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Standard Penetration Tests and Pile Behaviour under Lateral Loads in Clay

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Introduction

The bearing capacity and settlement of foundations under vertical loads in cohesionless soils can conveniently be estimated directly from the results of standard penetration tests for shallow and deep foundations (Terzaghi and Peck, 1967; Meyerhof, 1956; 1959; 1965; 1976). Very recently, Meyerhof (1995b) presented a method to estimate the behaviour of laterally loaded piles in cohesionless soils through SPT-N values. In that method, the existing correlations between the pressuremeter limit pressure p_l and standard penetration resistance N values are used.

In this paper, correlations between the results of static cone penetrometer, pressuremeter and standard penetration tests in cohesive soils are used to develop simple methods of analysis for preliminary estimates of ultimate lateral resistance and ground line displacements in rigid and flexible piles directly from standard penetration tests and pile geometry. The validity of the proposed method is examined by comparing with the published field test data on prototype piles.

Theory

Lateral Resistance

If a vertical flexible pile of diameter B and depth D is driven to full embedment in cohesive soil of average unit weight γ and standard penetration resistance N (blows / 0.3 m) and is subjected at its free-head to an ultimate horizontal load Q_n applied at ground level, it has been shown

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(Meyerhof et al., 1981; Meyerhof and Sastry, 1987b) that, in the absence of structural failure,

$$Q_n = 0.4\,\gamma\,B\,D_{eu}^2\,K_c \leq Q_{nl} \tag{1}$$

where,

$$Q_{nl} = 0.4\,p_l\,B\,D_{eu} \quad \text{for } D_{eu}/D_c \geq 1 \tag{2}$$

where, D_c = critical depth, which is about 5B in clays (Meyerhof, 1995a).

D_{eu} = ultimate effective depth of an equivalent rigid pile (Meyerhof et al., 1988),

K_c = resultant net lateral soil pressure coefficient for the shaft of an equivalent rigid pile; and

p_l = limit pressure obtained from the pressuremeter tests (Baguelin et al., 1978).

If the load is acting at some eccentricity e, the above Eqns. 1 and 2 are to be multiplied by the reduction factor due to the moment, r_c (Meyerhof et al., 1981) in Eqn. 3.

$$r_c = 1/(1 + 1.9\,e/D) \tag{3}$$

The ultimate effective depth ratio is (Meyerhof et al., 1988),

$$D_{eu}/D = 1.5\,K_{rc}^{0.12} \leq 1 \tag{4}$$

for free-head piles. For fixed-head piles, the value of D_{eu}/D is about one half the value for free-head piles. The relative stiffness factor (K_{rc}) is given by (Poulos and Davis, 1980)

$$K_{rc} = E_p\,I_p/E_s\,D^4 \tag{5}$$

where E_pI_p = flexural rigidity of the pile

E_s = average soil modulus along the embedded length of the pile.

The empirical relationship between p_l and N in cohesionless soils (Baguelin et al., 1978) is,

$$p_l = 50\,N\,(kPa) \tag{6}$$

But no such simple correlations were reported in cohesive soils. However, there are well established correlations between SPT-N values and cone penetration resistance, q_c. Similarly there are correlations between limit pressure, p_l measured with pressuremeter and cone penetration resistance, q_c. Hemce, logically correlations can be established between SPT-N values and pressuremeter limit pressure, p_l.

Kulhawy and Mayne (1990) quoted Szechy and Varga (1978) for identifying the consistency of cohesive soils using both SPT-N and CPT. The correlations are presented in Table 1. From the values presented, relationship between N and q_c can be represented by Eqn. 7

$$q_c = 200\,N\,(kPa) \tag{7}$$

The relationship between limit pressure, p_l obtained from pressuremeter and cone penetration resistance, q_c (Baguelin et al., 1978) is presented in Eqn. 8

$$q_c = p_l \quad \text{for soft clays} \tag{8a}$$

$$q_c = 2.5\,p_l \quad \text{for firm to soft clays} \tag{8b}$$

$$q_c = 4.0\,p_l \quad \text{for very stiff to hard clays} \tag{8c}$$

Table 1 : Consistency of Cohesive Soils using SPT and CPT

N value	Cone tip resistance q_c/p_a	Consistency	Consistency index
< 2	< 5	very soft	< 0.5
2 to 8	5 to 15	soft to medium	0.5 to 0.75
8 to 15	15 to 30	stiff	0.75 to 1.0
15 to 30	30 to 60	very stiff	1.0 to 1.5
> 30	> 60	hard	> 1.5

Note : p_a = atmospheric pressure

Thus for different consistencies of clays, direct relationship between p_l and N can be established using Eqns. 7 and 8, as follows (in kPa) :

$$p_l = 200\,N \quad \text{for soft clays} \tag{9a}$$

$$p_l = 80\,N \quad \text{for firm to stiff clays} \tag{9b}$$

$$p_l = 50\,N \quad \text{for very stiff to hard clays} \tag{9c}$$

Substituting these average relationships (Eqns. 9) in Eqn. 2, the ultimate lateral resistance of a free-head pile for $D_{eu}/D_e \geq 1$ is, approximately,

$$Q_{nl} = 80\,N\,B\,D_{eu}\,(\text{kN}) \quad \text{for very soft clays} \tag{10a}$$

$$Q_{nl} = 32\,N\,B\,D_{eu}\,(\text{kN}) \quad \text{for firm to stiff clays} \tag{10b}$$

$$Q_{nl} = 20\,N\,B\,D_{eu}\,(\text{kN}) \quad \text{for very stiff to hard clays} \tag{10c}$$

If the values of the $D_{eu}/D_c \leq 1$, the above capacities are to be reduced by a factor of D_{eu}/D_c (Meyerhof, 1995b).

For rigid piles, K_{rc} greater than about 0.01 (Poulos and Davis, 1980), Eqn. 10 simplifies to $\left(\text{for } D/D_c \geq 1\right)$:

$$Q_{nl} = 80\,N\,B\,D\,(\text{kN}) \quad \text{for very soft clays} \tag{11a}$$

$$Q_{nl} = 32\,N\,B\,D\,(\text{kN}) \quad \text{for firm to stiff clays} \tag{11b}$$

$$Q_{nl} = 20\,N\,B\,D\,(\text{kN}) \quad \text{for very stiff to hard clays} \tag{11c}$$

If the values of the $D/D_c \leq 1$ the above capacities are to be reduced by a factor of D/D_c (Meyerhof, 1995b). Thus Q_n can be directly calculated from the standard penetration resistance and pile geometry. For bored and non-displacement free-head piles the ultimate lateral resistance, Q_n is frequently about the same as those given above for displacement piles (Broms, 1964; Sastry and Meyerhof, 1987a). On the other hand, for fixed-head piles, the capacities are twice those of the corresponding free-head piles (Broms, 1964).

Further, for a pile group, the ultimate lateral resistance can be estimated from the smaller of either the sum of the above mentioned ultimate lateral resistance of the individual piles or the ultimate lateral block resistance of an equivalent pier consisting of the pile and enclosed soil mass of width B and effective depth.

Displacements and Moments

The lateral ground-line displacement, y_h of a vertically fully embedded flexible pile under horizontal load, Q (Q less than ultimate load) acting at ground level can be readily estimated from an equivalent rigid pile when (Poulos and Davis, 1981; Meyerhof et al., 1988)

$$y_h = Q\, I_{yH} / E_s\, D_c \tag{14}$$

where I_{yH} = elastic influence factor, which can be taken as 4 (Poulos and Davis, 1980), and

E_s = average soil modulus.

The theoratical relationship between D_e/D and K_{rc} for free-head pile is, approximately, given by (Meyerhof et al., 1988):

$$D_e/D = 2.1\, K_{rc}^{0.2} \tag{15}$$

and about one half this value for fixed piles.

The average value of E_s is given below from the established correlations between E_s and p_l shown (Baguelin et al., 1978) as follows:

$$E_s = 20\, p_l \quad \text{for soft and firm clays} \tag{16a}$$

$$E_s = 30\, p_l \quad \text{for soft and firm clays} \tag{16b}$$

and, after substituting Eqn. 9 into Eqn. 16,

$$E_s = 4400\, N \quad \text{for soft clays} \tag{17a}$$

$$E_s = 1600\, N \quad \text{for firm, stiff and hard clays} \tag{17b}$$

Thus the ground line deflection of a pile subjected to a load Q (kN) at ground level can be approximated using Eqns. 14 and 17 as:

$$y_h = Q/100\, N\, D_e \quad \text{for soft clays} \tag{18a}$$

$$y_h = Q/400\, N\, D_e \quad \text{for firm, stiff and hard clays} \tag{18b}$$

For bored piles, the values of deflections will be 1.5 to 3 times the deflections calculated for driven piles (Meyerhof and Sastry, 1987a).

The lateral displacements of pile groups under horizontal loads can be estimated from those of corresponding single piles under the same load per pile by using group displacement ratio (Poulos and Davis, 1980) or by using an equivalent rigid pier of width B and elastic effective depth D_e for a free-head group.

Field Cases

For the validity of the proposed method suggested above, a few test results from published field case were considered. Bhushan et al. (1979) reported the test results of concrete drilled piers in very stiff clays. Five test results of straight piers 2, 4, 6, 7, 8 tested in site A and B are considered in this analysis (see Table 2 for details). The pier 2, tested in the site A, the average undrained shear strength (c_u) of the soil was 260 kN/m^2. The E_s value is calculated using the relation $E_s = 70\, c_u$ (Poulos and Davis, 1980). The SPT-N values are 19, 34, 35, 44, 20 and 20 over a depth of 4.27 m (14 ft). GAI Consultants (1982) have analysed the test results in site B and they suggested the value of E_s as 17225 kN/m^2. The SPT-N values are 19, 30, 32, 22, 43, 24 and 31 in the test site B over a depth of 4.27 m (14 ft). For a rigid pile, N values are averaged along the pile embedment depth D. On the other hand, for flexible pile, the field N values are averaged along an equivalent rigid pile of ultimate effective depth D_{eu} (Meyerhof, 1995b).

Using the Eqn. 11c for very stiff clays, capacities were calculated and are presented in Table 2. It can be seen that the capacities are very close to the predicted ones. For calculating the deflections, these were assumed as bored piles and the calculated deflections, using Eqn. 18b, were multiplied by a factor of 1.5. It can be seen that there is a good comparison (refer Table 3).

Conclusion

On the basis of correlations between the results of pressuremeter, static cone penetrometer and standard penetration resistance tests in clays, simple methods of analysis have been developed to estimate the ultimate lateral resistance and ground-level displacements directly from standard penetration tests and pile geometry. For the corresponding flexible piles, similar analysis have been proposed using the equivalent rigid piles. Comparisons of lateral load tests on several piles in the field show excellent agreement. However, the methods proposed are to be verified with other large scale tests on piles in a variety of soil conditions.

The methods proposed in this paper are very useful for preliminary estimation of capacities in the field.

Table 2 : Comparison of Ultimate Capacities

Pile details	Pile geometry			e (m)	r_c	Average N (blows / 0.3 m)	D_c (m)	D_{eu} (m)	Ultimate load Q_n (kN) actual	Ultimate load Q_n (kN) predicted	Reference
	B (m)	D (m)	K_{rc}								
Pier No. 2 Site A	1.22	4.58	0.0292	0.23	0.91	28	6.1	4.48	1850	2050	Bhushan et al. (1979)
Pier No. 4 Site B	1.22	3.81	0.6406	0.23	0.89	28	6.1	3.81	1730	1450	Bhushan et al. (1979)
Pier No. 6 Site B	1.22	4.73	0.2712	0.23	0.91	29	6.1	4.73	2050	2370	Bhushan et al. (1979)
Pier No. 7 Site B	0.61	2.75	0.1490	0.23	0.86	29	3.1	2.75	715	750	Bhushan et al. (1979)
Pier No. 8 Site B	0.61	4.73	0.0169	0.23	0.91	29	3.1	4.35	1420	1400	Bhushan et al. (1979)

Table 3 : Comparison of Ultimate Capacities

Pile details	Pile geometry			Average N (blows / 0.3 m)	D_e (m)	Load Q (kN)	y_h (mm) observed	y_h (mm) predicted	Reference
	B (m)	D (m)	K_{rc}						
Pier No. 2 Site A	1.22	4.58	0.0292	28	4.58	1335	30	39	Bhushan et al. (1979)
Pier No. 4 Site B	1.22	3.81	0.6406	28	3.81	1110	38	39	Bhushan et al. (1979)
Pier No. 6 Site B	1.22	4.73	0.2712	29	4.73	890	19	24	Bhushan et al. (1979)
Pier No. 7 Site B	0.61	2.75	0.1490	29	2.75	535	20	25	Bhushan et al. (1979)
Pier No. 8 Site B	0.61	4.73	0.0169	29	4.73	890	30	25	Bhushan et al. (1979)

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