

PART VI
Appendix G

Other Methods of Estimating Ultimate Axial Capacity of Piles and Drilled Shafts, and Design Charts for Settlement

G.1 Method Based on the Standard Penetration Test (Canadian Foundation Engineering Manual):

This method is based on N-values obtained from Standard Penetration Test (SPT). This method has been described in Canadian Foundation Engineering Manual published by Canadian Geotechnical Society (CGS, 1985).

The capacity of a single pile in granular soils can be estimated from the results of SPT using the following expression as suggested by Meyerhof (1976).

$$R = mNA_t + nN'DA_s \quad (6.G.1)$$

Where,

- R = pile capacity (N)
- m, n = constants depending on type of pile (driven or cast-in-situ bored piles)
- N = SPT index at the pile toe
- A_t = pile toe area
- N' = average SPT index along the pile
- D = pile embedment length
- A_s = pile unit shaft area

The Standard Penetration Test is subject to a multitude of errors, and a lot of care must be exercised when using test results. For this reason, in this method a minimum factor of safety of 4 should be applied to calculate allowable capacity of a drilled shaft or bored pile.

G.2 Method Based on the Theory of Plasticity (Canadian Foundation Engineering Manual)

In this method the capacity of a single pile may be determined from the friction angle of the soil by use of the theory of plasticity (or bearing-capacity theory).

The capacity of a pile in a soil of uniform density increases in a linear manner with increase in effective overburden pressure at least to a certain depth called the critical depth. Investigations of single piles indicate that there is very little increase in toe resistance or unit shaft resistance below the critical depth. The ratio of the critical depth to the pile diameter increases with increase in the angle of shearing resistance. For most applications, the ratio ranges between a value of 7 at $\phi' = 30^\circ$ to a value of 22 at $\phi' = 45^\circ$.

The ultimate static resistance, R of a single pile is a function of the sum of the toe and shaft resistance, R_t and R_s , as follows:

$$R = R_t + R_s \quad (6.G.2)$$

Where for toe resistance:

$$R_t = A_t r_t = A_t \sigma'_D N_t \quad (6.G.3)$$

Where,

A_t = cross-sectional area of pile at toe

r_t = unit toe resistance = $\sigma'_D N_t$

σ'_D = unit effective vertical stress at the pile toe = $\gamma'D$ (below the critical depth, $D = D_c$)

γ' = submerged unit weight of soil

D = embedment length of the pile in soil

N_t = bearing-capacity coefficient as recommended by Canadian Foundation Engineering Manual

The expression for shaft resistance is as follows:

$$R_s = \sum_{Z=0}^D A_s r_s = \sum_{Z=0}^D A_s \beta \sigma'_z = \sum_{Z=0}^D A_s M K_s \tan \phi' \sigma'_z \quad (6.G.4)$$

Where,

A_s = shaft area per unit length of pile

r_s = unit shaft resistance along the pile

σ'_z = effective vertical stress at depth z (below the critical depth, $z = D_c$, use σ'_z)

β = shaft resistance coefficient = $K_s M \tan \phi'$

K_s = ratio between the horizontal effective soil stress to the vertical effective soil stress at the pile shaft

$M = \tan \delta' / \tan \phi'$

$\tan \phi'$ = soil friction

$\tan \delta'$ = soil-pile friction

The value of K_s is influenced by the angle of shearing resistance, the method of installation, the compressibility and original state of stress in the ground, and the size and shape of the pile. It increases with the in-situ density and angle of shearing resistance of the soil and with the amount of displacement. It is higher for displacement-type piles than for low-displacement-type piles such as H-piles. For bored piles, the value of K_s is usually assumed equal to the coefficient of earth pressure at rest, K_0 . For driven displacement-type piles, the value of K_s is normally assumed to be twice the value of K_0 .

The value of M ranges from 0.7 to 1.0, depending on the pile material (steel, concrete, wood) and method of installation (Bozozuk et al., 1978). The combined shaft resistance coefficient, β , is generally assumed to range from 0.3 to 0.8, where the lower value is used in clay and silt, and the higher value in coarse and dense soils (Burland, 1973).

Terzaghi and Peck (1967) reported typical values of angle of internal friction for different types of sands which are shown in Table 6.G.1. In the absence of test loading, this method recommends a factor of safety of at least 3 in order to calculate the allowable capacity.

Table 6.G.1: Typical Values of Angle of Internal Friction for Different Types of Sand (after Terzaghi and Peck, 1967)

Type of Sand	Angle of Internal Friction, ϕ'	
	Loose	Dense
Uniform sand, rounded particles	27°	35°
Well graded sand, angular particles	33°	45°
Sandy gravels	35°	50°
Silty sands	27° to 30°	30° to 34°
Inorganic silts	27° to 30°	30° to 35°

G.3 Tomlinson Method

This method of estimating ultimate axial load capacity of a single pile has been described by Tomlinson (1995). In this method, the design ultimate unit skin friction of an individual pile is given by the following expression:

$$q_z = K_s \sigma'_{vo} \tan \delta \quad (6.G.5)$$

Where, K_s = coefficient of horizontal soil stress,

σ'_{vo} = average effective overburden pressure over the length of the soil layer

δ = angle of wall friction

The value of coefficient K_s is related to the coefficient of earth pressure at rest (K_o) and also to the method of installation of the piles. Values of coefficient of horizontal soil stress (K_s) are shown in Table 6.G.2 while values of the angle of pile to soil friction (δ) for various interface conditions are shown in Table 6.G.3.

Table 6.G.2: Values of Coefficient of Horizontal Soil Stress, K_s (after Kulhawy, 1984)

Installation method	K_s/K_o
Driven piles, large displacement	1-2
Driven piles, small displacement	0.75-1.75
Bored and cast-in-place piles	0.71-1.0
Jetted piles	0.5-0.7

Table 6.G.3: Values of Soil Pile Friction Angle (δ) for Various Interface Conditions (after Kulhawy, 1984)

Pile/soil interface condition	Angle of pile to soil friction (δ)
Smooth (coated) steel/sand	$0.5\phi'$ to $0.7\phi'$
Rough (corrugated) steel/sand	$0.7\phi'$ to $0.9\phi'$
Precast concrete/sand	$0.8\phi'$ to $1.0\phi'$
Cast-in-place concrete/sand	$1.0\phi'$
Timber/sand	$0.8\phi'$ to $0.9\phi'$

The equation for estimating ultimate skin friction implies that in a uniform cohesionless soil the unit skin friction continues to increase linearly with increasing depth. This is not the case. Vesic (1970) showed that at some penetration depth between 10 and 20 pile diameters, a peak value of unit skin friction is reached which is not exceeded at greater penetration depths. Research has not yet established whether the peak value is a constant in all conditions, or is related to factors such as soil grain size or angularity. A peak value of 110 kN/m² has been recommended by Tomlinson (1995) for straight-sided piles.

The base resistance is obtained from the following equation:

$$Q_b = q_b A_b = N_q \sigma'_{vo} A_b \quad (6.G.6)$$

Where, N_q = bearing capacity coefficient

σ'_{vo} = average effective overburden pressure over the length of the soil layer

A_b = base area of pile

Comparisons of observed base resistances of piles by Nordlund (1963) and Vesic (1964) have shown that N_q values established by Berezantsev (1961) which take into account the depth to width ratio of the pile most nearly conform to practical criteria of pile failure.

G.4 Design Charts for Estimation of Settlement for Drilled Shafts

Reese and O'Neill (1988) developed the following charts, (Figures 6.G.1 to 6.G.4), to estimate the settlement of drilled shaft under service loads. These charts express the settlement in terms of the ratio of the mobilized resistance to the actual resistance. If the computed settlement is too large, these charts may be used to modify the design accordingly. It is important to mention that the notation used in these charts, in several instances, differ from that mentioned in the Code. They are indicated as under and should be carefully considered.

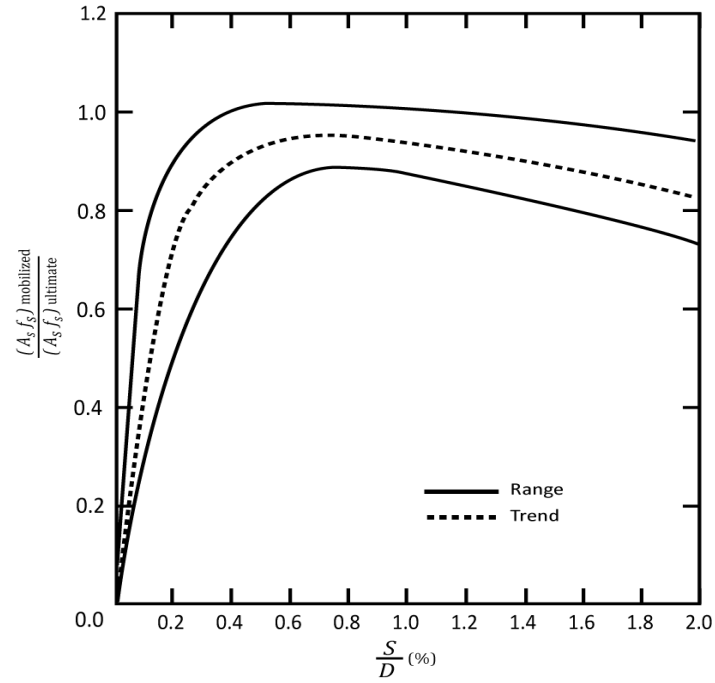


Figure 6.G.1 Normalised Curves for Load Transfer in Skin Friction vs Settlement for Drilled Shafts in Cohesive Soils (after Reese and O'Neill, 1988)

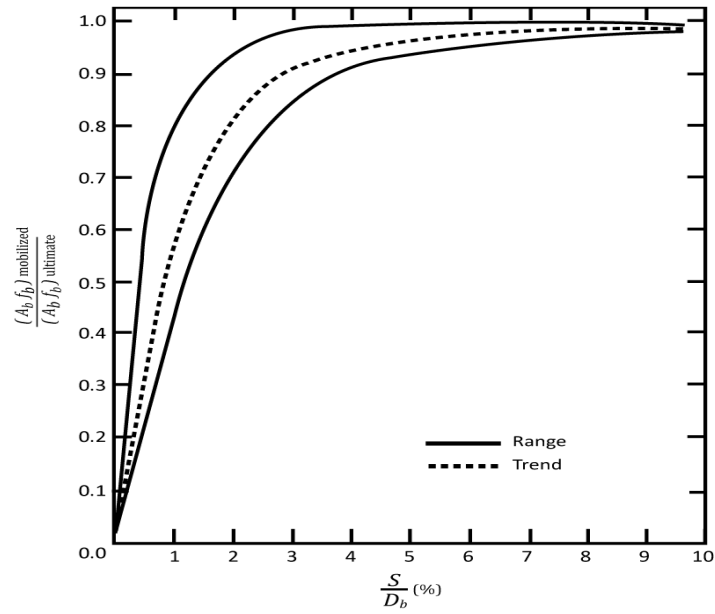


Figure 6.G.2 Normalised Curves for Load Transfer in End Bearing vs Settlement for Drilled Shafts in Cohesive Soils (after Reese and O'Neill, 1988)

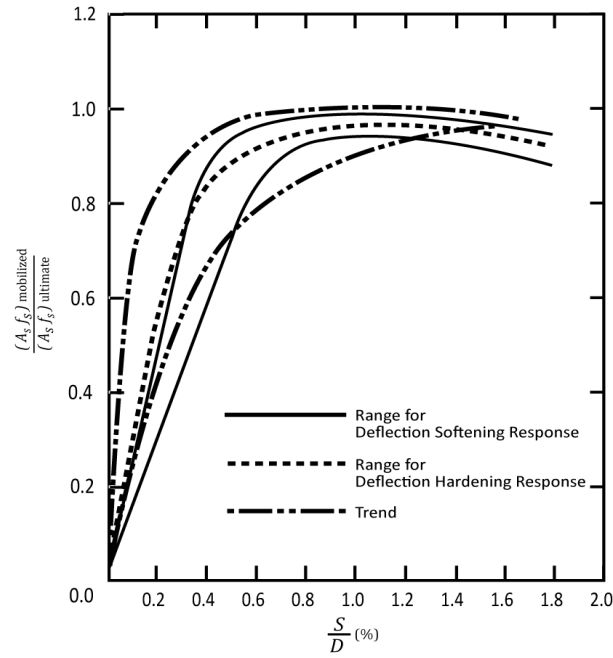


Figure 6.G.3 Normalised Curves for Load Transfer in Skin Friction vs Settlement for Drilled Shafts in Cohesionless Soils (after Reese and O'Neill, 1988)

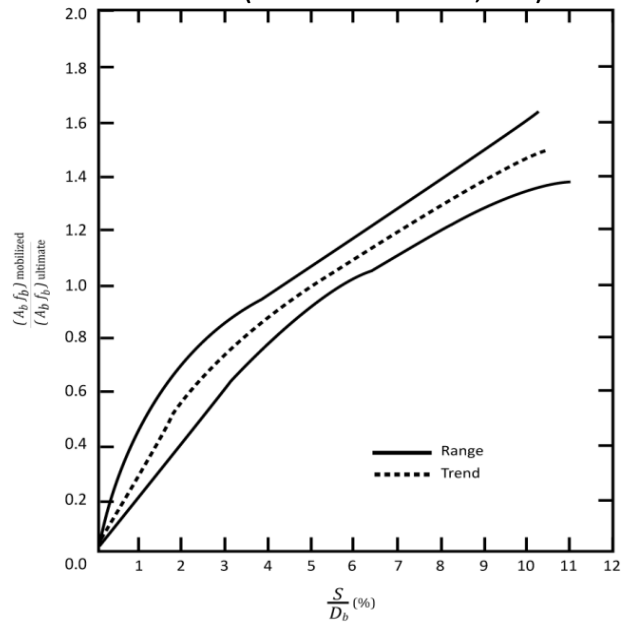


Figure 6.G.4 Normalised Curves for Load Transfer in End Bearing vs Settlement for Drilled Shafts in Cohesionless Soils (after Reese and O'Neill, 1988)

PART VI
Appendix H**References of Chapter 3 Part 6 (Soils and Foundations)**

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