sand are slightly curved and extend outwards from the corner of the foundation to the soil surface at an angle close to ϕ/2 to the vertical. Similar findings have been reported by a number of other investigators.

Despite these findings, Meyerhof and Adams theory adopts an equivalent *K* value acting on a vertical plane extending above the anchor. It is similar to the simple vertical slip surface model developed by Majer (1955). It may be noted from Table 2 that the *K* values for a cylindrical failure surface are higher than those for the truncated/curved failure wedges. For a cylindrical failure wedge, K value which is greater than unity and less than the K_p value have been used. On the other hand, K_o value has been adopted for the curved failure wedge.

Existing Uplift Capacity Analysis

Theories of designing uplift capacity of enlarged base pile are derived from researches conducted by Majer (1955), Balla (1961), Meyerhof and Adams (1968), Ovesen (1981), Dickin and Leung (1990,1992) and so on. Amendment on the design method has been made from time to time by taking into account more factors to produce more complete and accurate design. Generally, enlarged base pile gain uplift capacity through skin resistance between the pile shaft and the soil and bearing on top of the base.

The net ultimate resistance of a pile subjected to uplift forces:

$$Q_{un} = Q_{ug} - W$$

where Qun = Net uplift capacity

Qug = Gross uplift capacity

W = Effective weight of pile

The net uplift capacity of enlarged base pile can be calculated from formula generated by Meyerhof and Adams (1968) and Das Seeley (1975).

$$Q_{un} = B_q A_q \gamma L$$

where Bq = uplift factor

Aq = Area of bottom of enlarged base

γ = Unit weight of soil above the bottom of the bell

L = length of the pile

Uplift factor, Bq = 2(L/Db) Ku' tan ϕ [m(L/Db) + 1] + 1

Where Ku' = Uplift coefficient (0.9 for soil friction angle from 30o to 45o)

ϕ = angle of friction of soil

m = coefficient of shape factor

Table 2: Values of coefficient of shape factor, m base on soil friction angle, ϕ (Meyerhof and Adams, 1968)

ϕ	20	25	30	35	40	45	48
m	0.05	0.10	0.15	0.25	0.35	0.50	0.60

Values of uplift factor, B_q are dependent on embedment ratio, (L/D_b). Values of B_q become constant when critical embedment ratio, $(L/D_b)_{cr}$ is reached while $(L/D_b)_{cr}$ increases with increased soil friction angle, ϕ. Figure 2 show an relationship between uplift factor B_q and embedment ratio (L/D_b) based on angle of friction of soil.

Table 3: Relationship between critical embedment ratio, $(L/D_b)_{cr}$ and soil friction angle, ϕ (Meyerhof and Adams, 1968)

ϕ	20	25	30	35	40	45	48
$(L/D_b)_{cr}$	2.5	3	4	5	7	9	11

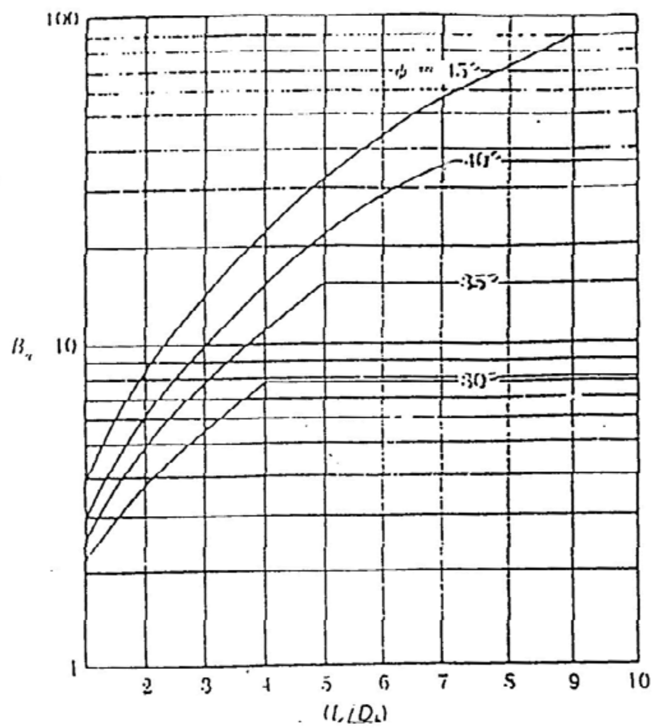


Figure 2: Variation of uplift factor, B_q with embedment ratio,(L/Db) and soil friction angle, ϕ (Braja M.Das, 1984)

Balla's Method for Estimating Uplift Capacity

A circular failure surface was noted by Balla (1961) in the breaking out resistance of mushroom foundation for pylons. He proposed a solution for the value of the pullout resistance for mushroom foundation.

$V = G_1 + T_v + G_2$

where V = The breaking out resistance

G_1 = The bulk weight of soil

G_2 = The weight of foundation slab

T_v = Shear reaction to skin friction

Formula to obtain G_1,

$G_1 = (D\text{-}v)^3 \gamma F_1 (\varphi, \lambda)$

where F_1 (φ, λ) = factor based on angle of friction of angle of soil and on a coefficient characteristic for the shape of the foundation body and given by

$\lambda = (D\text{-}v) / B$

γ = bulk unit weight of soil

Formula to obtain G_2 ,

$$G_2 = R_0^2\pi\,(D-v-m)(\gamma_e - \gamma) + 1/3\,\pi\, m\,(R_0^2 + R_0R_a + R_a^2) + R_0^2\,\pi v\gamma_e$$

Formula to obtain Tv,

$$T_v = (D-v)^3\left[\ (c/v)(1/D - v)F_2(\varphi,\gamma) + F_3(\varphi, \lambda)\ \right]$$

Where $F_2(\phi, \lambda)$ and $F_3(\phi, \lambda)$ are factor depending on the angle of friction and the formal coefficient.

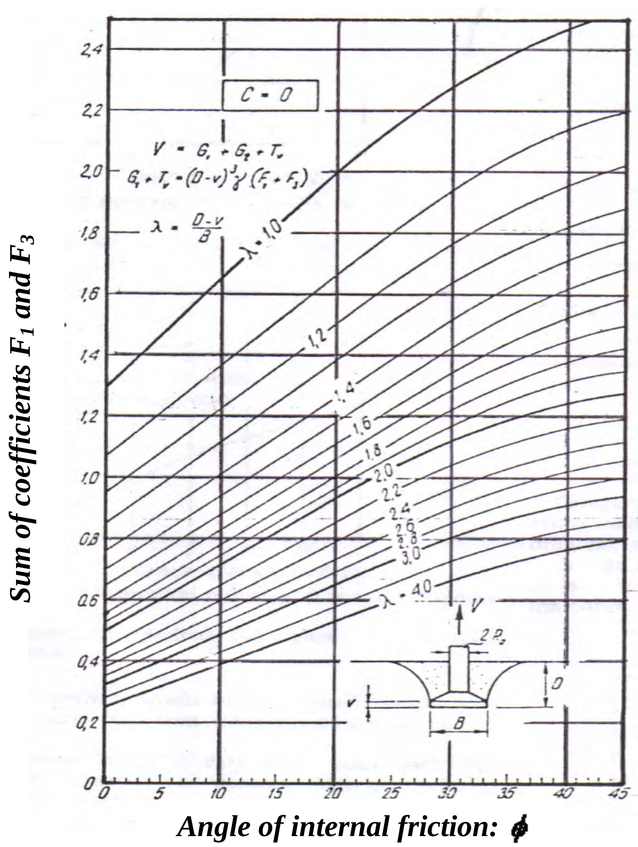


Figure 3: Coefficients of Breaking out resistance (Balla, 1961)

Meyerhof and Adams's Method for Estimating Uplift Capacity

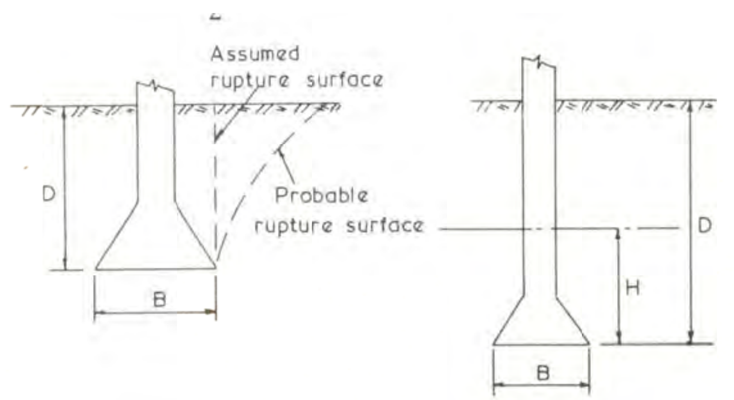
Meyerhof and Adams (1968) proposed a solution for the value of the pullout resistance, P_u.

For a shallow pile:

$$P_u = \pi B c_u D + Sf \frac{\pi}{2} B \gamma D^2 K_u \tan\phi + W$$

For a deep pile:

$$P_u = \pi B c_u H + Sf \frac{\pi}{2} B \gamma (2D - H) H K_u \tan\phi + W$$



(a) (b)

Figure 4: (a) Shallow pile (b) Deep pile

Where B = diameter of enlarged base

c_u = average undrained shear strength of soil

ϕ = angle of friction of soil

γ = bulk unit weight of soil

W = weight of soil and pile in cylinder of diameter B and height D

S_f = a shape factor

K_u = coefficient of lateral earth pressure

H = maximum height of rupture surface

According to Meyerhof and Adams, values for K_u and S_f may be assumed to be as follows:

$K_u = K_p \tan 0.67\,\phi$ (where K_p = coefficient of passive earth pressure)

$S_f = 1 + \frac{mD}{B}$ (with a maximum value, for deep piles, of $1 + \frac{mH}{B}$)

Table 4: The coefficient m and shape factors base on angle of friction of soil, ϕ (Meyerhof, 1968)

ϕ	20	25	30	35	40	45	48
H/B	2.5	3.0	4.0	5.0	7.0	9.0	11
m	0.05	0.10	0.15	0.25	0.35	0.50	0.60
S_f	1.12	1.3	1.6	2.25	3.45	5.5	7.6

Breakout Factor

Simple expressions and design chart were provided by researchers in order to facilitate the determination of the uplift capacity. The net ultimate uplift resistance, Qu is defined as the gross uplift capacity of the piles minus the weight difference between the piles and soil displaced by the piles. It can be expressed in terms of a dimensionless breakout factor. Breakout factors, N_U for piles embedded in sand were calculated by using the following definition:

$$N_u = \frac{Q_u}{\gamma' A L}$$

where Q_U = net ultimate uplift resistance

A = plan area of pile bell

L = depth of embedment.

γ' = unit weight of soil

Formulation of the breakout factor proposed by a number of researchers is summarized by Dickin and Leung (1990) in Table 5. From the table, it is noted that most of the researchers suggest breakout factor as a function of embedment ratio D/B_b and soil friction angle. Besides, some methods are mainly for the design of rectangular anchors. To use it in case of circular anchor, diameter b_b is taken as equal to that of a square anchor with equivalent width B_e having the same area as the circular anchor. Hence, $B_e = \sqrt{\frac{\pi b_b^2}{4}}$.

Unique relationship between effective breakout factor and effective embedment ratio is derived for a given sand density. Dickin and Leung (1990) also made a comparison between theoretical uplift breakout factors, N_U for piers in sand with bell diameter of 1 m and with friction angle of 40^o in Table 6. The comparison showed a large variation that exists between calculations from different design methods. Obviously, difficulties arise when it come to deciding which theory gives a reasonable estimation of uplift capacity of enlarged base pile.

Table 5: Existing design methods summarized by Dickin and Leung (1990)

Researcher	Method of Analysis	**Formulation**
Majer (1955)	Vertical slip surface	**N_u = 1+ 2K (L/b_b) tan φ where K is the coefficient of lateral stress in soil**
Balla (1961)	Tangential-curve slip surface	**N_u = (F1 + F3)** $\left(\frac{4}{\pi}\right)\left(\frac{L}{b_b}\right)^2$ **Where F1 and F3 are dependent on φ and γ' and obtained from chart provided by author**
Downs and Chieurzzi (1966)	Inverted cone slip surface; cone angle with vertical = φ	**N_u = 1+ 2** $\left(\frac{L}{b_b}\right)\tan\varphi+\frac{4}{3}\left(\frac{L}{b_b}\right)^2\tan^2\varphi+\left(\frac{b_s}{b_b}\right)^2$
Meyerhof and Adams (1968)	Pyramidal-shaped slip surface	**Nu = 2** $\left(\frac{L}{b_b}\right)K'_u\tan\varphi\left[m\left(\frac{L}{b_b}\right)+1\right]+1$ **Where K'$_u$ = 0.9 for $30^o < \phi < 45^o$ and m is the shape factor, dependent on φ**
Clemence and Veesaert (1977)	Inverted cone slip surface; cone angle with vertical = ϕ / 2	**N_u =** $\left[1+\left(\frac{L}{b_b}\right)\tan\left(\frac{\phi}{2}\right)\right]^2+4K_o\tan\varphi\cos^2\left(\frac{\varphi}{2}\right)\left[\frac{1}{2}\left(\frac{L}{b_b}\right)+\frac{1}{3}\left(\frac{L}{b_b}\right)^2\tan\left(\frac{\varphi}{2}\right)\right]$ **where K_o is the coefficient of lateral earth pressure.**
Andreadis and Harvey (1981)	Derived from model and field tests on anchor.	**Chart of N_u in terms of φ and L/b_b provided by authors**
Ovesen (1981)	Derived from centrifugal model tests on horizontal anchor plates	**N_u = 1 + (4.32 tan φ - 1.58)** $\left(\frac{L}{B_e}\right)^{1.5}$ **where** **B_e =** $\sqrt{(\pi b_b^2/4)}$
Rowe and Davis (1982)	Finite-element analysis giving uplift capacity of strip anchors	**N_u =** $F_\gamma R_\psi R_K R_R$ **where** F_γ **is a function of φ and L/bb,** R_ψ **is a function of ψ and L/b_b, R_k and R_R may be taken as unity; charts are provided by the authors.**
Surterland *et al*. (1982)	Inverted cone slip surface; cone angle is a function of φ	**N_u =** $\frac{8}{3}\left(\frac{L}{b_b}\right)^2\tan^2\alpha+4\left(\frac{L}{b_b}\right)\tan\alpha+1$ **where** $\alpha = 0.25\left[I_D\left(1+\cos^2\varphi\right)+1+\sin^2\varphi\right]\varphi$
Vermeer and Sutjiadi (1985)	Inverted cone slip surface; cone angle = ψ	**N_u = 1 + 2** $\left(\frac{L}{B_e}\right)\tan\varphi\cos\varphi_{cv}$ **where φ_{cv} is the critical state friction angle**
Murray and Geddes (1987)	**Inverted cone slip surface; cone angle = φ**	**N_u = 1 +** $\left(\frac{L}{B_e}\right)\tan\varphi\left[2+\frac{\pi}{3}\left(\frac{L}{B_e}\right)\tan\varphi\right]$

Notes: N_u = breakout factor; L = embedment depth; b_b = diameter of bell; φ = peak friction angle; ψ = dilatancy angle; γ' = soil unit weight; I_D = soil density index.

Table 6: Comparison between theoretical uplift breakout factors, N_U for piles in sand with bell diameter of 1 m and with angle of friction of 40°

Research		L/b_b		Remarks
source	1	3	5	
Majer (1955)	1.7	3.0	4.4	$K = K_0 = 0.4$
Balla (1961)	3.2	10.3	19.1	Assume $F_1 + F_3 = 0.6$ For L/b_b = 5, 7
Downs and Chieurzzi (1966)	3.6	14.5	32.9	Assume $b_s = 0$
Meyerhof and Adams (1968)	3.0	10.3	21.8	$K'_u = 0.9$; m = 0.35
Clemence and Veesaert (1977)	2.6	7.5	14.5	$K_0 = 0.4$
Ovesen (1981)	3.5	13.7	28.4	B_e = 0.886 m
Andreadis and Harvey (1981)	4.0	14.0	32.0	Extrapolated results For $L/b_b = 1$
Rowe and Davis (1982)	2.7 F_γ=1.8 R_ψ =1.05 S_f=1.4	9.3 F_γ=3.2 R_ψ =1.16 S_f=2.5	18.7 F_γ=4.8 R_ψ =1.30 S_f=3	$R_k = R_R = 1$ ψ = 17° B_e = 0.886 m
Surterland *et al.* (1982)	3.3	10.9	22.6	$I_D = 0.6$
Vermeer and Sutjiadi (1985)	2.7	5.1	8.5	B_e = 0.886 m $\phi_{cv} = 25.6^o$ ($\psi = 17^o$)
Murray and Geddes (1987)	3.8	15.1	34.0	B_e = 0.886 m

Influence of Foundation Geometry on Breakout Factor

Dickin and Leung (1990) conducted a laboratory study in the centrifuge simulating the uplift behaviour of belled pier in dry sand. The centrifugal model tests were employed to investigate the influence of foundation geometry on the uplift capacity of enlarged base piles in sand. The geometry of an enlarged base pile include shaft diameter b_s, bell diameter b_b, embedment ratio L/b_b and bell angle α. Besides, the effects of sand properties such as soil unit weight and density index were also investigated. The centrifuge tests conducted by Dickin and Leung (1990) suggest that the breakout factor decreases when bell angle increase for 1 m diameter belled piers in dense and loose sand. The influence of bell angle on breakout factor is small for α values less than 62^o but it decreases obviously for steeper angles as seen in Figure 5. This observation match the finding of Dewaikar (1985) who suggested a negligible influence of bell angle for α values between 15 and 60^o from his finite-element analysis. The unit weight of sands may differ due to a variety of grain shape, size and grading. Thus, an alternative relationship based on density index I_d shown in Figure 6 is more useful for design purposes. Figure 7 shows the influence of diameter ratio, b_s/b_b on breakout factor for enlarged base pile embedded at embedment ratio, L/b_b = 4 with a 45^o bell angle. It was observed that the breakout factor decreases steadily with increased diameter ratio. The variation of breakout factor with embedment ratio for 1 m diameter belled piers in dense (γ' = 16 kN/m^3) and loose (γ' = 14.3 kN/m^3) sand is given in Figure 8.

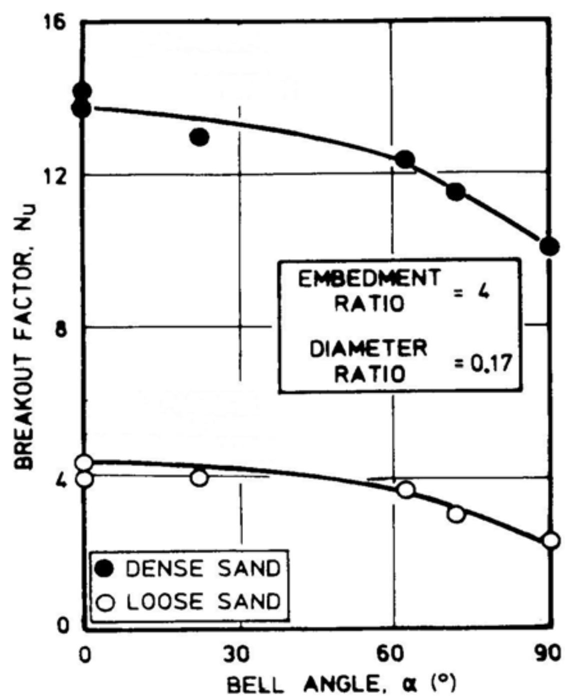


Figure 5: Variation of breakout factor N_u with bell angle α in centrifuge uplift tests (Dickin, 1988)

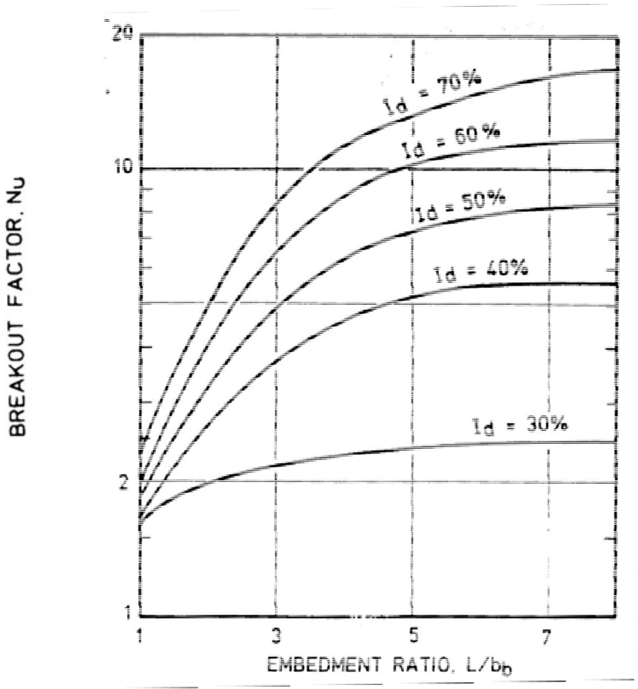


Figure 6: Design curve for belled piers in sand in centrifuge uplift tests (Dickin and Leung, 1992)

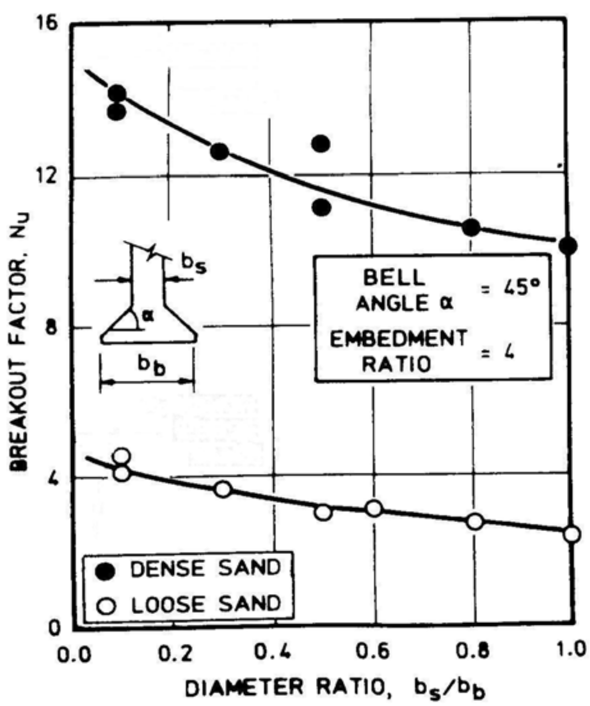


Figure 7: Variation of breakout factor N_u with diameter ratio, b_s/b_b in centrifuge uplift tests (Dickin, 1988)

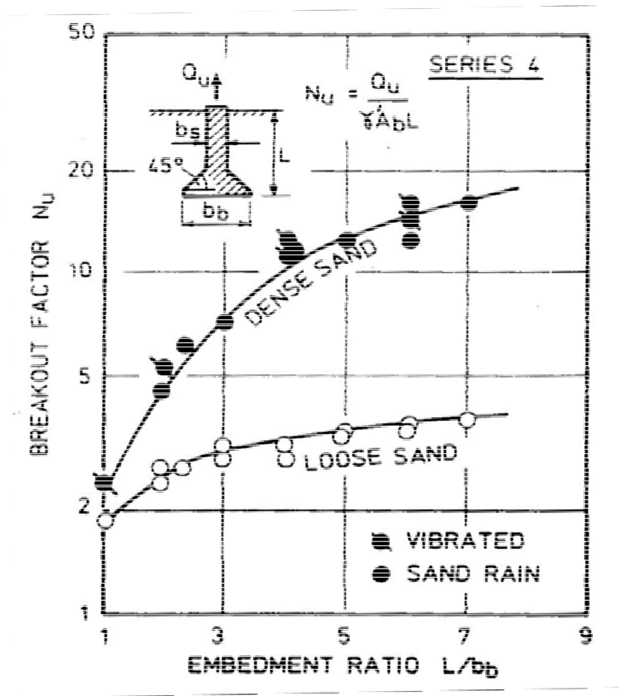


Figure 8: Variation of breakout factor N_u with embedment ratio L/b_b for 1 m diameter belled piers in sand (Dickin and Leung, 1990)

Buckingham Pi Theorem

Buckingham ' s Pi theorem states that if there are n variables in a problem and these variables contain m primary dimensions (for example M, L, T), the equation relating all the variables will have (n-m) dimensionless groups. In other words, if an equation involve n variables is dimensionally homogeneous, it can be reduced to a relationship among (n-m) independent dimensionless groups where m is the minimum number of reference dimensions required to describe the variables. Buckingham referred to these groups as π groups.

In order to perform dimensional analysis, one must first list all of the variables that are defining a problem. These variables include dimensional and non dimensional constants. They must be expressed in terms of basic or primary dimensions such as mass, length, and time or force, length, and time. Finally, the pi groups can be written in relation to one another to describe the problem in the form of $\pi_1 = f(\pi_2, \pi_3, \ldots \pi_{n-m})$. In this case, Π_1 would contain the dependent variable. The actual functional relationship, f, must be determined by experiment. If a relation subsists among any number of physical quantities of n different kinds with $Q_1, Q_2, \ldots Q_n$ represent one quantity of each kind and the remaining quantities of each kind are specified by their ratios r’, r’’, … , and so on, the relationship can be described in the form of $\pi_1 = f(\pi_2, \pi_3, \ldots \pi_{n-m}, r', r'' \ldots)$.

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