A Method to Estimate Shaft and Base Responses of a Pile from Pile Load Test Results

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ABSTRACT: A practical method for estimating initial shaft and base stiffnesses and ultimate shaft and base resistances of a pile from pile load test results has been proposed. The method employs hyperbolic relationships for the non-linear responses of shaft and base resistances, which are solved using iterative procedures to arrive convergence. A large number of empirical correlations are reported in the literature but many-a-times they either under-estimate or overestimate the pile response. Similarly, numerical tools that can predict shaft and base resistances typically would depend on the expertise of the engineer and also based on various input parameters. Thus, the applicability of the tools therefore too is uncertain. The present method discussed in this paper, would help engineers to estimate shaft and base responses of the actual site using the initial Pile load test results. The analytical solutions of the method are discussed in detail and the proposed method is applied to few load - displacement data available from pile load test results to illustrate its efficacy.

KEYWORDS: Pile capacity, Load test, Compressive load, Settlements, Shaft and base responses

1. INTRODUCTION

Pile foundation is the most effective and economical solution in transferring large vertical loads to deeper depths. As the pile is subjected to axial load, the soil mass surrounding the pile is vital in providing vertical support for the pile. The nature of pile–soil interaction is three dimensional and to complicate the problem further, response of soil is nonlinear. Finding a closed form solution to such a problem is extremely difficult. Several methods have been developed to study and predict the response of the composite pile–soil system. Faruque and Desai (1982) implemented both numerical and geometric non-linearities in their three dimensional finite element model. Rajashree and Sitharam (2001) developed a nonlinear finite element model of batter piles under lateral loading using a hyperbolic relation for static load condition and modified hyperbolic relation, including degradation and gap for cyclic load condition.

The ultimate capacity of the pile is estimated considering the strengths and unit weights of soil layers with depth, overburden pressure and other relevant parameters. The estimated capacities always need to be validated by conducting initial maintained load tests. The estimated capacity may differ with the actual at site since the values of strength, stiffness, interface resistance between pile and soil, lateral earth pressure coefficient with depth and soil stratification, etc., differ from the design parameters considered for estimating the ultimate capacity of pile. The estimation of axial capacity of piles involves considerable uncertainties in selection of appropriate design parameters and the design rules are not always consistent with the installation procedures/processes involved.

Several methods are used to predict the response of the composite pile–soil system. Whitaker and Cooke (1966), Burland et al. (1966) and Poulos (1972) proposed a priori methods to predict the load-settlement curve/plot till failure load. Different methods for the estimating the ultimate pile capacity based on a pile load test to failure are proposed by Brinch-Hansen (1963), Chin (1971a&b and Chin and Vail, 1973), Davisson (1972), Fellenius (1989) etc. These methods relate to different principles, such as limiting maximum settlement and ratio of settlement to load. Chin (1971) and Brinch-Hansen (1963) relate to the shape of the load-settlement curve and hence can be conceptually used for determining the ultimate capacity.

Axial load transfer curves were initially put forth by Seed and Reese (1957) and Coyle and Reese (1966) way back in the 1950s. Based on different degrees of complexity and number of soil parameters different curve types are reported in the literature, based on theory, on experience, or on both. They are in general developed for specific ground and pile types (Armaleh and Desai 1987; Fleming 1992; Frank 1974, 1985; Hirayama 1990; Kraft et al. 1981; Liu et al. 2004; Randolph and Wroth 1978; Vijayvergiya 1977; Wang et al. 2012; Zhang et al. 2010, Bohn et al. 2016). The stiffness of the load transfer curves is either derived from measured soil deformation parameter or from an empirical relation. Randolph and Wroth (1978) and Fleming (1992) used an equivalent deformation modulus or oedometer modulus based on correlations with measured soil resistance parameters for the response of pile tip. The linear curves of Randolph correspond to elastic estimation with the use of usual correlations to determine the stiffness from the linear portion of the curve. Hirayama (1990) employed hyperbolic functions to model side and base load transfer applicable for bored piles in sands and clays. The parameters used for defining these functions were based on the results of conventional in-situ and laboratory tests. Fleming (1992) suggested method to determine interaction parameters by back-analysis of pile load test data.

2. PROBLEM DEFINITION

Estimating ultimate resistance, initial stiffness of shaft and base of pile based on load displacement from pile load test is important as it permits verification of the a-priori predictions based on geotechnical parameters, geometry (shape, length and diameter), construction methodology and other uncertainties involved at site during installation of the pile. The present research addresses the above problem of estimating ultimate shaft and base resistances and the initial shaft and base stiffness's from initial pile load test at site. A single pile of diameter, d, and length, L (Figure 1) and with load, P, is considered. For this analysis, the pile soil system is modelled in terms of Winkler type model with different non-linear responses for the shaft-soil and base resistances.

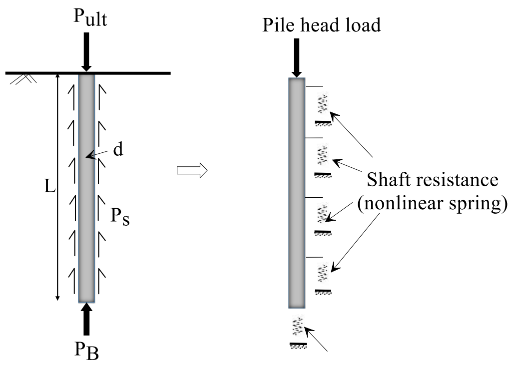

Figure 1 Definition sketch of a single rigid pile system

3. FORMULATION AND COMPUTATION OF PILE RESPONSE

A rigid pile subjected to axial loading derives its resistance from shaft and base of the pile. Figures 2a, b and c represent idealized representations (assumed to be hyperbolic) of load test results of a rigid pile, shaft (pile-soil) and base resistances respectively. The slopes of the curves 'A', 'B', and 'C' represent the stiffness modulus, k_p, of the rigid pile, k_τ of shaft and k_b base stiffness's respectively. P_u, τ_{max} and q_u are the ultimate load on pile, ultimate shaft and base resistances, respectively.

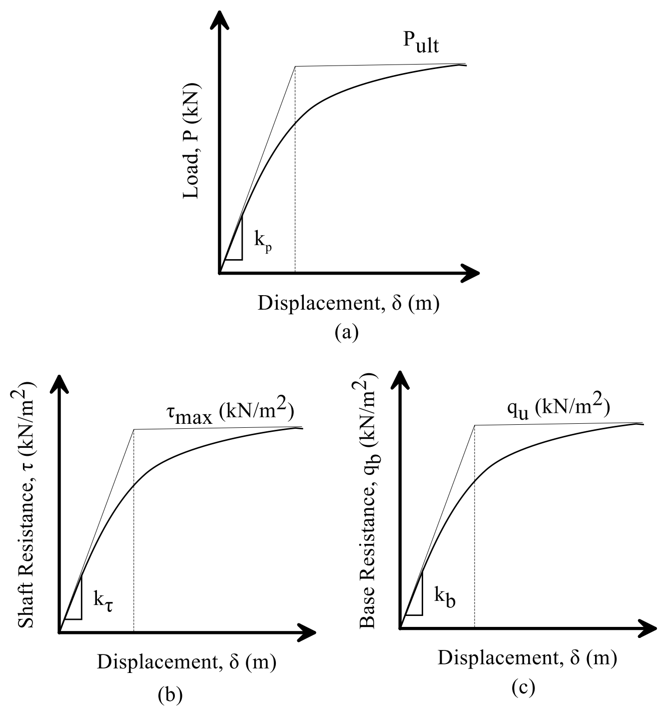

Figure 2 (a) Typical pile load-settlement curve; Assumed (b) Shaft and (c) Base Resistance curves

The load, P, in the pile at a displacement, δ, is expressed as the sum of the resistances mobilized along the shaft and the base as

$$\begin{aligned} P &= k_p.\delta \\ &= \pi.d.L.\tau + \frac{\pi d^2}{4}.q_b \\ &= \pi.d.L.k_\tau.\delta + \frac{\pi d^2}{4}.k_b.\delta \end{aligned} \qquad [1]$$

where τ and q_b are the shaft and base resistances respectively.

Simplifying the stiffness, k_p, is expressed in terms of shaft and base stiffnesses as

$$k_p = \pi.d.L.k_\tau + \frac{\pi.d^2}{4}.k_b \qquad [2]$$

Normalizing Equation (2) with respect to the pile's cross-sectional area ($\pi d^2/4$)

$$k_p^* = \frac{4.L}{d}k_\tau + .k_b \qquad [3]$$

where $k_p^* = k_p / \left(\pi.d^2/4\right)$- normalized stiffness of the pile, k_τ and k_b are shaft and base stiffnesses respectively,

Ultimate load capacity, P_u, of a rigid pile is the sum of ultimate shaft and base resistances as

$$P_u = P_s + P_b \qquad [4]$$

$$P_u = \pi.d.L.\tau_{max} + \frac{\pi d^2}{4}.q_u \qquad [5]$$

Where P_s & P_b are the ultimate shaft and base resistances of pile. Normalizing the Equation (5) with $\pi d^2/4$

$$P_u^* = \frac{4.L}{d}\tau_{max} + q_u \qquad [6]$$

where $P^*_u = P_u/(\pi d^2/4)$.

The shaft resistance, τ, corresponding to a displacement, δ, can be expressed in hyperbolic relation as

$$\tau = \frac{k_\tau.\delta}{1 + \frac{k_\tau.\delta}{\tau_{max}}} \qquad [7]$$

Similarly, the base resistance, q_b, corresponding to a displacement, δ, can be expressed in hyperbolic relation as

$$q_b = \frac{k_b.\delta}{1 + \frac{k_b.\delta}{q_u}} \qquad [8]$$

The load, P_1, corresponding to displacement, δ_1, during loading is expressed in terms of the parameters defined already as

$$P_1 = \pi.d.L.\frac{k_\tau.\delta_1}{1 + \frac{k_\tau.\delta_1}{\tau_{max}}} + \frac{\pi.d^2}{4}.\frac{k_b.\delta_1}{1 + \frac{k_b.\delta_1}{q_u}} \qquad [9]$$

Normalizing Equation (9) by $\pi d^2/4$, and simplifying one gets

$$\frac{P_1^*}{\delta_1} = \frac{4.L}{d}\frac{k_\tau.\tau_{max}}{\tau_{max} + k_\tau.\delta_1} + \frac{k_b.q_u}{q_u + k_b.\delta_1} \qquad [10]$$

Substituting for k_b and q_u from Equations (3) and (6), Equation 8 is modified to

$$\frac{P_1^*}{\delta_1} = \frac{4.L}{d}\frac{k_\tau.\tau_{max}}{\tau_{max} + k_\tau.\delta_1} + \frac{1}{A_1}\left\{\left(k_p^* - \frac{4.L}{d}k_\tau\right).\left(P_u^* - \frac{4.L}{d}\tau_{max}\right)\right\} \qquad [11]$$

where $A_1 = \left(P_u^* - \frac{4.L}{d}\tau_{max}\right) + \left(k_p^* - \frac{4.L}{d}k_\tau\right).\delta_1$

or

$$\frac{P_1^*}{\delta_1} - \frac{k_p^*.P_u^*}{A_1} = \left\{\frac{4.L}{d}\frac{.\tau_{max}}{\tau_{max} + k_\tau.\delta_1} - \frac{4.L}{d.A_1}.P_u^* + 16\left(\frac{.L}{d}\right)^2\frac{\tau_{max}}{A_1}\right\}.k_\tau - \frac{4.L}{d.A_1}\tau_{max}.k_p^* \qquad [12]$$

Expressing in simple terms, Equation (12) becomes

$$d_1 = C_1.k_\tau + C_2.\tau_{max} \qquad [13]$$

where $d_1 = \dfrac{P_1^*}{\delta_1} - \dfrac{k_p^*.P_u^*}{A_1}$,

$$C_1 = \frac{4.L}{d}.\left\{\frac{\tau_{\max}}{\tau_{\max} + k_\tau.\delta_1} - \frac{P_u^*}{A_1}.+\frac{4.L}{d}\frac{\tau_{\max}}{A_1}\right\} \;\&$$

$$C_2 = -\frac{4.L}{d.A_1}.k_p^*$$

Similarly P_2 is evaluated for displacement δ_2 as

$$\frac{P_2^*}{\delta_2} = \frac{4.L}{d}\frac{k_\tau.\tau_{\max}}{\tau_{\max} + k_\tau.\delta_2} + \frac{k_b.q_u}{q_u + k_b.\delta_2} \quad [14]$$

$$d_2 = C_3.k_\tau + C_4.\tau_{\max} \quad [15]$$

where $d_2 = \dfrac{P_2^*}{\delta_2} - \dfrac{k_p^*.P_u^*}{A_2}$,

$$A_2 = \left(P_u^* - \frac{4.L}{d}\tau_{\max}\right) + \left(k_p^* - \frac{4.L}{d}k_\tau\right).\delta_2 ,$$

$$C_3 = \frac{4.L}{d}.\left\{\frac{\tau_{\max}}{\tau_{\max} + k_\tau.\delta_2} - \frac{P_u^*}{A_2}.+\frac{4.L}{d}\frac{\tau_{\max}}{A_2}\right\} \;\&\; C_4 = -\frac{4.L}{d.A_2}.k_p^*$$

Equations (13) and (15) are solved to get k_τ as

$$k_\tau = \frac{C_4 d_1 - C_2 d_2}{C_1 C_4 - C_2 C_3} \quad [16]$$

Since the ultimate shaft resistance, P_s, is often mobilized at relatively smaller displacement than δ_2, Equation (15) can be simplified considering full mobilization of ultimate shaft resistance as

$$P_2^* = \frac{4.L}{d}.\tau_{\max} + \frac{k_b.q_u.\delta_2}{q_u + k_b.\delta_2} \quad [17]$$

Substituting for the values of q_u and k_b in Equation (17), τ_{max} is obtained as

$$\tau_{\max} = \frac{\left\{P_2^*.A_2 - \left(\frac{k_p^*}{k_\tau} - \frac{4L}{d}\right).P_u^*.k_\tau.\delta_2\right\}}{\left\{\frac{4L}{d}\left(A2 - k_p^*.\delta_2 + \frac{4L}{d}.k_\tau.\delta_2\right)\right\}} \quad [18]$$

Equations (13) and (18) are solved by iterative process for k_τ, k_b, τ_{max} and q_u.

3.1 Estimation of P_U and k_P based on Chin (1971a)

The ultimate pile load capacity, P_u, and pile stiffness modulus, k_p, are the two key parameters required to determine the four unknown parameters (k_τ, k_b, τ_{max} and q_u) of interest. From the available methods during literature review to define ultimate capacity and stiffness modulus, Chin's method (1971a) is adopted in the present analysis as Chin's (1971a) method assumes the relationship between load, P_u, and settlement, S, (Figure 3) of a pile as hyperbolic, which co-relates to the present study.

According to Chin's (1971) method, the ratio of S/P_u plotted against S would be a straight line fitted to the points (Figure 3). The inverse of the intercept, C_2, and of the slope, C_1, of the fitted line give respectively the initial pile stiffness, k_p, and the ultimate pile load, P_u.

$$\frac{S}{P_u} = C_1.S + C_2 \quad [17]$$

$$P_u = 1/C_1 \quad [18]$$

$$k_p = 1/C_2 \quad [19]$$

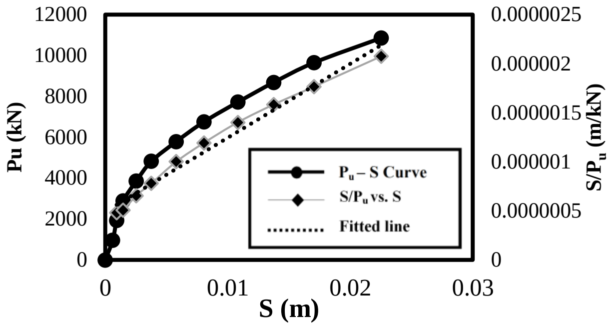

Figure 3 Ultimate failure load according to Chin's extrapolation for Test pile no. 1 at Dharaikuri

3.2 SPT-N based methods for computing Axial Pile capacity

The shaft and tip resistances for bored piles in sandy soils can be computed using SPT based correlations available in the literature. Many researchers propose empirical methods derived from back-analysis of databases of pile loading tests in correlation with N value. The end bearing capacity is assumed to be proportional to a representative N_e value around the pile base:

$$q_u = \beta.N_e \quad [20]$$

The limit skin friction stress at a given depth is proportional to the N value at this depth:

$$\tau_{\max} = \eta.N \quad [21]$$

β and η have the unit of stress and are respectively called tip resistance factor and skin friction factor. Methods 1-9 summarized in Table 1.

Table 1 Empirical correlations for Shaft and Base resistances from SPT-N

Sl. No.	References	η (kPa)	β (kPa)
1	IS Code -2911	$\frac{13..L}{B}$	2
2	Bazaraa & Kurkur (1986)	135 if D ≤ 0.5 m 270xD else (B in m)	0.67 if D ≤ 0.5 m 1.34xD else(B in m)
3	Decourt (1982)	400 in sand 250 in residual silty sand	τ_{max} =10x(N/3+1) (in kPa)
4	Lopes & Laprovitera (1988)	98.4 in sand 87.0 in silty sand	1.62 in sand 1.94 in silty sand
5	Meyerhof (1976)	120	1
6	Shioi & Fukui (1982)	100	1
7	Aoki & Veloso (1975)	286 in sand 228 in silty sand	2.00 in sand 2.28 in silty sand
8	Reese & O'Neill (1989)	60 if B=0.52-1.27 76/D if B>1.27 m (D in m)	3
9	Robert (1997)	115	1.90

The above mentioned empirical correlations are utilized for comparing the estimated results of the method proposed in this paper.

4. APPLICATION OF THE PROPOSED METHOD

The proposed method is applied to load displacement plots of pile load test results obtained from initial load tests carried out at major bridge locations for the on-going National Highway NH31D. The soil profiles at these three locations are nearly similar with non-cohesive silty sand up to 10 m followed by fine sand. The ground water table is close to ground level with seasonal variation of 5 to 6m. The detailed properties of the soil strata in the load test sites are reported in Table 2. The pile capacity is estimated using the procedure described in IS 2911 Part 1 section 2 for piles installed in non-cohesive soils. The estimated shaft and end bearing resistance for the dimensions of the pile are presented in Table 1. The Initial load tests were conducted on the test piles to validate the estimated safe axial capacities. The load displacement data available is used to estimate initial stiffness modulus, k_p, of the pile for the ultimate load P_u,on pile. The data is analyzed to estimate normalized shaft and base stiffness, k_τ and k_b, respectively corresponding to ultimate shaft, τ_{max}, and base resistances, q_u. For estimating the base and shaft stiffness's Equations (1) to (18) are used to perform regression analysis. The detailed explanation for each test pile location is briefed below.

Table 2 Geotechnical Properties of the load test site

Sl. No	Site Location	Depth	Stratum description	Shear Parameters			N	N''	γ (kN/m^3)
				C_u (kPa)	φ	φ*			
1.	Dharaikuri, WB	0-2.4m	Top fill of firm/soft clayey silt with sand and gravel	-	-	-	7	7	17
		2.4-5.5m	Medium Dense Sand with traces of gravels	0	34	31	34	30	18
		5.5-7m	Very Dense Sand with traces of gravels & pebbles	0	34	31	88	55	20
		7-35m	Gravel Pebbles and boulders in sandy matrix	0	34	34	100	51	20
2.	Mahananda, WB	0-6m	Silty Clay/Clayey Silt with medium plasticity (CI/MI)	40	0	0	8	8	17.5
		6-10.88m	Medium Dense to Dense Silty/Clayey Sand (SM/SC/SW/SP)	0	34	31	30	23	18.5
		10.88-35m	Very Dense Silty/ Clayey Sand with gravels (SM/SC/SW/SP)	0	34	31	100	49	20
3.	Karala, WB	0-5.5m	Loose Silty/Clayey Sand (SM/SC/SW/SP)	0	28	25	6	8	17.5
		5.5-10m	Medium Dense to Dense silty/clayey Sand (SM/SC/SW/SP)	0	34	31	31	24	19
		10-35m	Very Dense Silty/ Clayey Sand with gravels (SM/SC/SW/SP)	0	34	31	81	38	20

C_u- Cohesion, φ- Friction angle, φ* Reduced friction angle, N-Observed SPT-N Value, N''-Corrected SPT-N, γ-Unit weight of the soil

Case study 1: The load displacement plot generated for initial pile load test (PLT) carried out on a pile for a major bridge at Dharaikuri of West Bengal is shown in Figure 4. The pile capacity is estimated using the procedure described in IS 2911 Part 1 section 2 for piles installed in non-cohesive soils. The estimated shaft and end bearing resistance for the dimensions of the pile are 2942.2 kN and 1357.8 kN respectively.

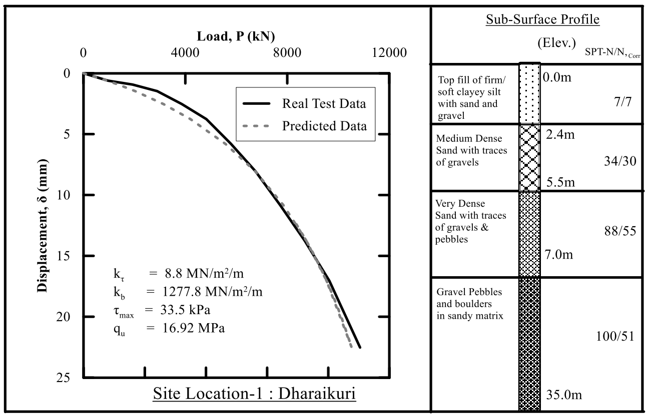

Figure 4 Comparison of measured and predicted load-displacement curves for pile load test (PLT) at Dharaikuri, W. Bengal

The length and diameter of the pile installed are 24 m and 1 m respectively. Using linear regression over the approximated linear portion of the curve, the ultimate pile capacity, P_u is obtained as 15.8MN (Since, the slope of the fitted line, C_1 = 6.326E-05) and the initial stiffness, k_p is 1666 MN/m (Since, the intercept C_2 = 6E-07) from Figure 3. The shaft and base resistance parameters, k_τ, k_b, τ_{max} and q_u, estimated based on the proposed method are 8.79 MN/m^3; 1,277.84 MN/m^3; 33.5 kPa and 16.92 MPa respectively. The load – settlement curve predicted based on the estimated parameters compares very well as is to be expected.

For the present site, the skin and base resistances values attained show higher base response than compared to shaft resistance, this is accounts to pile interaction with Gravelly pebbles.

Case study 2: Figure 5 presents similar analysis, results and comparison of measured and predicted load-settlement curve for initial pile load test (PLT) carried out at a major bridge location across River Mahananda. The length and diameter of the pile installed are 24m and 1m respectively. The ultimate load and initial stiffness modulus, k_p, of the pile are computed as 16.34 MN and 3,125MN/m respectively based on which the corresponding values of the parameters k_τ, k_b, τ_{max} and q_u contributing to the pile response are 25.0 MN/m^3; 1,578.8 MN/m^3; 56 kPa, and 16.3 MPa respectively. In this case, the shaft and base responses fairly contribute equally when compared to other two cases.

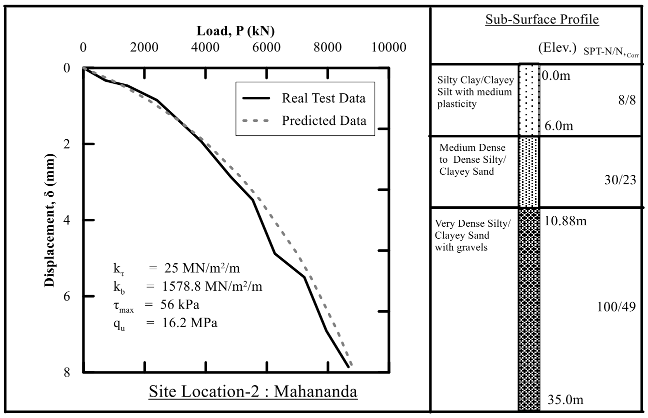

Figure 5 Comparison of measured and predicted load-displacement curves for pile load test (PLT) at Mahananda, W. Bengal

Case study 3: Similar results as the above two case studies are presented in Figure 6 comparing measured and predicted load-settlement curves for pile load test (PLT) carried out at a major bridge foundation across River, Karala. The length and diameter of the pile installed are 25m and 1m respectively.

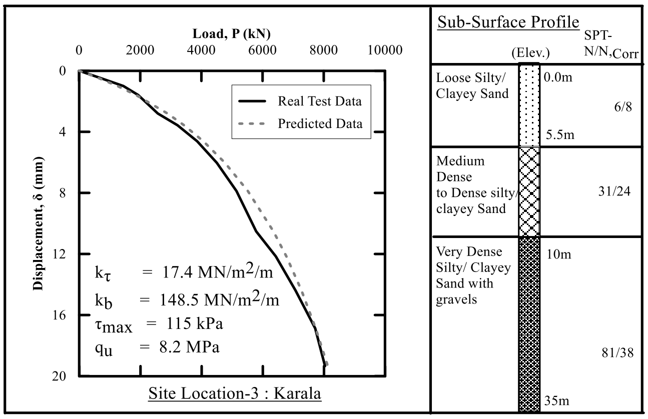

Figure 6 Comparison of measured and predicted load-displacement curves for pile load test (PLT) at Karala, W. Bengal

The ultimate load and initial stiffness modulus, k_p are computed as 15 MN and 1428 MN/m respectively. The parameters k_τ, k_b, τ_{max} and q_u contributing to the pile response in resisting loads are 17.4 MN/m^3; 148.5 MN/m^3; 115 kPa, and 8.178 MPa respectively. The load-displacement plot from PLT is compared with estimated data, and matches well with the actual.

As can be seen from the soil profile, major portion of the pile length of about 15 m is surrounded by dense sand which resulted in higher skin frictional resistance and lower tip resistance.

5. RESULTS AND DISCUSSION

The main purpose of this study obtains the shaft and base response parameters i.e. k_τ, k_b, τ_{max} and q_u based on load-settlement curve of a PLT test. Figures 4-6 present some of the comparative load-displacement curves along with relevant sub-surface profiles. A close match in the load-displacement curves are obtained for all the three sites of the present study. These comparisons confirm that non-linear load transfer functions employed in the present study successfully map the load displacement behaviour in the field.

Table 3 summarizes the data from the three test sites and compares the estimated results with the shaft and base resistance values, calculated using various empirical correlations. As can be observed, the values obtained from the proposed method are will comparable with the correlations detailed in IS 2911 (Part 1 Sec 2) i.e. pertaining to SPT-N. Along with IS Code 2911 method, the results obtained are well in agreement with the methods of Aoki & Veloso (1975), Decourt (1982) and Bazaraa & Kurkur (1986). The values attained using the methods Shioi & Fukui (1982), Lopes & Laprovitera (1988) and Robert (1997) are significantly less than the present method implying under prediction of the pile bearing capacity.

Meyerhof's method under-predicts in most of the cases with an average value of 2.8 for the ratio μ. The pessimistic prediction which may justify the large use of this method was already highlighted by many authors working on similar databases. According to Table 1, the factor β and η of Reese's method for bored piles seem to be lowest, resulting in underestimation of the shaft and base responses compared to the present proposed.

The practical implication of the method is thus made evident from the comparisons made and application of the method to available test data. Future work will compare the shaft and base responses derived from avaialble numerical tools to the values estimated using this study.

Table 3 Summary of predicted results using the proposed method and empirical correlations

Test Location	**Site Location-1: Dharaikuri**		**Site Location-2: Mahananda**		**Site Location-3: Karala**		**Site Location-1: Dharaikuri**		**Site Location-2: Mahananda**		**Site Location-3: Karala**	
Method	τ_{max} (kPa)	q_u (MPa)	τ_{max} (kPa)	q_u (kPa)	τ_{max} (kPa)	q_u (kPa)	$\frac{\tau_{max,est}}{\tau_{max,m}}$	$\frac{q_{u,est}}{q_{u,m}}$	$\frac{\tau_{max,est}}{\tau_{max,m}}$	$\frac{q_{u,est}}{q_{u,m}}$	$\frac{\tau_{max,est}}{\tau_{max,m}}$	$\frac{q_{u,est}}{q_{u,m}}$
Present Method	34	16.9	56	16200	115	8200	1.0	1.0	1.0	1.0	1.0	1.0
IS Code 2911	88	15.9	67	15288	57	11856	0.4	1.1	0.8	1.1	2.0	0.7
Bazaraa & Kurkur (1986)	59	13.8	45	13230	38	10260	0.6	1.2	1.2	1.2	3.0	0.8
Decourt (1982)	157	12.75	122	12250	105	9500	0.2	1.3	0.5	1.3	1.1	0.9
Lopes & Laprovitera (1988)	86	44.37	65	4263	55	3306	0.4	3.8	0.9	3.8	2.1	2.5
Meyerhof (1976)	44	61.20	33	5880	28	4560	0.8	2.8	1.7	2.8	4.0	1.8
Shioi & Fukui (1982)	44	51.0	33	4900	28	3800	0.8	3.3	1.7	3.3	4.0	2.2
Aoki & Veloso (1975)	101	11.63	76	11172	65	8664	0.3	1.5	0.7	1.5	1.8	0.9
Reese & O'Neill (1989)	146	3.06	110	2940	94	2280	0.2	5.5	0.5	5.5	1.2	3.6
Robert (1997)	84	5.86	64	5635	54	4370	0.4	2.9	0.9	2.9	2.1	1.9

6. CONCLUSION

This paper reported a simple method based on the analysis of initial pile load test data for the purpose of determining the shaft-soil and base stiffnesses and the ultimate shaft and base resistances. The proposed method was applied to few test data, and the estimated shaft and base response parameters could accurately interpret the load-transfer behaviour. This was evident as the predicted load – displacement plots based on the computed results compare well with the measured load - settlement curves. Hence, It is now possible to estimate the average shaft and base stiffnesses and resistances of the pile as installed from initial load test and compare the predictions with assumed values. A large data base can be built for all such cases which may help in revising the Indian Standard (IS) Code.

7. ACKNOWLEDGMENTS

Authors acknowledge Dr. Vidyaranya Bandi for providing pile load test results to carry out the study.

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