Geotech Geol Eng (2007) 25:151–161
DOI 10.1007/s10706-006-9000-z

ORIGINAL PAPER

Uplift capacity of single piles: predictions and performance

K. Shanker · P. K. Basudhar · N. R. Patra

Received: 24 July 2005 / Accepted: 18 July 2006 / Published online: 28 September 2006

Abstract The paper pertains to the development of a simple semi-empirical model for predicting the uplift capacity of piles embedded in sand. Various pile and soil parameters such as length (L), diameter (d) of the pile and angle of friction (ϕ), soil–pile friction angle (δ) and unit weight (γ) of the soil which have direct influence on the uplift capacity of the pile are incorporated in the analysis. A comparative assessment of the ultimate uplift capacity of piles predicted by using the proposed theory and some of the available theories are made with respect to each other and with reference to the measured values obtained from model tests in the laboratory. For this purpose experimental data have been collected from the literature and also from model tests conducted as a part of the present investigation. The study shows the proposed model has an excellent potential in predicting the uplift capacity of piles embedded in sand that are consistent with model pile test results.

Keywords Angle of friction · Diameter · Length · Model tests · Pile · Sand · Soil–pile friction angle · Unit weight · Uplift capacity

K. Shanker · P. K. Basudhar · N. R. Patra (✉)
Civil Engineering Department, Indian Institute of Technology Kanpur, Kanpur 208016, India
e-mail: nrpatra@iitk.ac.in

Notations

The following symbols are used in this paper.

A_s Embedded surface area
A_1 Net uplift capacity factor
D_r Relative density
d Pile diameter
K_s Lateral earth pressure coefficient
K_u Theoretical uplift coefficient factor
L Embedded length of pile
P_u Ultimate uplift capacity of pile
P_{nu} Net ultimate uplift capacity of pile
Δz Thickness of wedge element
ϕ Angle of internal friction of the soil
ψ Dilatancy angle
θ Angle of failure surface with horizontal
β Angle of failure surface with vertical
δ Pile-soil friction angle
f Unit skin friction
γ Unit weight of the soil

Introduction

Prediction is an integral component of the practice of geotechnical engineering, based on which decisions and actions are taken depending on the assessment of the accuracy and reliability of such predictions (Lambe 1973). Structures supported on piles are very often subjected to large lateral loads due to wind or wave load and the resulting moments induce tension in some of the piles. Unlike the prediction of ultimate load carrying capacity of piles under compressible loads the same under uplift force is an area which is least studied. Very few papers are available to model and predict the behavior of piles under uplift loads (Vesic 1970; Meyerhof 1973; Das et al. 1977; Das 1983; Rao and Venkatesh 1985; Chattopadhyay and Pise 1986; Alawneh et al. 1999). However, so far published test results in sand have indicated that the skin frictional resistance of piles to uplift loads is appreciably lower than that mobilized in resistance to compressive loading (Vesic 1970; Nicola and Randolph 1993; Ramasamy et al. 2004). With the increasing use of straight shafted piles to resist uplift loads necessitated accurate assessment of uplift resistance for safe and economical design of pile foundations. Few theories have been developed based on the limit equilibrium method to find the net uplift capacity of the pile (Meyerhof 1973; Das 1983; Chattopadhyay and Pise 1986) and validated through experimental measurements. The above theories differ mainly in their assumptions with regard to the shape and extent of the failure surface. Based on the available literature, some of these methods are taken up for further studies. Predictive capabilities of these methods are re-evaluated with reference to the reported experimental data available in literature in addition to data that are obtained as a part of the present investigation. There after a simple semi-empirical method has been proposed for such prediction.

Existing predictive models

In the following sections along with some of the commonly applied predictive models that were developed quite early and reported by Johnson and Kavanagh (1968) and Das (2003) are presented along with the models developed at a later stage (Meyerhof 1973; Das 1983; Chattopadhyay and Pise 1986). For the sake of completeness these are described here in brief.

Standard model

Assuming that failure takes place on a cylindrical surface along the shaft the net uplift capacity of a vertical pile can be estimated as follows,

$$P_{\mathrm{nu}} = \frac{\pi}{2} K_{\mathrm{s}} d \gamma L^2 \tan\delta \tag{1}$$

Where, K_{s} is the lateral earth pressure coefficient, d is the diameter of the pile, γ is the unit weight of the soil, L is the length of the pile, δ is the soil–pile friction angle.

The accuracy of the predictions made by using this approach would mainly depend on the correctness of the assumed value of the coefficient of lateral earth pressure (K_{s}). As suggested by Levacher and Sieffert (1984) and Das (2003) for bored piles K_{s} can be taken as equal to K_0 = (1- sinϕ).

Truncated cone model

Field engineers generally estimate the uplift capacity of the pile by assuming a slip surface as a truncated inverted cone with the enveloping sides rising at $\phi/2$ degrees from the vertical. Dead weight within the frustum is usually considered as the ultimate uplift capacity of the pile.

$$P_{\mathrm{nu}} = \frac{\pi}{3} L^3 \tan^2 \frac{\phi}{2} \gamma \tag{2}$$

Meyerhof's model (1973)

Ignoring the weight of the pile he suggested an expression for the pull-out resistance assuming that under axial pull the failed soil mass has a roughly similar shape as for a shallow anchor. Thus,

$$P_{\mathrm{nu}} = \frac{\pi}{2} K_{\mathrm{u}} d \gamma L^2 \tan\delta \tag{3}$$

Where K_{u} = uplift coefficient and can vary with in wide limits and depend not only on the soil properties, but also on the type of pile and method of installation.

Das's model (1983)

Based on the model test results, he reported that the unit skin friction at the soil–pile interface increases linearly with depth up to a critical embedment ratio. The critical embedment ratio is dependent on the relative density (D_r) and expressed as,

$$(L/d)_{cr} = 0.156D_r + 3.58 \quad (\text{For} D_r \leq 70\%) \tag{4}$$

and

$$(L/d)_{cr} = 14.5 \quad (\text{For} D_r \geq 70\%) \tag{5}$$

The net ultimate uplift capacity of piles in sand can be estimated as,

$$P_{nu} = \frac{1}{2} p\gamma L^2 K_u \tan\delta \qquad [\text{If} L/d \leqslant (L/d)_{cr}] \tag{6}$$

$$P_{nu} = \frac{1}{2} p\gamma L_{cr}^2 K_u \tan\delta + p\gamma L_{cr} K_u \tan\delta (L - L_{cr}) \quad [\text{If} L/d > (L/d)_{cr}] \tag{7}$$

Chattopadhyay and Pise's model (1986)

They proposed a generalized theory to evaluate uplift resistance of a circular vertical pile embedded in sand. Assuming the failure surface to be curved, they estimated the net uplift capacity of a pile embedded in sand as,

$$P_{nu} = A_1 \gamma \pi d L^2 \tag{8}$$

Where A_1 = Net uplift capacity factor and depends on ϕ, δ, and L/d ratio.

Proposed model

In developing the method, for the sake of simplicity, the failure surface is assumed to be a truncated cone with the edges passing through the tip of the pile at an angle of β with respect to the vertical axis of the pile as shown in Fig. 1. The angle depends on factors like friction angle and angle of dilatancy (ψ), which is a function of relative density of the soil. From literature it is found that this angle has been assumed to be any one of the following, namely dilatancy angle (ψ), $\phi/2$ or a function of ϕ (Dickin and Leung 1990).

During uplift of a pile, an axi-symmetric solid body of revolution of soil along with the pile assumed to move up along the resulting surface. The movement is resisted by the mobilized shear strength of the soil along the failure surface and self weight of the soil and pile. In the limiting equilibrium condition, ultimate capacity of the pile attained. A circular wedge of thickness ΔZ at a height Z above the tip of the pile is considered. Forces acting on the wedge are sown in Fig. 2(a). For evaluating the mobilized shear resistance ΔT along the failure surface of length ΔL, at limiting condition it is assumed that $\Delta T = \Delta R \tan \phi$, in which ΔR is normal force acting on the failure surface of the wedge. Further the coefficient of lateral earth pressure (K) within the wedge is taken as (1-sinϕ) tanδ/tanϕ. This expression for K has been chosen so that $\delta = \phi$, $K = K_o = (1 - \sin \phi)$, and for other values of δ, K is a function of K_o, δ, and ϕ (Chattopadhyay and Pise 1986).

From Fig. 2(b)

$$\Delta R = \Delta Q\cos\theta + K\Delta Q\sin\theta \tag{9}$$

where

$$\Delta Q = \gamma\left(L - Z - \frac{\Delta Z}{2}\right)\Delta L \tag{10}$$

$$\Delta R = \gamma\left(L - Z - \frac{\Delta Z}{2}\right)(\cos\theta + K\sin\theta)\frac{\Delta Z}{\sin\theta} \tag{11}$$

and

$$\Delta T = \gamma\left(L - Z - \frac{\Delta Z}{2}\right)(\cos\theta + K\sin\theta)\frac{\Delta Z\tan\phi}{\sin\theta} \tag{12}$$

Considering the vertical equilibrium of the circular wedge, and assuming that weight of the pile of length ΔZ is equal to the weight of the soil corresponding to the volume occupied by the pile for the length ΔZ;

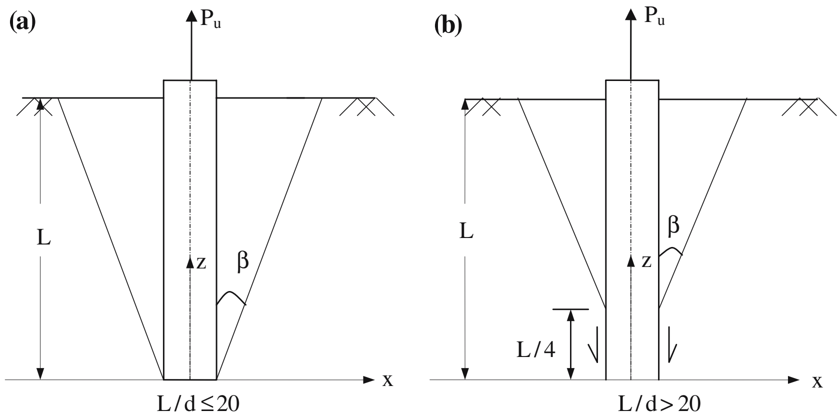


Fig. 1 Pile and failure surface

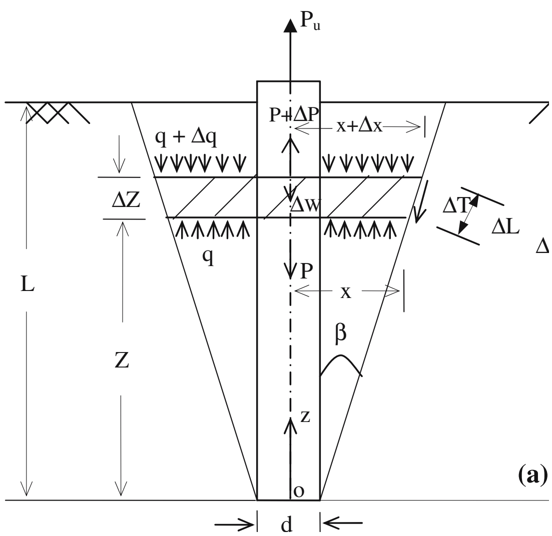


Fig. 2 Free body diagram of the wedge

$$(P+\Delta P)-P+q\pi x^2-(q+\Delta q)\pi(x+\Delta x)^2-\Delta W$$
$$-2\pi\left(x+\frac{\Delta x}{2}\right)\Delta T\sin\theta=0 \tag{13}$$

Substituting the value of ΔT from Eq. 12 in Eq. 13 and simplifying

$$\frac{\Delta P}{\Delta Z}=\pi q\frac{\Delta x}{\Delta Z}(2x+\Delta x)+\pi\frac{\Delta q}{\Delta Z}(x+\Delta x)^2$$
$$+\pi\frac{\Delta q}{\Delta Z}(x+\Delta x)^2+\pi(x+\Delta x)^2\gamma$$
$$+2\pi\left(x+\frac{\Delta x}{2}\right)\gamma\left(L-Z-\frac{\Delta Z}{2}\right)$$
$$(\cos\theta+K\sin\theta)\tan\phi \tag{14}$$

In the limit, Eq.14 can be written after substituting $q = \gamma\ (L - Z)$

$$\frac{dp}{dZ}=2\pi\left(\frac{Z}{\tan\theta}+\frac{d}{2}\right)\gamma(L-Z)\frac{1}{\tan\theta}$$
$$+2\pi\left(\frac{Z}{\tan\theta}+\frac{d}{2}\right)\gamma(L-Z)$$
$$\times(\cos\theta+K\sin\theta)\tan\phi \tag{15}$$

$$\frac{dP}{dZ}=C_1(L-Z)+C_2\left(LZ-Z^2\right) \tag{16}$$

Where

$$C_1=\pi d\gamma\left[\frac{1}{\tan\theta}+(\cos\theta+K\sin\theta)\tan\phi\right] \tag{17}$$

$$C_2 = \frac{2\pi\gamma}{\tan\theta}\left[\frac{1}{\tan\theta} + (\cos\theta + K\sin\theta)\tan\phi\right] \tag{18}$$

Hence gross uplift capacity of the pile P_u is given by

$$\begin{aligned} P_u &= \int_0^L \mathrm{d}P\mathrm{d}Z \\ &= \int_0^L \left[C_1(L-Z) + C_2\left(LZ - Z^2\right)\right]\mathrm{d}Z \end{aligned} \tag{19}$$

$$P_u = \frac{C_1}{2}L^2 + \frac{C_2}{6}L^3 \tag{20}$$

Net uplift capacity

$$P_{nu} = P_u - \frac{\pi d^2}{4}L\gamma \tag{21}$$

Based on earlier studies (Chattopadhyay and Pies 1986) regarding the nature of the slip surface, for piles with $L/d > 20$, in the present analysis the failure surface is assumed to be tangential to the pile surface up to 0.25 L from the tip of the pile. Hence Eq. 19 is integrated between the limits 0 to 0.75 L and added to the skin friction developed in the remaining length.

Present experimental investigation

Tests on model piles were conducted in a steel tank (size $990 \times 975 \times 970$ mm). The tank was sufficiently large to take care of the effect of the edges of the tank on the test results as the zone of influence of the pile due to loading is reported to be in the range of 3–8 pile diameter (Kishida 1963).

Model piles were prepared from mild steel rod of 20×20 mm cross section. The length of embedment of pile, L in sand bed was 200, 400, 600 and 800 mm resulting L/d as 10, 20, 30 and 40, respectively. The model piles were embedded in homogeneous dry sand bed composed of uniformly graded Ennore sand (uniformity coefficient = 1.71 and specific gravity = 2.65).The values of the maximum and minimum dry unit weight of the sand were found to be 16.2 and 14.74 kN/m^3 respectively. Sand was poured uniformly in the tank by using rail fall technique to prepare loose and medium dense bed. The measured values of γ, ϕ, δ and D_r being 15.4 kN/m^3, 34°, 22°, and 34.4% in loose state and15.8 kN/m^3, 38°, 26°, and 54.3% in medium dense state, respectively.

Piles were subjected to tensile loading through a pulley arrangement with a flexible wire whose one end is attached with the pile cap and the other end with a loading pan over which dead loads are gradually placed in stages. A schematic diagram of the complete experimental set-up with the loading system and a pile in place and ready for test is shown in Fig. 3. Two dial gauges with magnetic base having sensitivity of 0.01 mm were used to measure the displacement; the gauges were positioned 180° apart and placed on the pile cap keeping them equidistant from the load axis. The load displacement curves for all the piles are presented in Figs. 4 and 5. From these curves the gross ultimate uplift capacity of the pile was determined using double tangent method and there after subtracting the weight of the pile and pile cap from the above value the net ultimate capacity of the pile was found.

Results and discussions

The experimental results obtained from the present investigation and several others available in the literature on the subject have been collated. Using a particular set of predictions as well as experimental observation made by a particular investigator, predictions were made for the same set using all the other predictive models as described earlier. The obtained results are then compared with each other and the results are presented as follows. Predictions were made by assuming different trial values of β, the angle the slip surface makes with the vertical. From several trials so made it was observed that at an angle equal to ϕ/4 the predicted values were in very good agreement with the experimental results. Therefore, the same is used for further predictions. In Figs. 6–11 the predicted and the measured data (common to all the figures for any particular set of data) are plotted to show the

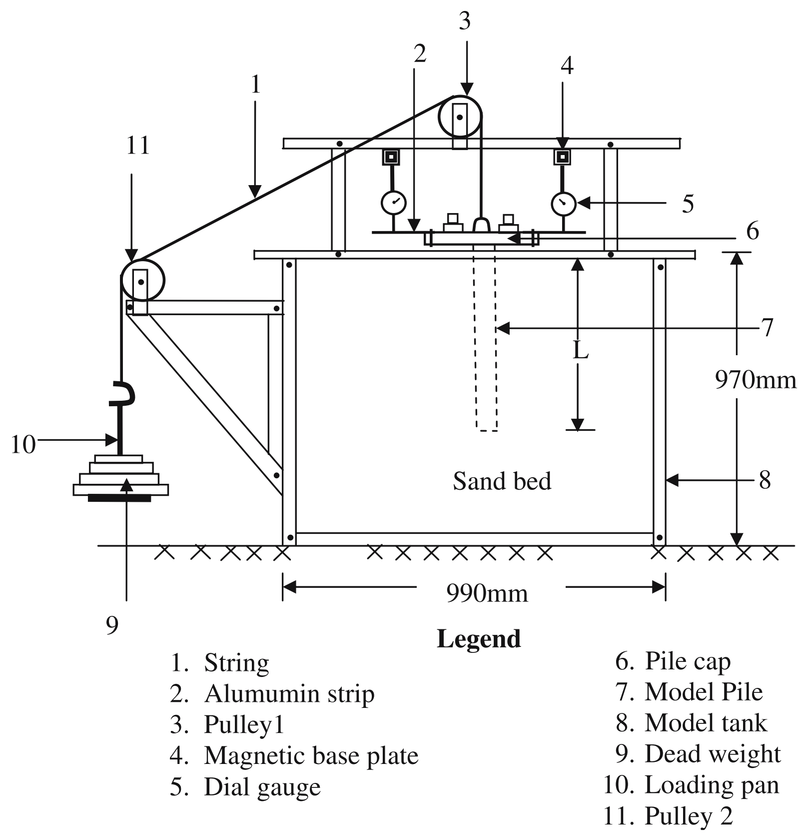


Fig. 3 Experimental setup

quality of the predictions. In Table 1 quantitative comparison of the same is shown; for better appreciation of the relative predicting capability of the model a cumulative frequency table for the data corresponding to the percentage of errors is presented in Table 2.

Figure 6 shows the comparisons between Meyerhof (1973) predictions with the measured net uplift capacity of a pile. From this figure a good agreement between both can be observed as evident from the fact that most of the data points lie very close or around the ideal line. For most of the cases at higher ϕ values the application of the theory results in over estimation of the value. From the accompanied table showing a quantitative comparative study it can be seen that the absolute relative errors between the predicted values (23 data out of 28) lie in general with in the

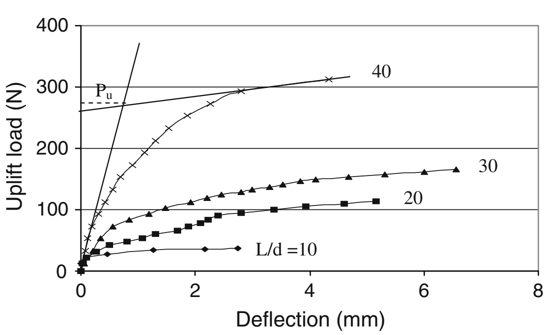


Fig. 4 Load-deflection curves for loose soil

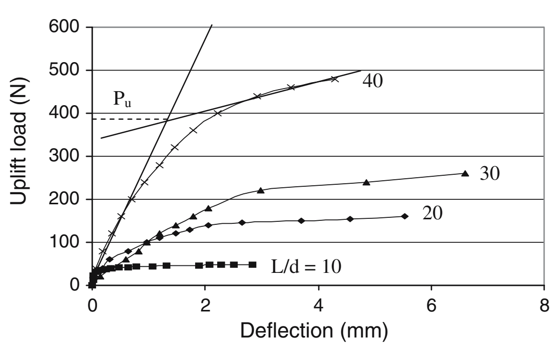
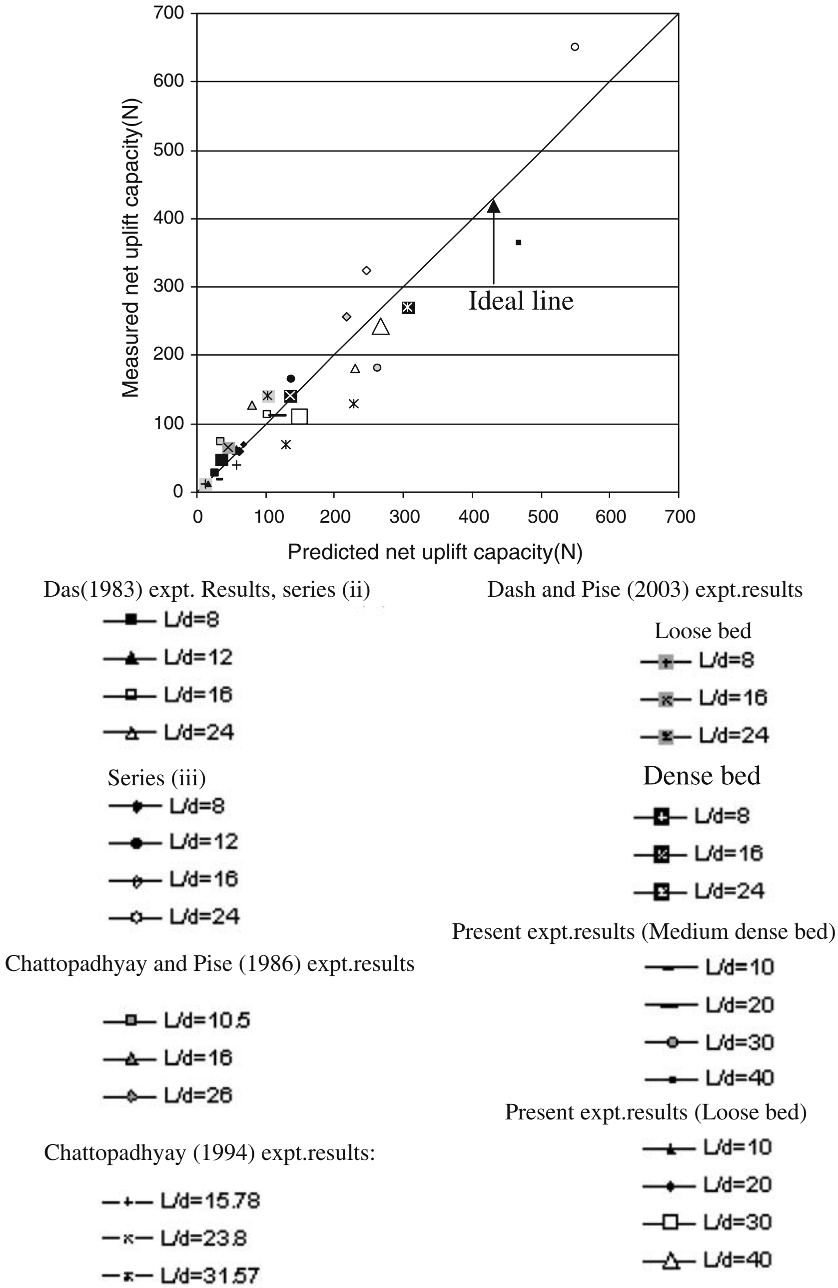


Fig. 5 Load-deflection curves for medium dense soil

Fig. 6 Measured V_s predicted (Meyerhof' model 1973) net uplift capacity



range of 2–45% but in some cases the errors are as high as 55–90%.

Figure 7 shows that the model proposed by Das (1983) is fairly accurate as 75% of the data (21 data out of 28) lies on or around the ideal line with an error varying from 5% to 45%. However some points are located far away from the ideal line with errors lying in the range of 51–72%. One may interpret these data to be outliers and exclude them from the total set of the measured data. But it may not be proper to do so as when other theories are used some of these data are located very near to the ideal line.

From Fig. 8 it is observed that the model proposed by Chattopadhyay and Pise (1986) under estimates the net uplift capacity when L/d ratio is

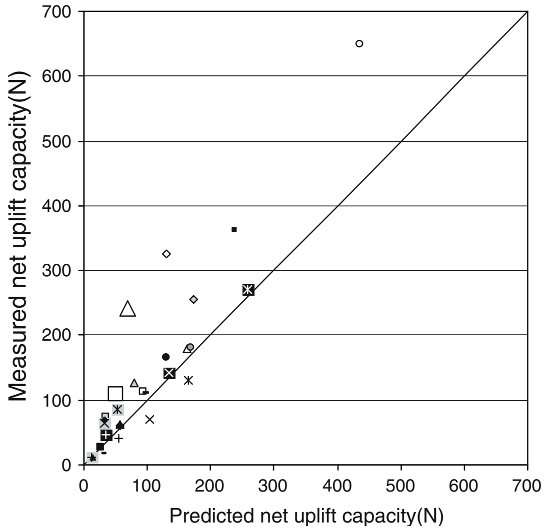


Fig. 7 Measured V_s predicted (Das' model 1983) net uplift capacity

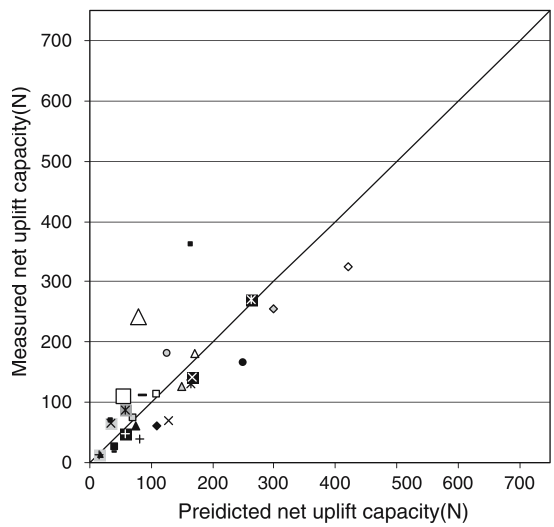


Fig. 8 Measured V_s predicted (Chattopadhyay and Pise' model 1986) net uplift capacity

30 and above. From Table 1, it is found that for 60% of the data, error is more for the above said *L*/*d* ratio. However, the rest of the data (17 out of 28) are close to the ideal line with error less than 45%.

Figure 9 shows the comparison of the values of the uplift resistance computed by using the present model with the values as measured in the laboratory scale model tests. It is seen that in this case 95% of the data lies on and around the ideal line. Unlike some of the earlier methods, these predictions are reasonably good for long piles also. From Table 1 it is seen that for 75% data (21 out of 28) the error is less than 45%.

The predictions made by using the cone model are shown in the Fig. 10; it is observed that the method over predicts the uplift capacity in most of the cases when *L*/*d* ratio is greater than or equal to 20. As the *L*/*d* ratio increases the percentage of error also increases very drastically. The predictions substantially over estimate the uplift capacity values for long piles and should not be used in practice.

Standard method is commonly used in practice or even in class room teaching and finds its place in most of the text books (Lambe and Whitman 1969; Das 2003). Figure 11 shows that the use of this model results in under prediction and hence their use in practice is on the safer side.

From the above study it is found that Meyerhof's model (1973) is by far the best giving good predictions in 82% of the cases when compared with model test results; but the adoption of the method needs the use of graphical charts for choosing the uplift coefficient (K_u). Das's model (1983) comes out second in terms of predictions (75% cases turning out to be good); however, to apply this method use of the Meyerhof's charts are needed to choose the value of the uplift coefficient. Use of Chattopadhyay and Pise's model (1986) gives good predictions (in 61% cases); but, one needs to use several charts developed by using a complicated analysis procedure to get the values of the net uplift capacity factors. For avoiding the use of charts it is necessary to estimate the gross uplift capacity factor which involves numerical integration. The presently developed method of analysis leads to predictions that are very close (in 75% cases) to the measured values; thus, these predictions are better than those proposed by Chattopadhyay and Pise' model (1986), matches equally with those of Das's model (1983) and marginally inferior to those by Meyerhof's model (1973). But, the proposed model is simple; it neither involves any complicated analysis nor needs any graphs.

Table 1 Predicted and measured net uplift capacity of the pile

L/d ratio	Expt. results (N)	Meyerhof's model (1973)		Das's model (1983)		Chattopadhyay and Pise's model (1986)		Proposed model		Truncated cone model		Standard model	
		(N)	% error	(N)	% error	(N)	% error	(N)	% error	(N)	% error	(N)	% error
Das (1983) expt. Results, series(ii): γ = 15.79 kN/m^3, ϕ = 34°, δ = 30.5°, d = 0.0254 m, D_r = 47.6%													
8	27	25.7	4.8	25.7	4.8	40.7	− 50.7	22.0	18.5	12.9	52.0	6.7	75.0
12	62	57.8	6.8	57.4	7.4	75	− 21.0	62.5	− 0.8	43.8	29.4	15.2	75.5
16	114	102.8	8.8	92.8	18.6	108.3	5.0	133.0	− 16.6	103.7	9.0	27.0	76.3
24	180	230.8	− 28.2	163.2	9.3	171.0	5.0	202.0	− 12.2	350.0	− 94.5	60.7	66.0
Das (1983) expt.results, series (iii): γ = 16.88 kN/m^3, ϕ = 40.5°, δ = 39.2°, d = 0.0254 m, D_r = 72.9%													
8	60	61.2	− 2.0	57.5	4.2	108	− 80.0	31.5	47.5	20.2	66.3	7.9	86.7
12	165	137.7	16.5	129.4	22.2	249	− 50.9	92.0	43.2	68.1	58.7	18.0	89.0
16	325	245.4	24.5	229.0	29.8	421.5	− 29.7	196.0	39.7	161.0	50.46	31.8	90.0
24	650	551.0	15.2	435.0	33.0	791.5	− 21.8	281.0	56.7	544.0	16.2	71.5	88.9
Chattopadhyay and Pise (1986) expt. results: γ = 16 kN/m^3, ϕ = 41°, δ = 34°, d = 0.019 m													
10.5	74	34.78	61.3	34.8	53.0	70	5.4	23.5	68.0	18.7	74.6	4.4	94.0
16	127	80.36	36.6	79.65	37.3	149.3	− 17.6	82.0	35.4	65.8	48.2	17.6	92.5
26	256	217.4	15.0	173.5	32.0	298.3	− 16.5	141.0	44.9	292	− 14.3	27.0	91.8
Chattopadhyay (1994) expt.results: γ = 17.0 kN/m^3, ϕ = 40°, δ = 25°, d = 0.019 m													
15.78	40	57	− 42.5	54.84	− 37.1	81	− 102.5	62.0	− 55.0	63.7	− 59.2	7.6	80.9
23.8	70	128.5	− 83.57	104	− 48.5	127.8	− 82.6	90.8	− 29.7	294.8	− 183.0	17.1	75.5
31.57	130	228	− 75.38	165	− 26.9	165	− 26.9	197.0	− 51.5	509.4	− 291.8	30.36	76.6
Dash and Pise (2003) expt. results: γ = 15.0 kN/m^3, ϕ = 30°, δ = 21°, d = 0.025 m, D_r = 35%													
8	12.2	11.3	7.37	11.2	8.2	14.4	− 18.0	14.5	− 18.0	9.0	26.2	4.5	63.1
16	65.2	45.2	44.2	31.6	51.5	33.4	48.8	86.9	− 33.3	72.2	− 10.7	18.65	71.4
24	86.7	101.8	− 17.3	52.1	39.9	57.4	33.8	131.0	− 51.0	243.5	− 180.8	40.7	53.0
γ = 16.4 kN/m^3, ϕ = 38°, δ = 29°, d = 0.025 m, D_r = 80%													
8	47.6	33.9	28.7	33.9	28.8	57.2	− 20.2	23.0	51.6	16.3	65.8	5.5	88.5
16	142	135.6	4.5	134	5.6	166.2	− 17.0	142.0	0	130.3	8.3	22.0	84.5
24	271	305	− 12.5	259	4.4	262.5	3.1	211.0	22.0	439.7	− 62.3	49.4	81.7
Present expt. results: γ = 15.8 kN/m^3, ϕ = 38°, δ = 26°, D_r = 54.3%													
10	17.3	29.3	− 69.4	29.3	− 69.2	35.7	− 106.4	19.19	− 10.9	15.7	9.3	4.73	72.6
20	112	117	− 4.5	98.5	12	86.3	22.9	120.47	− 7.6	125.5	− 12.0	18.94	83.0
30	181.6	266.5	− 46.8	169	6.9	127.2	30.0	178.0	2.0	423.5	− 133.0	42.6	76.5
40	363	468	− 28.9	239	34.2	165	54.5	391.0	− 7.71	1004.0	− 176.6	75.8	79.1
γ = 15.4 kN/m^3, ϕ = 34°, δ = 22°, D_r = 34.4%													
10	12.6	16.7	− 32.5	15.2	− 20.6	16.1	− 27.8	15.45	− 22.6	12.1	4.0	4.38	65.0
20	70	66.9	4.5	32.9	53	34.6	50.6	96.5	− 37.85	96.5	− 27.4	17.5	75.0
30	108.6	150	− 38	50.6	53.4	54.7	49.6	143.0	− 31.6	325.0	− 199.7	39.5	63.6
40	242.4	267	− 10.1	68.3	71.8	80	67.0	311.0	− 28.3	771.5	− 218.3	70.22	71.0

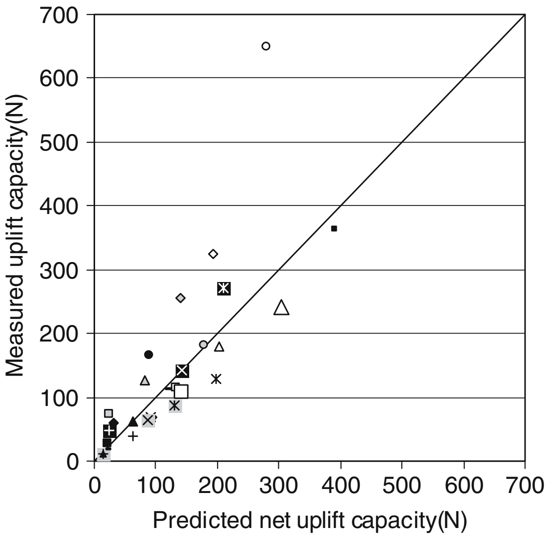


Fig. 9 Measured V_s predicted (proposed method) net uplift capacity

Conclusions

A simple semi-empirical approach of estimating the uplift capacity of single piles in sand based on an assumed inverted and truncated conical slip surface has been proposed here. Analysis of the data revealed that best predictions with very

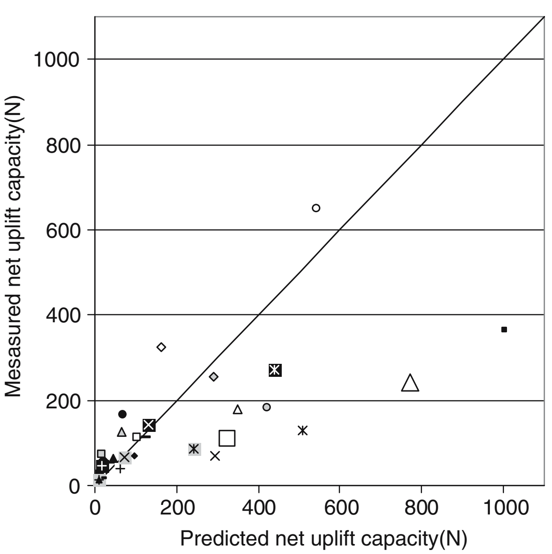


Fig. 10 Measured V_s predicted (truncated cone method) net uplift capacity

Table 2 Cumulative frequency distribution

Absolute error (%)	Number of data points						Cumulative data points					
	Meyerhof's model (1973)	Das's model (1983)	Chattopadhyay and Pise's model (1986)	Proposed model	Truncated cone model	Standard model	Meyerhof's model (1973)	Das's model (1983)	Chattopadhyay and Pise's model (1986)	Proposed model	Trunca-ted cone model	Standard model
0–5	5	3	3	3	1	0	5	3	3	3	1	0
5–10	4	5	1	2	3	0	9	8	4	5	4	0
10–15	2	2	0	2	3	0	11	10	4	7	7	0
15–20	3	0	4	3	2	0	14	10	8	10	9	0
20–25	1	2	4	2	0	0	15	12	12	12	9	0
25–30	3	3	3	2	3	0	18	15	15	14	12	0
30–35	1	3	2	2	0	0	19	18	17	16	12	0
35–40	2	3	0	4	0	0	21	21	17	20	12	0
40–45	2	0	0	1	0	0	23	21	17	21	12	0
45–50	1	1	2	1	1	0	24	22	19	22	13	0
> 50	4	6	9	6	15	28	28	28	28	28	28	28

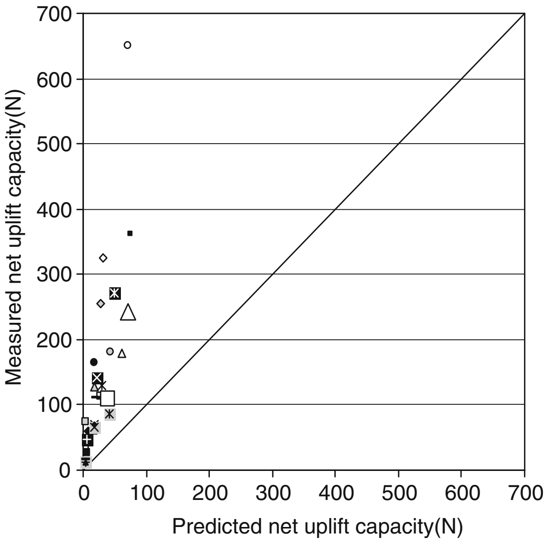


Fig. 11 Measured V_s predicted (standard method) net uplift capacity

good agreement with experimental results (and comparable with predictions made with other available models) are made when the angle that the envelope of the conical failure surface makes with the vertical is equal to $\phi/4$. The effects of parameters like length to diameter ratio, pile friction angle (δ), angle of shearing resistance of the soil (ϕ), on the uplift capacity are incorporated in the proposed analysis. It is found that adoption of the proposed model to predict the ultimate uplift capacity of single piles gives values that are in close agreement with the reported test results. Predictive capabilities of the some of the available methods are also checked vis-a-vis the presently developed method and the experimental data. It is found that these methods are reasonably good in predicting the uplift capacity of single piles with the exception of the conventional truncated cone model that overestimate the pile capacity and errors on the unsafe side. The standard method under predicts the pile capacity, and, as such safe to use in practice.

References

Alawneh AS, Malkawi AIH, Al-Deeky H (1999) Tension tests on smooth and rough model piles in dry sand. Can Geotech J 36:746–753

Chattopadhyay BC, Pise PJ (1986) Uplift capacity of piles in sand. J Geotech Eng 112(9):888–904

Das BM (1983) A procedure for estimation of uplift capacity of rough piles. Soils Found 23(3):122–126

Das BM (2003) Principles of foundation engineering, 5th edn. Thomson Brooks/Cole, Australia

Das BM, Seeley GR, Pfeile TW (1977) Pull out resistance of rough rigid piles piles in granular soils. Soils Found 17(3):72–77

Dash BK, Pise PJ (2003) Effect of compressive load on uplift capacity of model piles. J Geotech Geoenv Eng ASCE 129(11):987–992

Dicking EA, Leung CF (1990) Performance of piles with enlarged bases subjected to uplift forces. Can Geotech J 27(5):546–556

Johnson SM, Kavanagh CT (1968) The design of foundations for buildings. McGraw-Hill Book company, New York

Kishida H (1963) Stress distribution by model piles in sand. Soils Found 4(1):1–23

Lambe TW, Whitman RV (1969) Soil mechanics. John Wiley & Sons, Inc., NY, USA

Lambe TW (1973) Technical report, Proceedinds of the 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow

Levacher DR, Sieffert JG (1984) Test on model tension piles. J Geotech Eng ASCE 110(12):1735–1748

Meyerhof GG (1973) Uplift resistance of inclined anchors and piles, Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engrg, Moscow, vol. 2, pp 167–172

Nicola AD, Randolph MF (1993) Tensile and compressive shaft capacity of piles in sand. J Geotech Eng ASCE 119(12):1952–1973

Ramasamy G et al (2004) Studies on skin friction in piles under tensile and compressive load. Ind Geotech J 34(2):276–289

Rao KS, Venkatesh KH (1985) Uplift behavior of short piles in uniform sand. Soils Found 25(4):1–7

Vesic AS (1970) Tests on instrumented piles, Ogeechee River site. J Soil Mech Fdtn Div ASCE 96(2):561–584