Foundation Design using Standard Penetration Test (SPT) N-value

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CHAPTER 1

Standard Penetration Test: Corrections and Correlations

1.1 General

This chapter mainly focuses on the Standard Penetration Test, its correction and correlations with different soil properties.

1.2 Standard Penetration Test (SPT)

The Standard Penetration Test (SPT) is widely used to determine the in-situ properties of soil. The test is especially suited for cohesionless soils as the correlation between the SPT value and φ is now well established. In Bangladesh this test is widely used for all types of soil. The test was introduced by the Raymond Pile Company in 1902 and remains today as the most common in-situ test worldwide. The procedures for the SPT are detailed in ASTM D 1586 and AASHTO T-206.

The test consists of driving a split spoon sampler (Figure 1.1) into the soil through a borehole 55 to 100 mm (2 to 4 inch) in diameter at the desired depth. It is done by a hammer weighing 63.5 kg (140 lb) dropping onto a drill rod from a height of 750 mm (30 inch). The number of blows N required to produce a penetration of 300 mm (12 inches) is regarded as the penetration resistance. To avoid seating errors, the blows for the first 150 mm (6 inches) of penetration are not taken into account; those required to increase the penetration from 150 mm to 450 mm constitute the N-value. The operation of SPT is shown in Figure 1.2.

It is important to point out that several factors contribute to the variation of the standard penetration number 'N' at a given depth for similar soil profiles. Among these factors are the SPT hammer efficiency, borehole diameter, sampling method, rod length, water table and overburden pressure important. The most two common types of SPT hammers used in the field are the safety hammer and donut hammer. They are usually dropped using a rope with two wraps around a pulley. The configurations of the hammers are shown in Figure 1.3.

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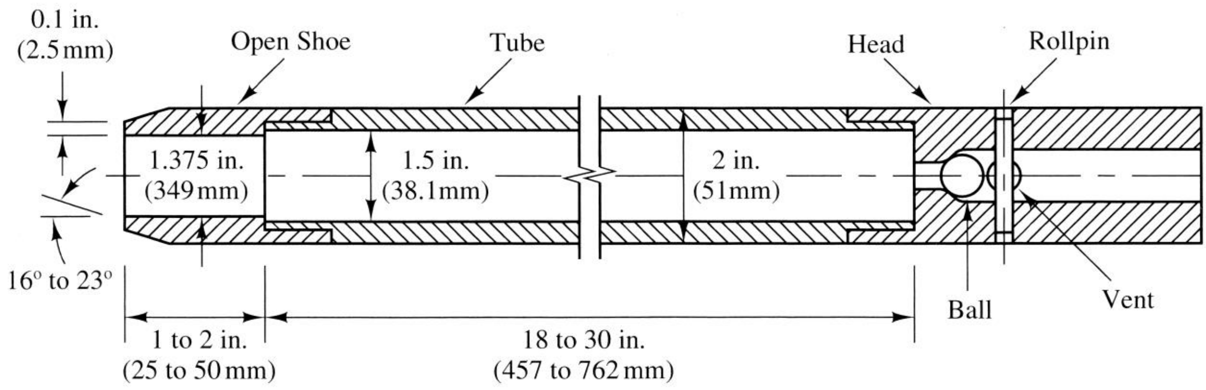

Figure 1.1 Schematic diagram of split spoon sampler (Coduto, 2001 pp.117, BNBC, 2015 fig. 6.D.2)

Usually SPT is conducted at every 1.5 m or 2 m depth or at the change of stratum. In hard formations, the testing is discontinued if N value is found to be over 100 and it is termed refusal.

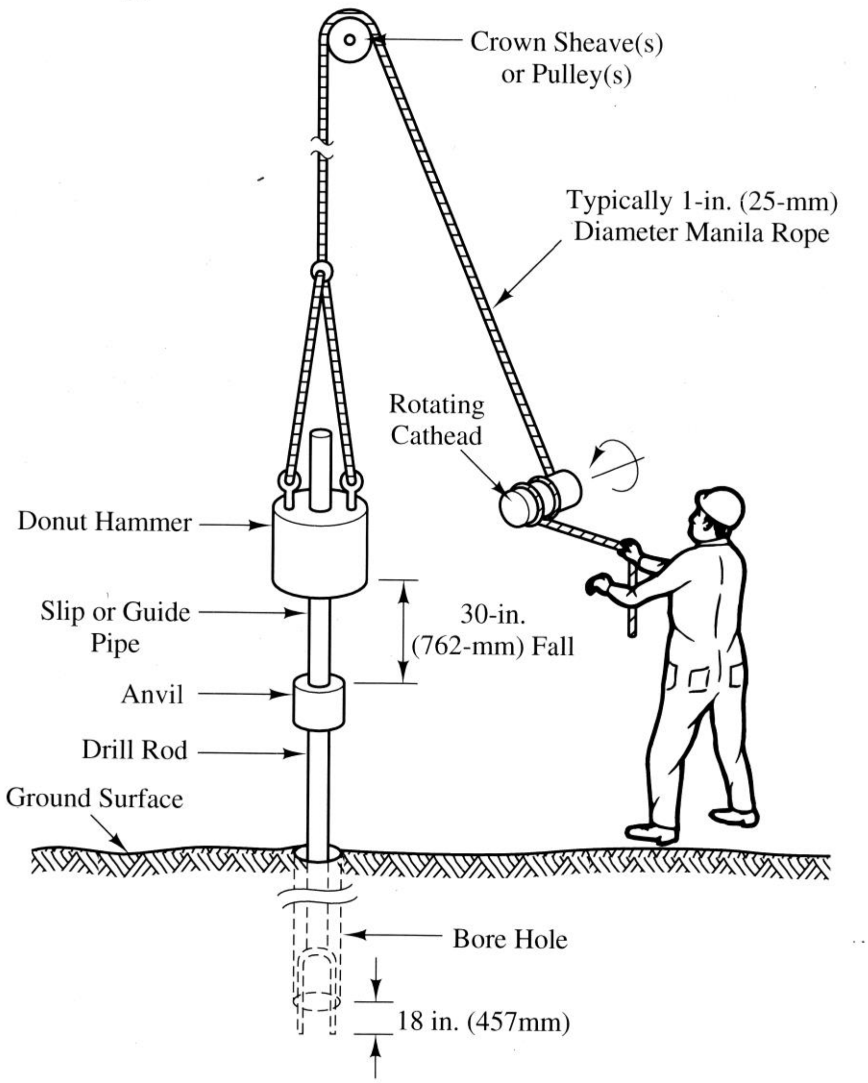

Figure 1.2 Demonstration of standard penetration test with donut hammer (Coduto, 2001 pp. 117 and BNBC 2015 figure 6.D.3)

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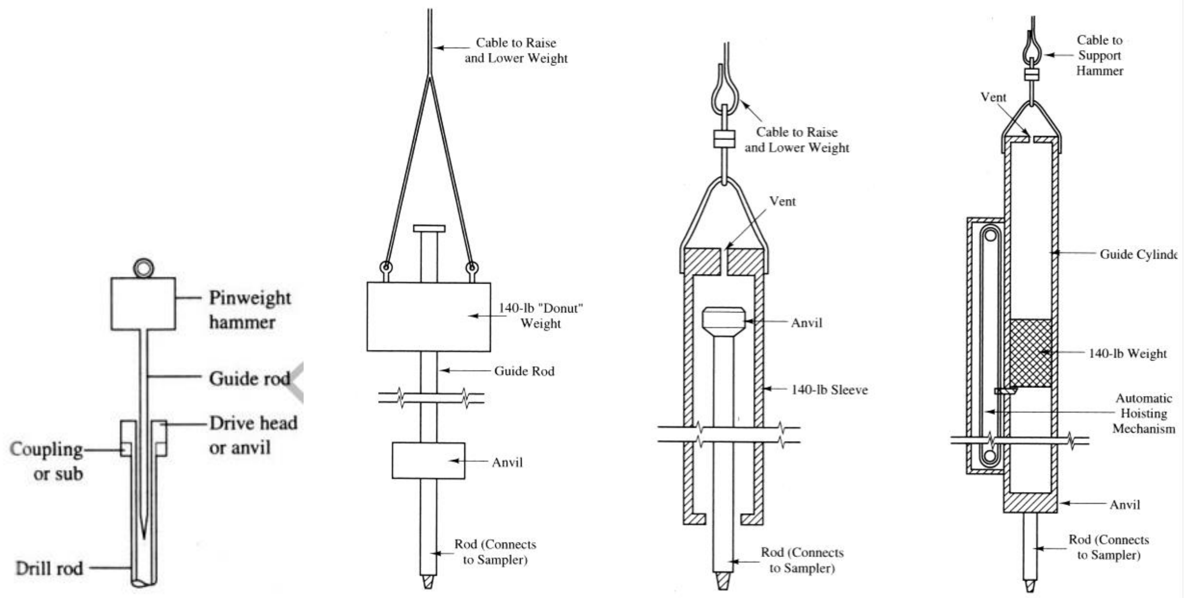

(a) pin (drop) (b) Donut (c) Safety (d) automatic

Figure 1.3 Different types of hammer used in SPT (Coduto, 2001, pp. 121, BNBC 2015 figure 6.D.4)

1.3 Termination of Standard Penetration Test (SPT)

The test can be terminated if the following three conditions appear in the field.

- A total of 50 blows have been applied during any one of the three 6-in. (150 mm) increments.
- A total of 100 blows have been applied.
- There is no observed advance of the sampler during the application of 10 successive blows of the hammer.

1.4 Correction of Standard Penetration Test (SPT)

The use of SPT correction factor is often confusing. *Corrections for field procedures* (Energy Correction) are always appropriate, but the *overburden pressure correction* may or may not be appropriate depending on the procedures by those who developed the analysis method under consideration.

For cohesive soil there is no need for overburden pressure correction (Peck et al.,1974 pp. 114). For Cohesionless soil at first overburden pressure correction is made, then if it is fine sand or silt under water table with N value >15, *dilatancy correction* is made. For coarse sand dilatancy correction is not required. Correction process can be represented by flowchart as shown in Figure 1.4.

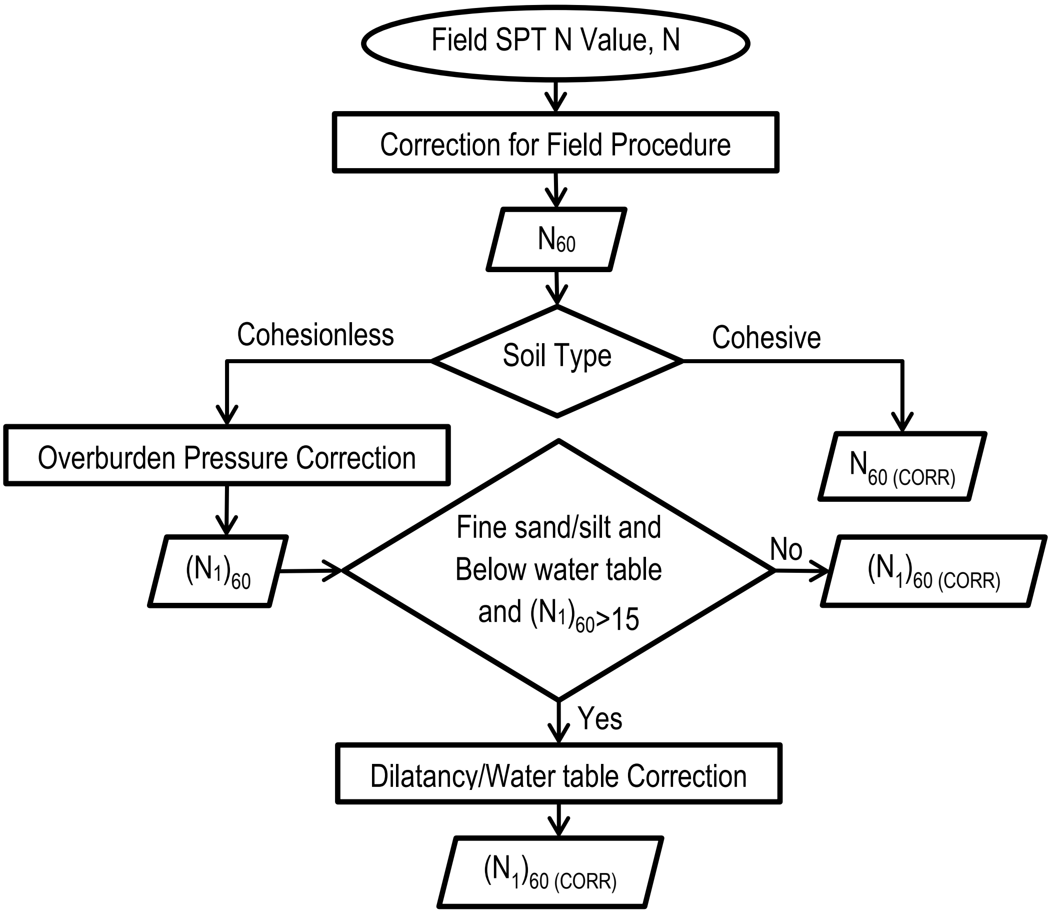

Figure 1.4 Flow chart of different types of correction of SPT N value

Different types of corrections are described briefly in the following articles.

1.4.1 Correction of SPT Value for Field Procedures

On the basis of field observations, it appears reasonable to standardize the field SPT number as a function of the input driving energy and its dissipation around the sampler around the surrounding soil. The variations in testing procedures may be at least partially compensated by converting the measured N to N_{60} as follows (Skempton, 1986)

$$N_{60} = \frac{E_H C_B C_S C_R N}{0.60} \quad (1.1)$$

Where,

N_{60} = Corrected SPT N-value for field procedures

E_H = Hammer efficiency (Table 1.1)

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C_B = Borehole diameter correction (Table 1.1)

C_S = Sampler correction (Table 1.1)

C_R = Rod length correction (Table 1.1)

N = Measured SPT N-value in field

This correction is to be done irrespective of the type of soil.

Table 1.1: Correction table for field procedure of SPT N-value

SPT Hammer Efficiencies (BNBC 2015 Table 6.D.4)		
Hammer Type	**Hammer Release Mechanism**	**Efficiency, E_H**
Automatic	Trip	0.70
Donut	Hand dropped	0.60
Donut	Cathead+2 turns	0.50
Safety	Cathead+2 turns	0.55-0.60
Drop/Pin	Hand dropped	0.45
Borehole, Sampler and Rod Correction Factors (BNBC 2015 Table 6.D.5)		
Factor	**Equipment Variables**	**Correction Factor**
Borehole Dia Factor, C_B	65 – 115 mm (2.5-4.5 in)	1.00
	150 mm (6 in)	1.05
	200 mm (8 in)	1.15
Sampler Correction, C_S	Standard sampler	1.00
	Sampler without liner (not recommended)	1.20
Rod Length Correction, C_R	3 – 4 m (10-13 ft)	0.75
	4 – 6 m (13-20 ft)	0.85
	6 – 10 m (20-30 ft)	0.95
	>10 m (>30 ft)	1.00

1.4.2 Correction of SPT Value for Overburden Pressure

In cohesionless soils, the overburden pressure affects the penetration resistance. For SPT made at shallow levels, the values are usually too low. At a greater depth, the same soil at the same density index would give higher penetration resistance. It was only as late as in 1957 that Gibbs & Holtz (1957) suggested that corrections should be made for field SPT values for depth.

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As the correction factor came to be considered only after 1957, all empirical data published before 1957 like those by Terzaghi is for uncorrected values of SPT. Since then a number of investigators have suggested overburden correction. Gibbs & Holtz took standard pressure of 280 kN/m^2 (corresponding to a depth of 14 m) and duly made overburden correction for other overburdens. Thornburn suggested a standard pressure of 138 kN/m^2 (corresponding to a depth of 7 m). Finally, Peck et. al. (1974) suggested a standard pressure of 100 kN/m^2 (Equivalent to 1 tsf or 1 kg/cm^2 overburden corresponding to a depth of 5 m). As such, all field SPT values are to be corrected by the correction factor given by them as

$$(N_1)_{60} = C_N \times N_{60} \leq 2N_{60} \tag{1.2}$$

Where,

C_N = Overburden pressure correction factor

The following relationships are widely used for C_N.

Peck et. al.'s relationship (1974): (BNBC 2015 Eq 6.D.2)

$$C_N = 0.77 \log\left(\frac{2000}{\sigma_0'}\right) \qquad \text{(1.3 SI)}$$

Where σ_0' is in kN/m^2 or kPa.

$$C_N = 0.77 \log\left(\frac{20}{\sigma_0'}\right) \qquad \text{(ENG)}$$

Where σ_0' is in tsf.

Liao and Whitman's relationship (1986):

$$C_N = \sqrt{\frac{100}{\sigma_0'}} \qquad \text{(1.4 SI)}$$

Where σ_0' is in kN/m^2 or kPa.

$$C_N = \sqrt{\frac{1}{\sigma_0'}} \qquad \text{(ENG)}$$

Where σ_0' is in tsf.

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1.4.3 Correction of SPT Value for Water Table

In addition to corrections of overburden, investigators suggested corrections of SPT-value for water table in the case of fine sand or silt below water table. Apparently, high N-values may be observed especially when observed value is higher than 15 due to dilatancy effect. In saturated, fine or silty, dense or very dense sand the N-values may be abnormally great because of the tendency of such materials to dilate during shear under undrained conditions. The pore pressure affects the resistance of the soil and hence the N value. In such cases, following correction is recommended (Terzaghi and Peck, 1948).

$$(N_1)_{60\,(CORR)} = 15 + \frac{1}{2}[(N_1)_{60} - 15] \tag{1.5}$$

For coarse sand this correction is not required. In applying this correction, overburden correction is applied first and then this diltancy correction is used.

1.5 Correlations between SPT N values and Different Parameters of Soil

The SPT has been used to correlate different soil parameters i.e., unit weight γ, relative density D_r, angle of internal friction φ and undrained compressive strength q_u. It has also been used to estimate the bearing capacity of foundations and for estimating the stress-strain modulus E_s.

Terzaghi and Peck give the following correlation (Table 1.2 and Table 1.3) between SPT value and other soil parameters.

Linear relationships of above correlation with average values can be very helpful in analytical problems. Some of the correlations are given below.

Correlation with unconfined compressive strength of cohesive soil shown in Equation (1.6 is a modified form of Terzaghi & Peck's (1967) relationship. This correlation was initially with field N-value (BNBC 2015 Eq. 6.D.7). Correlation with angle of internal friction of cohesionless soil shown in Equation (1.7 was originally in a graphical representation by Peck et. al 1974. (Shioi and Fukui, 1982; BNBC 2015 Eq. 6.D.5)

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Table 1.2: Penetration Resistance and Soil Properties on the Basis of SPT (Cohesionless Soil: Fairly reliable) (Peck et. al. 1974; Bowles, 1977; BNBC 2015 Table 6.D.6)

SPT N-value		0 to 4	4 to 10	10 to 30	30 to 50	>50
Compactness		**very loose**	**loose**	**medium**	**dense**	**very dense**
Relative Density, D_r (%)		0 to 15	15 to 35	35 to 65	65 to 85	85 to 100
Angle of Internal Friction, φ(°)		<28	28 to 30	30 to 36	36 to 41	>41
Unit Weight (moist)	**pcf**	<100	95 to 125	110 to 130	110 to 140	>130
	kN/m^3	<15.7	14.9 to 19.6	17.3 to 20.4	17.3 to 22.0	>20.4
Submerged unit weight	**pcf**	<60	55 to 65	60 to 70	65 to 85	>75
	kN/m^3	<9.4	8.6-10.2	9.4 to 11.0	10.5 to 13.4	>11.8

Table 1.3: Penetration Resistance and Soil Properties on the Basis of SPT (Cohesive Soil: rather unreliable) (Peck et. al. 1974; Bowles, 1977; BNBC 2015 Table 6.D.7)

SPT N-value		0 to 2	2 to 4	4 to 8	8 to 16	16 to 32	>32
Consistency		**very soft**	**soft**	**medium**	**stiff**	**very stiff**	**hard**
Unconfined Comp. Test	**lb/ft^2**	0 to 250	250 to 500	500 to 1000	1000 to 2000	2000 to 4000	>4000
	kPa	0 to 25	25 to 50	50 to 100	100 to 200	200 to 400	>400
Unit Weight (Saturated)	**pcf**	<100	100 to120	110 to 125	115 to130	120 to 140	>130
	kN/m^3	<15.7	15.7 to 18.8	17.3 to 19.6	18.1 to 20.4	18.8 to 22.0	>20.4

$$q_u = 12.5N_{60}\ (kN/m^2) \qquad (1.6\ \text{SI})$$

$$q_u = N_{60}/8\ (tsf) \qquad (\text{ENG})$$

$$\phi^\circ = 27 + 0.3(N_1)_{60} \qquad (1.7)$$

Correlations of N-value with q_u and ϕ by other researchers are given in Table 4 and Table 5.

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Table 1.4: Correlations between N-value and q_u for fine grained soil

Soil Type	q_u (kPa)	References
Highly plastic clay Medium to low plastic clay Plastic silts and clays with failure planes	24N 14.4N 6.7N	Sowers (1953) Sowers (1962)
Fine-grained soil	12.5N	Terzaghi & Peck (1967)
Clay Silty clay	25N 20N	Sanglerat (1972)
Fine-grained soil	$58N^{0.72}$	Hara et al. (1974) Kuhawy and Mayne (1990)
Plasticity Index<20 20<Plasticity Index<30 Plasticity Index>30	(6-7)N (4-5)N 4.2N	Stroud (1974)
Clay with high plasticity Clay with medium plasticity Clay with low plasticity	25N 15N 7.5N	Schmertmann (1975) Sowers (1979)
Fine-grained soil	1.39N+74.2	Ajayi & Balogun (1988)
Clay	12.5N $15N_{60}$	Decourt (1990)
High plasticity with LL>51% Medium plasticity with LL=36-50% Low plasticity with LL<35%	16N 15N 13N	Serajuddin and Chowdhury (1996)
High plastic clay Low plastic clay Clay Fine-grained soil	$9.5N_{field}$ $13.63N_{60}$ $6.7N_{field}$ $9.83N_{60}$ $8.66N_{field}$ $12.38N_{60}$ $8.64N_{field}$ $12.36N_{60}$	Sivrikaya & Togrol (2006)
Fine-grained soil	$4.1N_{60}$	Hettiarachchi & Brown (2009)

Correlation was made with unit weight of soil using the average value presented in Table 2 and 3.

For cohesive soil

$$\gamma_{sat} = 16.8 + 0.15N_{60}\ (kN/m^3) \tag{1.8 SI}$$

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$$\gamma_{sat} = 107 + 0.95N_{60}\ (pcf) \qquad \text{(ENG)}$$

For cohesionless soil

$$\gamma_{moist} = 16.0 + 0.1N_{60}\ (kN/m^3) \qquad \text{(1.9 SI)}$$

$$\gamma_{moist} = 102 + 0.65N_{60}\ (pcf) \qquad \text{(ENG)}$$

$$\gamma_{submerged} = 8.8 + 0.01N_{60}\ (kN/m^3) \qquad \text{(1.10 SI)}$$

$$\gamma_{submerged} = 56 + 0.45N_{60}\ (pcf) \qquad \text{(ENG)}$$

Table 1.5 Correlations between N-value and ϕ for cohessionless soils

Soil Type	$\phi°$	References
Sandy soil	0.3N+27	Peck et. al. (1953)
Angular and well-graded soil particles Round and well-graded or angular and uniform-graded soil particles Round and uniform-graded soil particles	$(12N)^{0.5}+25$ $(12N)^{0.5}+20$ $(12N)^{0.5}+15$	Dunham (1954)
Sandy soil	$(20N)^{0.5}+15$	Osaki et al. (1959)
Granular soil	$27.1+0.3N_{60}+0.00054(N_{60})^2$	Peck et al. (1974) Wolff (1989)
Sandy soil	$(15N)^{0.5}+15\leq 45$ (N>5)	Japan Road Association (1990)
Cohesionless soil	$\phi = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_0}{p_a}\right)}\right]^{0.34}$ Where Pa= atmospheric pressure in the same unit as σ'_0	Schmertmann (1975) Kuhawy and Mayne (1990)

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Example 1.1

Find out the corrected SPT N-value.

Given: Field N-value = 15, Depth = 6 m below ground level, Soil Type: Fine sand with trace mica, no water table was observed within this depth. Standard penetration test was performed with standard split spoon sampler and hand dropped donut hammer. Bore diameter was 100 mm.

Solution

Step 1: Correction for Field procedure is made for all types of soil

E_H = Hammer efficiency (Table 1.1) = 0.60

C_B = Borehole diameter correction (Table 1.1) = 1.00

C_S = Sampler correction (Table 1.1) = 1.00

C_R = Rod length* correction (Table 1.1) = 0.85

*total rod length = depth+legth above borehole (typically 1~2m; let 1.5m) = 6+1.5=7.5m

N = Measured SPT N-value in field = 15

$$N_{60} = \frac{E_H C_B C_S C_R N}{0.60} = \frac{0.60 \times 1.00 \times 1.00 \times 0.85 \times 15}{0.60} = 12.75 \approx 13$$

Step 2: Soil type is cohesionless, overburden pressure correction must be made. For overburden pressure correction effective overburden pressure and hence unit weight of soil must be known. Assume average unit weight of soil from N-value as follows

$$\gamma_{moist} = 16.0 + 0.1N_{60}\ (kN/m^3) = 16.0 + 0.1 \times 13 = 17.3\ kN/m^3$$

$$\sigma_0 = \gamma z = 17.3 \times 6 = 103.8\ kN/m^2$$

$$\sigma_0' = \sigma_0 - u = 103.8 - 0 = 103.8\ kN/m^2$$

Where u is pore water pressure. Here no water table is observed at 6 m.

$$(N_1)_{60} = C_N \times N_{60} = 0.77 \log\left(\frac{2000}{\sigma_0'}\right) \times N_{60}$$

$$(N_1)_{60} = 0.77 \log\left(\frac{2000}{103.8}\right) \times 13 = 0.99 \times 13 \approx 13 \leq 2N_{60}$$

Step 3: Since soil in not under water table and $(N_1)_{60} < 15$ hence no need for dilatancy correction.

$$N_{corr} = (N_1)_{60} = 13\ (\boldsymbol{Ans})$$

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Problems

1.1 Write a short note on Standard Penetration Test. What are the standards to be followed during SPT. On what condition SPT is terminated.

1.2 What are the corrections usually made to field N-value? Why these corrections are made? How to correct the field N-value? Describe it using a flow chart.

1.3 Write down some useful correlations of SPT N-value with shear strength of soil.

1.4 Find out the corrected SPT N-value for the given borelog. Standard penetration test was performed with standard split spoon barrel and hand dropped donut hammer. Bore diameter was 100 mm. Water table was observed 2m below ground level.

Depth (m) below GL	Field N-value	Soil type (field identification)
1.5	8	Organic Clay
3.0	7	Organic Clay
4.5	15	Fine Sand
6.0	22	Fine Sand
7.5	26	Coarse Sand

(Hint: Repeat Example 1.1 for all the soil layer)

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CHAPTER 2

Bearing Capacity of Shallow Foundation from N-value

2.1 General

Ultimate bearing capacity of shallow foundation for a c-ϕ soil is the major topic discussed in this chapter. Also the procedure to calculate the allowable capacity of soil from SPT N-value is shown here.

2.2 Bearing Capacity of Soil

The ability of the underlying soil to bear the load of the foundation without overstressing the soil in terms of either shear failure or excessive settlement is termed as bearing capacity of soil. This is often termed as bearing capacity of foundation.

2.3 Basic Definitions

Footing: A foundation constructed of masonry, concrete or other material under the base of a wall or one or more columns for the purpose of spreading the load over a larger area at shallower depth of ground surface.

Foundation: Lower part of the structure which is in direct contact with the soil and transmits loads to the ground.

Shallow Foundation: There is no particular definition of shallow foundation. Different researchers have defined it differently.
According to BNBC 2015 Draft, a foundation unit that provides support for a structure transferring loads at a small depth below the ground. Generally, the depth is less than two times the least dimension of the foundation.

BNBC 1993 states that, a foundation unit that provides support for a structure by transferring loads to soil or rock at shallow depths. Usually, the depth to width ratio is less than unity and the depth is within 3m (10 ft) from the surface.

Deep Foundation: A foundation unit that provides support for a structure transferring loads by end bearing and/or by shaft resistance at considerable depth below the ground. Generally, the depth is at least five times the least dimension of the foundation.

Bearing Capacity: This is a general term used to describe, the load carrying capacity of a foundation soil that enables to bear and transmit loads from a structure.

2.4 Different Definitions of Bearing Capacity

Different researchers have given different definitions of bearing capacity synonymously or distinctly hence it is often confusing.

Gross Pressure: The total pressure at the base of a footing due to the weight of the superstructure and the original overburden pressure.

Net Pressure: The gross pressure minus the surcharge pressure i.e. the overburden pressure of the soil at the foundation level.

Ultimate Bearing Capacity or Gross Ultimate Bearing Capacity (q_{ult})

Maximum pressure that a foundation soil can withstand *without the occurrence of shear failure* of the foundation.

The gross bearing capacity is inclusive of the pressure exerted by the weight of the soil standing on the foundation (called the surcharge pressure).

Net Ultimate Bearing Capacity ($q_{ult\text{-}net}$)

It is the net pressure that can be applied to the footing by external loads that will just initiate shear failure of the supporting soil. It is equal to ultimate bearing capacity minus the stress due to the weight of the footing and any soil or surcharge directly above it. Assuming the density of the footing (concrete) and soil (γ) are close enough to be considered equal, then

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$$q_{ult-net} = q_{ult} - \gamma D_f \qquad \textbf{(2.1)}$$

Where,

D_f = Depth of footing

Safe Bearing Capacity (q_{safe})

It is the bearing capacity after applying the factor of safety (FS). A factor of safety of between 2.0 to 3.0 (depending on the extent of soil exploration, quality control and monitoring of construction) shall be adopted to obtain safe bearing pressure when dead load and normal live load is used. There are two types of Safe Bearing Capacity

Safe Net Bearing Capacity ($q_{safe\text{-}net}$)

It is the maximum net intensity of loading that the foundation will safely carry without the risk of shear failure of soil irrespective of any amount of settlement that may occur. It is obtained by dividing the ultimate net bearing capacity by a suitable factor of safety.

$$q_{safe-net} = q_{ult-net}/FS \qquad \textbf{(2.2)}$$

Safe Gross Bearing Capacity

It is the maximum gross pressure which the soil can carry safely without shear failure.

$$q_{safe} = q_{safe-net} + \gamma D_f \qquad \textbf{(2.3)}$$

It is thus the maximum intensity of loading that can be transmitted to the soil without the risk of shear failure, irrespective of the settlement that may occur.

Safe Bearing Pressure ($q_{safe\text{-}pr}$)

The maximum average pressure of loading that the soil will safely carry without the risk of *permissible settlement*.

Allowable bearing capacity/pressure or Design Bearing Capacity (q_a or q_{all})

The maximum allowable net loading intensity on the soil at which the soil neither fails in shear nor undergoes excessive or intolerable settlement detrimental to the structure. This is the *minimum of safe bearing capacity and safe bearing pressure* The conventional design of a foundation is based on the concept of bearing capacity or allowable bearing pressure.

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2.5 Presumptive Bearing Capacity

Building codes of various organizations in different countries gives the allowable bearing capacity that can be used for proportioning footings. These are “Presumptive bearing capacity values based on experience with other structures already built. As presumptive values are based only on visual classification of surface soils, they are not reliable. These values don't consider important factors affecting the bearing capacity such as the shape, width, depth of footing, location of water table, strength and compressibility of the soil. Generally these values are conservative and can be used for preliminary design or even for final design of small unimportant structure. BNBC 2015 recommends Table 2.1 for uniform soil in the absence of test results.

Table 2.1: Presumptive Values of Bearing Capacity for Lightly Loaded Structures*
(BNBC 2015 Table 6.3.7)

Type of Material	Safe Bearing Capacity, kPa
1. Soft Rock or Shale	440
2. Gravel, sandy gravel, silty sandy gravel; very dense and offer high resistance to penetration during excavation (soil shall include the groups GW, GP, GM, GC)	400**
3. Sand (other than fine sand), gravelly sand, silty sand; dry (soil shall include the groups SW, SP, SM, SC)	200**
4. Fine sand; loose & dry (soil shall include the groups SW, SP)	100**
5. Silt, clayey silt, clayey sand; dry lumps which can be easily crushed by finger (soil shall include the groups ML, MI, SC, MH)	150
6. Clay, sandy clay; can be indented with strong thumb pressure (soil shall include the groups CL, CI, CH)	150
7. Soft clay; can be indented with modest thumb pressure (soil shall include the groups CL, CI, CH)	100
8. Very soft clay; can be penetrated several centimeters with thumb pressure (soil shall include the groups CL, CI, CH)	50
9. Organic clay & Peat (soil shall include the groups OI, OH, OL, Pt)	To be determined after investigation.
10. Fills	To be determined after investigation.

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Type of Material	Safe Bearing Capacity, kPa
* two stories or less (Occupancy category A, B, C and D) ** 50% of these values shall be used where water table is above the base, or below it within a distance equal to the least dimension of foundation.	

2.6 Allowable/Permissible/Tolerable Settlement

Allowable or limiting settlement of a building structure will depend on the nature of the structure, the foundation and the soil. Different types of structures have varying degrees of tolerance to settlements and distortions. These variations depend on the type of construction, use of the structure, rigidity of the structure and the presence of sensitive finishes.

As a general rule, a total settlement of 25 mm (1 inch) and a differential settlement of 20 mm (0.75 inch) between columns in most buildings shall be considered safe for buildings on isolated pad footings on sand for working load (un-factored). A total settlement of 40 mm (1.5 inch) and a differential settlement of 20 mm (0.75 inch) between columns shall be considered safe for buildings on isolated pad footings on clay soil for working load. Buildings on raft can usually tolerate greater total settlements.
Permissible settlements suggested by various authors are summarized in Table 2.2

Also BNBC 2015 suggested Table 2.3 to follow as a guideline.

Table 2.2: Permissible Total Settlement from various authors

Types of Foundation	Soil type	Maximum total allowable settlement	Reference
Isolated	Sand	25	Terzaghi and Peck (1967)
Rafts, continuous	Sand	50	Tomlinson (1980)
Isolated	Sand	40	Skempton and MacDonald (1956)
	Clay	65	
Rafts, continuous	Sand	45-65	
	Clay	65-100	

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Table 2.3: Permissible Total Settlement, Differential Settlement and Angular Distortion (Tilt) for Shallow Foundations in Soils (in mm) (Adapted from NBCI, 2005) (BNBC 2015 Table 6.3.8)

Type of Structure	Isolated Foundations						Raft Foundation					
	Sand and Hard Clay			Plastic Clay			Sand and Hard Clay			Plastic Clay		
	Maximum Settlement	Differential Settlement	Angular Distortion	Maximum Settlement	Differential Settlement	Angular Distortion	Maximum Settlement	Differential Settlement	Angular Distortion	Maximum Settlement	Differential Settlement	Angular Distortion
Steel Structure	50	0.0033 L	1/300	50	0.0033 L	1/300	75	0.0033 L	1/300	100	0.0033 L	1/300
RCC Structures	50	0.0015 L	1/666	75	0.0015 L	1/666	75	0.0021 L	1/500	100	0.002 L	1/500
Multistoried Building												
(a) RCC or steel framed building with panel walls	60	0.002 L	1/500	75	0.002 L	1/500	75	0.0025 L	1/400	125	0.0033 L	1/300
(b) Load bearing walls												
(i) L/H = 2 *	60	0.0002 L	1/5000	60	0.0002 L	1/5000	Not likely to be encountered					
(ii) L/H = 7 *	60	0.0004 L	1/2500	60	0.0004 L	1/2500	Not likely to be encountered					
Silos	50	0.0015 L	1/666	75	0.0015 L	1/666	100	0.0025 L	1/400	125	0.0025 L	1/400
Water Tank	50	0.0015 L	1/666	75	0.0015 L	1/666	100	0.0025 L	1/400	125	0.0025 L	1/400

Notes: The values given in the Table may be taken only as a guide and the permissible total settlement, differential settlement and tilt (angular distortion) in each case should be decided as per requirements of the designer.

L denotes the length of deflected part of wall/ raft or centre to centre distance between columns.

H denotes the height of wall from foundation footing.

* For intermediate ratios of L/H, the values can be interpolated.

2.7 Allowable Bearing Capacity from N-value for Cohesionless Soil

It is difficult to collect undisturbed sample in cohesionless soil hence extensive research have been made to find out the allowable bearing capacity of shallow foundation in cohesionless soil from SPT N-value. Among them, Terzaghi and Peck (1948, 1967), Meyerhof (1956, 1965, 1974), Bowles (1968, 1977, 1982, 1988, 1997), Teng (1962), D'Allolonia et al. (1968) Parry (1977), Peck and Bazarra (1969), Peck (1974), Mohan et al. (1971), **Burland and Burbridge (1985)** are extensively used all over the world. Some are in graphical form and others are in equations. Few methods are summarized in Table 2.4.

There is no particular method incorporated in BNBC 2015 to calculate the bearing capacity. According to BNBC 2015, any established method to calculate bearing capacity is applicable. Author modified the Bowles (1997) method slightly for 60% energy correction including overburden pressure correction and water table correction if applicable. Since water table correction is already reflected in N-value hence no need for water table correction factor. Allowable bearing capacity equations are given in Equation 2.3 to 2.5.

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Table 2.4 Allowable Bearing Capacity from N-value using 25.4mm (1 inch) settlement criteria.

Allowable Bearing Capacity	References
Originally in graphical form $q_a(kPa) = 12N\frac{1}{c_w c_d}\left(\frac{s}{25.4}\right)\ for\ B \leq 1.2m$ $q_a\ (kPa) = 8N\left(\frac{3.28B+1}{3.28B}\right)^2\frac{1}{c_w c_d}\ \left(\frac{s}{25.4}\right)\ for\ B > 1.2m$ $q_a(kPa) = 8N\frac{1}{c_w c_d}\left(\frac{s}{25.4}\right)\ for\ raft$ Where c_d = Depth factor $=1 + 0.25\frac{D_f}{B}$ c_w = water correction factor $=2 - \frac{D_w}{2B} \leq 2$ for surface footings $=2 - \frac{D_f}{2B} \leq 2$ for fully submerged footing $d_w \leq d_f$ s = tolerable settlement (mm) B = width of the footing (m) D_f = depth of footing (m) D_w= depth of water (m) N = Lowest (average) uncorrected N-value from depth of footing to D_f+B every 0.76m (2.5ft). water table correction suggested.	Terzaghi and Peck (1948)
$q_a(kPa) = 12NF_d\left(\frac{s}{25.4}\right)\ for\ B \leq 1.2m$ $q_a\ (kPa) = 8N\left(\frac{3.28B+1}{3.28B}\right)^2 F_d\left(\frac{s}{25.4}\right)\ for\ B > 1.2m$ $q_a(kPa) = 8NF_d\left(\frac{s}{25.4}\right)\ for\ raft$ Where F_d = Depth factor $=1 + 0.33\frac{D_f}{B} \leq 1.33$ s = tolerable settlement (mm) B = width of the footing (m) D_f = depth of footing (m) N = Average uncorrected N-value from depth of footing to D_f+B. Only water table correction suggested.	Meyerhof (1956)

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Allowable Bearing Capacity	References
$q_a(kPa) = 19.16(N_1)_{55}F_d\left(\frac{s}{25.4}\right)\ for\ B \leq 1.2m$ $q_a\ (kPa) = 11.98(N_1)_{55}\left(\frac{3.28B+1}{3.28B}\right)^2 F_d\left(\frac{s}{25.4}\right)\ for\ B > 1.2m$ $q_a\ (kPa) = 11.98(N_1)_{55}F_d\left(\frac{s}{25.4}\right)$ for rafts Where F_d = Depth factor $=1 + 0.33\frac{D_f}{B} \leq 1.33$ s = tolerable settlement (mm) B = width of the footing (m) D_f = depth of footing (m) $(N_1)_{55}$= Statistical average of corrected N value (55% energy with overburden pressure correction) for the footing influence zone of about 0.5B above footing base to at least 2B below.	Bowles (1997)
$q_{all-net}(kPa) = 34.5(N-3)\left(\frac{3.28B+1}{2\times 3.28B}\right)^2 R_{w2}F_d\left(\frac{s}{25.4}\right)$ Where s = tolerable settlement (mm) B = width of the footing (m) N= Corrected N-value for overburden pressure at a depth of 0.5B below the foundation level (Gibbs and Holtz, 1957) R_{w2} = Water table correction factor $=\frac{1}{2}\left(1 + \frac{D_{w2}}{B}\right)$ D_{w2} = Depth of WT below the base of the footing (m) F_d = Depth factor $=1 + \frac{D_f}{B} \leq 2.0$ B = width of the footing (m) D_f = depth of footing (m)	Teng (1962)
$q_a(kPa) = \frac{N_m}{B}\left(\frac{s}{300}\right)$ Where s = tolerable settlement (mm) B = width of the footing (m) N_m= “representative value of N” at a depth of 0.75B below the foundation level	Parry (1977)

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$$q_a(kPa) = 20N_{Design}F_d\left(\frac{s}{25.4}\right)\ for\ B \le 1.2m \tag{2.4}$$

$$q_a\ (kPa) = 12.5N_{Design}\left(\frac{3.28B+1}{3.28B}\right)^2 F_d\left(\frac{s}{25.4}\right)\ for\ B > 1.2m \tag{2.5}$$

$$q_a\ (kPa) = 12.5N_{Design}F_d\left(\frac{s}{25.4}\right) \text{for rafts} \tag{2.6}$$

Where

F_d = Depth factor $=1 + 0.33\frac{D_f}{B} \le 1.33$

s = tolerable settlement (mm)

B = width of the footing (m)

D_f = depth of footing (m)

N_{Design} = Special Weighted Average N value (60% energy with overburden pressure correction and water table correction if applicable) for the footing influence zone.

2.8 Allowable Bearing Capacity from N-value for Cohesive Soil

Extensive research have been made on granular soil to find the bearing capacity from field test but few on cohesive soil since penetration test data is unreliable. Skempton (1951) proposed equations for bearing capacity of footings founded on purely cohesive soils based on extensive investigations which can be modified to establish a relationship among net allowable bearing capacity, SPT N-value and bearing capacity factor N_c. According to him the bearing capacity factor N_c is a function of the depth of foundation and also of its shape. The equation for net ultimate bearing capacity, $q_{net\text{-}ult}$ is as follows:

$$q_{ult-net} = cN_c \tag{2.7}$$

Where

c = cohesion (kN/m^2) = $\frac{q_u}{2}$

q_u = unconfined compressive strength (kN/m^2)

N_c = bearing capacity factor

$=5\left(1 + 0.2\frac{D_f}{B}\right) \le 7.5$ for strip footing

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$$=6\left(1+0.2\frac{D_f}{B}\right) \leq 9 \text{ for square and circular footing}$$

$$=5\left(1+0.2\frac{B}{L}\right)\left(1+0.2\frac{D_f}{B}\right) \leq 9 \text{ for rectangular footing when } \frac{D_f}{B} \leq 2.5$$

$$=7.5\left(1+0.2\frac{B}{L}\right) \leq 9 \text{ for rectangular footing when } \frac{D_f}{B} > 2.5$$

Df = Depth of footing (m)

B = Width of footing or diameter in case of circular footing (m)

L = Length of footing (m)

Putting Equation (1.6) in Equation (2.7) and dividing by Factor of Safety=3, net safe bearing capacity can be obtained as shown in Equation (2.8).

$$q_{safe-net} = \frac{q_{ult-net}}{FS} = \frac{cN_c}{FS} = \frac{q_u \times N_c}{2 \times 3} = \frac{12.5N_{60} \times N_c}{6} \approx 2N_{60}N_c \tag{2.8}$$

Study shows the shear failure in cohesive soil is governing criteria for footing failure rather than settlement behavior hence net allowable bearing capacity can be expressed as follows

$$q_{all-net}\,(kPa) = 2N_{Design}N_c \tag{2.9}$$

Where

N_{Design} = Special Weighted Average N value (60% energy) for the footing influence zone.

2.9 Determination of design N-value (N_{Design})

Due to uncertainty in field procedure in Standard Penetration Test and also to consider all the N-value in influence zone of a foundation, author suggested a method to calculate the design N-value which should be used in allowable bearing capacity of shallow foundation rather than for a particular N-value. All the N-value from the influence zone is taken under consideration by giving highest weightage to the closest N-value from the footing base. N_{Design} is given by Equation (2.10).

$$N_{Design} = \frac{\sum_{i=1}^{n} \frac{N_i}{i^2}}{\sum_{i=1}^{n} \frac{1}{i^2}} \tag{2.10}$$

Where

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n = number of layers at which N-values are available from footing base to 2B or to a depth upto which soil types are approximately the same.

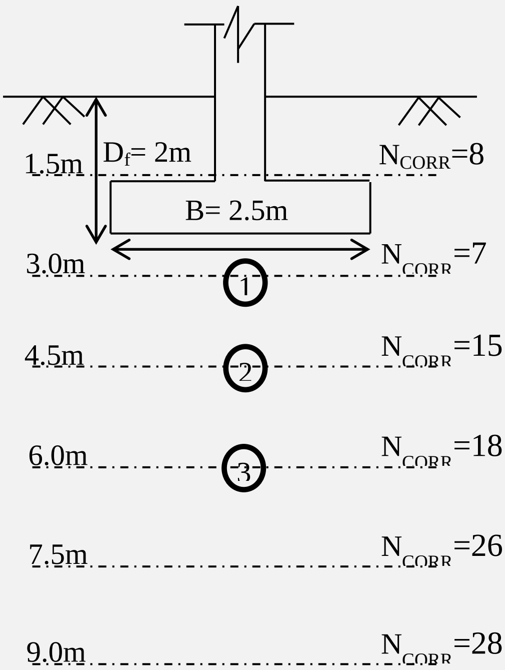
N_i = Corrected N-value at i-th layer from the footing base.

Example 2.1

The corrected N-value for a bore-log is given. Find the design N-value for a 2.5m square footing. The base of the footing is located at 2m below the existing ground level (EGL).

Depth (m) below GL	Corrected N Value	Soil type
1.5	8	Cohesive
3.0	7	Cohesive
4.5	15	Cohesive
6.0	18	Cohesive
7.5	26	Cohesionless
9.0	28	Cohesionless

Solution



To find the design N-value at first SPT layer must be identified upto 2B from the footing base or same soil type whichever is less.

For the given problem

$D_f + 2B = 2 + 2 \times 2.5 = 7m$

From the bore log it can be seen that upto 7m soil types show a similar profile.

There are three SPT layers of same soil type from 2 to 7m denoted as ①,② and③

Using Equation (2.10)

$$N_{Design} = \frac{\sum_{i=1}^{n} \frac{N_i}{i^2}}{\sum_{i=1}^{n} \frac{1}{i^2}} = \frac{\frac{7}{1^2} + \frac{15}{2^2} + \frac{18}{3^2}}{\frac{1}{1^2} + \frac{1}{2^2} + \frac{1}{3^2}} = 9.37 \approx 9$$

$N_{Design} = 9$ **(Ans)**

Alternatively, for ease in calculation the lowest N-value from depth of footing to D_f+B can be taken as suggested by Terzaghi and Peck (1948). Here in this case

$$N_{Design} = 7 \text{ (Ans)}$$

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Example 2.2

Find out the allowable bearing capacity in granular soil

Given

Depth of footing, D_f = 5ft; Width of footing, B= 8ft; Length of footing, L=10ft

N_{Design} = 20 ; Consider permissible settlement = 25.4mm

Solution

Since all the equations are given in SI units all the parameters must be converted into SI unit (ratio terms are not necessary to convert).

Here

D_f = 5/3.28 = 1.52m; B= 8/3.28 = 2.44m; L=10/3.28=3.05m and s=25.4mm

$F_d = 1 + 0.33\frac{D_f}{B} = 1 + 0.33\frac{1.52}{2.44} = 1.206 \leq 1.33$ and $B = 2.44m > 1.2m$

Using Equation (2.5)

$$q_a\ (kPa) = 12.5 N_{Design}\left(\frac{3.28B+1}{3.28B}\right)^2 F_d\left(\frac{s}{25.4}\right)$$

$$q_a\ (kPa) = 12.5 \times 20 \times \left(\frac{3.28 \times 2.44 + 1}{3.28 \times 2.44}\right)^2 \times 1.206 \times \left(\frac{25.4}{25.4}\right)$$

$q_a = 381.6\ kPa \approx 380\ kPa$ **(Ans)**

Example 2.3

Find out the net-allowable bearing capacity in cohesive soil

Given

Depth of footing, D_f = 5ft; Width of footing, B= 6ft; Length of footing, L=8ft

N_{Design} = 9

Solution

Here

D_f = 5/3.28 = 1.52m; B= 6/3.28 = 1.83m and L=8/3.28=2.44m

$\frac{D_f}{B} = \frac{1.52}{1.83} = 0.831 \leq 2.5$

Using Equation (2.9) $q_{all-net}\ (kPa) = 2N_{Design}N_c$

Where $N_c = 5\left(1 + 0.2\frac{B}{L}\right)\left(1 + 0.2\frac{D_f}{B}\right)$ for rectangular footing $\frac{D_f}{B} \leq 2.5$

$$N_c = 5\left(1 + 0.2\frac{1.83}{2.44}\right)\left(1 + 0.2\frac{1.52}{1.83}\right) = 6.705$$

$q_{all-net}\ (kPa) = 2N_{Design}N_c = 2 \times 9 \times 6.705$

$q_{all-net} = 120.7\ kPa\ \approx 120\ kPa$ **(Ans)**

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Problems

2.1 What are Shallow and Deep foundations. Name some of them

2.2 Define

(a) Allowable Bearing Capacity

(b) Net-safe Bearing Capacity

(c) Ultimate Bearing Capacity

2.3 Write short note on

(a) Presumptive Bearing Capacity

(b) Permissible Settlement

2.4 A bore-log is given below for SPT. Standard penetration test was performed with standard split spoon barrel and hand dropped donut hammer. Bore diameter was 100 mm. Water table was observed 2m below ground level.

Given, Depth of footing, D_f = 2m; Width of footing, B= 3.05m; Length of footing, L=3.66m. Find out the allowable bearing capacity of the given soil.

Depth (m) below GL	Field N-value	Soil type (field identification)
1.5	8	Silty Clay
3.0	7	Silty Clay
4.5	15	Fine Sand
6.0	22	Fine Sand
7.5	26	Coarse Sand

(Hint: first correct the field N-value as Problem 1.4 then find the design N-value as discussed in Example 2.1 then follow Example 2.2 or Example 2.3 based on soil type)

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CHAPTER 3

Geotechnical Design of Piles from N-values

3.1 General

In this chapter uses of N-value to find out the capacity of Precast and Cast in-situ concrete piles are briefly discussed. Also illustrative examples have been given for practical design purpose.

3.2 Basic Definitions

Pile: A slender deep foundation unit made of materials such as steel, concrete, wood, or combination thereof that transmits the load to the ground by skin friction, end bearing and lateral soil resistance.

Driven or Precast Piles (Displacement type): A pile foundation pre-manufactured and placed in ground by driving, jacking, jetting or screwing. This is displacement type pile since the soil is displaced by the placement of the pile.

Bored or Cast in-situ or Cast in place Piles (Replacement type): A pile formed into a preformed hole of ground, usually of reinforced concrete having a diameter smaller than 600 mm. Pile having more than 600mm diameter is termed as Pier. This is replacement type pile since soil is replaced by the pile materials.

Batter or Raker Pile or Inclined Pile: The pile which is installed at an angle to the vertical in order to carry lateral loads along with the vertical loads.

Screw or Auger Pile: A pre-manufactured pile consisting of steel helical blades and a shaft placed into ground by screwing.

Pile Cap: A pile cap is a special footing needed to transmit the column load to a group or cluster of piles.

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Pile Head or Top: The upper small length of a pile.

Pile Toe or Tip: The bottom end of a pile.

Pile Shoe: A separate reinforcement or steel form attached to the bottom end (pile toe) of a pile to facilitate driving, to protect the pile toe, and/or to improve the toe resistance of the pile.

3.3 Load Bearing Mechanism of Piles

The ultimate load capacity of a pile consists of two parts. One part is due to friction called skin friction or shaft friction or side shear, and the other is due to end bearing at the base or tip or the end of the pile. According to BNBC (2015), if the skin friction is greater than about 80% of the end bearing load capacity, the pile is called a ***friction pile*** and, if the opposite occurs then it is called an ***end bearing pile***. If the end bearing is fully neglected, the pile is called a ***floating pile***.

3.4 Design Considerations

In determining the design capacity of piles following items shall be considered

a) Ultimate geotechnical capacity (axial and lateral).
b) Structural capacity of pile section (axial and lateral).
c) The allowable axial load on a pile shall be the least value of the above two capacities.

In determining the geotechnical design axial capacity, BNBC (2015) suggests to consider the followings which are generally overlooked by designers:

a) The influence of fluctuations in the elevation of *ground water table* on capacity.
b) The *effects of driving piles* on adjacent structure and slopes.
c) The effects of *negative skin friction* or down loads from consolidating soil and the effects of *uplift loads* from expansive or swelling soils.
d) The *influence of construction techniques* such as auger boring or jetting on pile capacity.

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e) The difference between the supporting capacity single pile and that of a *group of piles*.
f) The capacity of an underlying stratum to support load of the pile group.
g) The *possibility of scour* and its effect on axial lateral capacity.

3.5 Geotechnical Design of Pile for Axial Loading

As stated earlier, the ultimate load capacity (Q_{ult}) of a pile consists of two parts. One part is due to friction called skin friction or shaft friction or side shear (Q_s) and the other is due to end bearing at the base or tip of the pile (Q_b). The ultimate axial capacity of a pile shall be determined in accordance with the following for compression loading as suggested by BNBC 2015 in art 3.10.

$$Q_{ult} = Q_s + Q_b - W \quad \textbf{(3.1)}$$

For uplift loading

$$Q_{ult} \leq 0.7Q_s + W \quad \textbf{(3.2)}$$

Where

W = weight of the pile

The total skin friction for n-number of SPT layers can be calculated as

$$Q_s = \sum_{i=1}^{n}(A_s)_i(f_s)_i = (A_s)_{total}(f_s)_{avg} \quad \textbf{(3.3)}$$

End bearing at the vicinity of pile tip can be calculated as

$$Q_b = A_b f_b \quad \textbf{(3.4)}$$

Where

A_s = skin friction area (perimeter area) of the pile = Perimeter × Length

f_s = skin frictional resistance on unit surface area of pile that depends on soil properties and loading conditions (drained or undrained)

A_b = end bearing area of the pile = Cross-sectional area of pile tip (bottom)

f_b = end bearing resistance on unit tip area of pile, that depends on soil properties to a depth of 2B (B is the diameter for a circular pile section or length of sides for a square pile section) from the pile tip and loading conditions (drained or undrained)

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The allowable or working axial load capacity shall be determined as follows

$$Q_{allow} = \frac{Q_{ult}}{FS} \tag{3.5}$$

Where, FS is a gross factor of safety usually greater than 2.5. Often, for compression loading, the weight term is neglected if the weight, W, is considered in estimating imposed loading.

3.6 Methods to determine Axial Pile Capacity

BNBC (2015) provides the following provisions to determine the ultimate bearing capacity (skin friction and/or end bearing) of a single vertical pile

a) By the use of static bearing capacity equations
b) By the use of SPT and CPT
c) By load tests
d) By dynamic methods

In this chapter the use of SPT is briefly discussed. Author suggests going through BNBC (2015) for further reading.

3.7 Axial Pile Capacity from N-value as per BNBC (2015)

BNBC (2015) discussed the unit skin friction and end bearing capacity of driven and bored pile in separate sections. According to it, since N-value is an indirect measure of relative density hence angle of internal friction for cohesionless soil and cohesion for cohesive soil, hence, N-value can be used to determine the skin friction and end bearing for pile. BNBC (2015) method is summarized in Table 3.1.

Empirical formulas accumulated in Table 3.1 can be used to calculate the unit skin friction and end bearing for a particular pile and soil type where $\overline{N}_{60}$ is the average N-value (corrected for field procedure) over the pile shaft length and N_{60} is the N-value (corrected for field procedure) in the vicinity of pile tip. Then equation (3.3) and (3.4) is used to determine the total skin friction and end bearing. After that, equation (3.1) is used to calculate the ultimate axial capacity. Dividing it with a suitable safety factor gives the allowable axial capacity. A higher ***factor of safety of 3.5*** is suggested to estimate allowable capacity. This procedure is illustrated through Examples.

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Table 3.1 Axial Pile Capacity from N-value as per BNBC (2015)

Pile Type	Soil Type	Unit Ultimate Capacity	
		Skin Friction, f_s (kPa)	**End Bearing, f_b (kPa)**
Driven	**Sand**	$2\bar{N}_{60} \leq 60$	$40N_{60}\left(\frac{L}{D}\right) \leq 400N_{60} \leq 11000$
	Non-plastic silt	$1.7\bar{N}_{60} \leq 60$	$30N_{60}\left(\frac{L}{D}\right) \leq 300N_{60} \leq 11000$
	Cohesive	$1.8\bar{N}_{60} \leq 70$	$45N_{60} \leq 4000$
Bored	**Sand**	$1\bar{N}_{60} \leq 60$	$15N_{60}\left(\frac{L}{D}\right) \leq 150N_{60} \leq 4000$
	Non-plastic silt	$0.9\bar{N}_{60} \leq 60$	$10N_{60}\left(\frac{L}{D}\right) \leq 100N_{60} \leq 4000$
	Cohesive	$1.2\bar{N}_{60} \leq 70$	$25N_{60} \leq 4000$

Example 3.1

Find out the allowable pile capacity of a cast in-situ pile of 500 mm diameter and pile tip at 10.5m below EGL. The soil bore log is given below. Pile cutoff depth is 1m below EGL.

Depth (m) below GL	Field N-value	Soil type (field identification)	E_H, C_B, C_s, C_R from Table 1.1	N_{60} from Eq. (1.1)
1.5	8	Silty Clay	0.60, 1 ,1 ,0.75	6
3.0	7	Silty Clay	0.60, 1 ,1 ,0.85	6
4.5	15	Non-plastic silt	0.60, 1 ,1 ,0.85	13
6.0	22	Non-plastic silt	0.60, 1 ,1 ,0.95	21
7.5	26	Fine Sand	0.60, 1 ,1 ,0.95	25
9.0	28	Fine Sand	0.60, 1 ,1 ,1.00	28
10.5	40	Coarse Sand	0.60, 1 ,1 ,1.00	40
12.0	45	Coarse Sand	0.60, 1 ,1 ,1.00	45

Solution

The soil data is considered at every SPT layer

A pile from 1.0m to 10.5 m below existing ground level with 500mm diameter

Length of Pile, L = 10.5-1.0 = 9.5 m

Diameter of Pile, D = 500mm = 0.5m

Skin friction area, $(A_s)_{total}$ = perimeter×length= $\pi DL = 3.1416 \times 0.5 \times 9.5 = 14.92\ m^2$

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End bearing area, $A_b = \frac{\pi}{4}D^2 = \frac{3.1416}{4} \times 0.5^2 = 0.196\ m^2$

Depth (m)	**N_{60}**	**Soil type**	**Unit Skin Friction, f_s (kPa) from Table 3.1**	**End Bearing, f_b (kPa) from Table 3.1**
1.5	6	Silty Clay	Using $1.2\bar{N}_{60} \le 70$ $= 1.2 \times 6 = 7.2$	Using $25N_{60} \le 4000$ $= 25 \times 6 = 150$
3.0	6	Silty Clay	Using $1.2\bar{N}_{60} \le 70$ $= 1.2 \times 6 = 7.2$	Using $25N_{60} \le 4000$ $= 25 \times 6 = 150$
4.5	13	Non-plastic silt	Using $0.9\bar{N}_{60} \le 60$ $= 0.9 \times 13 = 11.7$	Using $100N_{60} \le 4000$ $= 100 \times 13 = 1300$
6.0	21	Non-plastic silt	Using $0.9\bar{N}_{60} \le 60$ $= 0.9 \times 21 = 18.9$	Using $100N_{60} \le 4000$ $= 100 \times 21 = 2100$
7.5	25	Fine Sand	Using $1\bar{N}_{60} \le 60$ $= 1.0 \times 25 = 25$	Using $150N_{60} \le 4000$ $= 150 \times 25 = 3750$
9.0	28	Fine Sand	Using $1\bar{N}_{60} \le 60$ $= 1.0 \times 28 = 28$	Using $150N_{60} \le 4000$ $= 150 \times 28 = 4200$ $\therefore 4000$
10.5	40	Coarse Sand	Using $1\bar{N}_{60} \le 60$ $= 1.0 \times 40 = 40$	Using $150N_{60} \le 4000$ $= 150 \times 40 = 6000$ $\therefore 4000$
12.0	45	Coarse Sand	Using $1\bar{N}_{60} \le 60$ $= 1.0 \times 45 = 45$	Using $150N_{60} \le 4000$ $= 150 \times 45 = 6750$ $\therefore 4000$

Average unit skin friction, $(f_s)_{avg} = \frac{7.2+7.2+11.7+18.9+25+28+40}{7} = 19.71\ kPa$

Unit end bearing at pile tip vicinity, $f_b = 4000\ kPa$

Neglecting the weight of pile, Eq. (3.1) can be rewritten as

Ultimate Pile Capacity, $Q_{ult} = Q_s + Q_b = (A_s)_{total}(f_s)_{avg} + A_b f_b$

$= 14.92 \times 19.71 + 0.196 \times 4000 = 294.1 + 784 = 1078.1\ kN$

Allowable Pile Capacity, $Q_{allow} = \frac{Q_{ult}}{FS} = \frac{1078.1}{3.5} = 308\ kN$ **(Ans)**

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3.8 Axial Pile Capacity from Shear Strength Parameters

Axial pile capacity can be calculated from shear strength parameters [cohesion (c) and the angle of internal friction (ϕ)] of soil which are inherent properties of the soil. There are many methods suggested by different authors (Alpha method, Beta Method, Lambda Method, etc.). In this article $\alpha - method$ for cohesive soil and $\beta - method$ for cohesionless soil is discussed as BNBC (2015) incorporates both of them.

3.8.1 Unit Skin Friction by $\alpha -$ method for cohesive soil

The unit ultimate skin friction (f_s) in clay can be expressed by following equation

$$f_s = \alpha c_u \qquad (3.6)$$

Where

α = adhesion factor

c_u = undrained shear strength = cohesion, c = $\frac{q_u}{2}$

There are several methods proposed empirically from test data by different authors to estimate the adhesion factor. American Petroleum Institute, API (1984) is based on total stress analysis. It neglects the effective stress effects in soil but widely used. There is also a new method proposed by Kolk and Velde (1996) which considers both cohesion and effective stress. BNBC (2015) integrates the API (1984) method to estimate adhesion factor for driven pile given as follows

$$\begin{aligned} &\alpha = 1.0 \text{ for } c_u \leq 25\ kPa \\ &\alpha = 0.5 \text{ for } c_u \geq 70\ kPa \\ &\alpha = 1 - \left(\frac{c_u - 25}{90}\right) \text{ for } 25\ kPa < c_u < 70 kPa \end{aligned} \qquad (3.7)$$

For bored pile, adhesion factor is chosen to be 0.7 times the value for driven piles by Fleming et al. (1985) and can be expressed as follows

$$\begin{aligned} &\alpha = 0.7 \text{ for } c_u \leq 25\ kPa \\ &\alpha = 0.35 \text{ for } c_u \geq 70\ kPa \\ &\alpha = 0.7 - \left(\frac{c_u - 25}{90}\right) \text{ for } 25\ kPa < c_u < 70 kPa \end{aligned} \qquad (3.8)$$

BNBC (2015) suggests in Art 3.10.4.6 that the skin friction, f_s may be taken as $2/3^{rd}$ (0.67 times) the value of driven pile.

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3.8.2 Unit Skin Friction by $\beta -$ method for cohesionless soil

Numerous techniques have been proposed to compute the skin friction in piles in sandy soil and still it is not completely understand.

The unit ultimate skin friction (f_s) in sandy can be expressed by following equation

$$f_s = \beta \sigma'_z \qquad (3.9)$$

Where

β = Friction factor due to overburden pressure.

c_u = undrained shear strength = cohesion, c = $\frac{q_u}{2}$

Different researchers (i.e., McClelland (1974), Meyerhof (1976), Kraft and Lyons (1974)) suggested different values of β.

BNBC (2015) integrates the Meyerhof (1976) method to estimate friction factor, β for driven and bored pile are given below

Driven Pile (3.10)

$\beta = 0.44$ for $\phi' = 28^o$ (a)

$\beta = 0.75$ for $\phi' = 35^o$ (b)

$\beta = 1.20$ for $\phi' = 37^o$ (c)

Bored Pile

$\beta = 0.10$ for $\phi' = 33^o$ (d)

$\beta = 0.20$ for $\phi' = 35^o$ (e)

$\beta = 0.35$ for $\phi' = 37^o$ (f)

3.8.3 Equations of Unit End Bearing, f_b

BNBC (2015) suggests to use Skempton (1959) equation to find out the unit end bearing in cohesive soil

$$f_b = (c_u)_b N_c \leq 4000kPa \qquad (3.11)$$

Where

$(c_u)_b$ = undrained shear strength at the base of the pile (kPa)

N_c = Bearing capacity factor for deep foundation, usually 9

$= 6\left(1 + 0.2\frac{L}{B}\right) \leq 9$.

L = Length of pile, m

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B = width for square pile or diameter for circular pile at the tip of pile, m

BNBC (2015) adopted API (1984) method to find out the unit end bearing in granular soil as follows

$$f_b = (\sigma_z')_b N_q \quad (3.12)$$

Where

σ_z' = effective stress at pile tip $\leq 240kPa$

N_q = bearing capacity factor

= 8 to 12 for loose sand

=12 to 40 for medium sand

=40 for dense sand

Table 1.2 can be used to find the compactness of sand. BNBC (2015) suggests in Art 3.10.4.6 that the end bearing, f_b may be taken as 1/3rd (0.33 times) the value of driven pile.

3.9 Selection of Factor of Safety

Factor of Safety is a very important issue in design. Project cost directly depends on the proper use of factor of safety. The recommended values of overall factor of safety on ultimate axial load capacity of driven bored pile and drilled shaft on specified construction control is presented in Tables 6.3.10a and 6.3.10b in BNBC (2015).

Table 3.2 Factor of Safety for Deep Foundation (Coduto, 1994; BNBC, 2015)

Structure	Design Life (yrs.)	Probability of Failure	Design Factor of Safety			
			Good Control	Normal Control	Poor Control	Very Poor Control
Monument	>100	10^{-5}	2.30	3.00	3.50	4.00
Permanent	25-100	10^{-4}	2.00	2.50	2.80	3.40
Temporary	<25	10^{-3}	1.40	2.00	2.30	2.80
Proper Subsoil Investigation			Yes	Yes	Yes	Yes
Proper Review of Subsoil Report			Yes	Yes	Yes	Yes
Supervision by Competent Geotechnical/Foundation Engineer			Yes	Yes	Yes	No
Load Test Data			Yes	Yes	Yes	No
Qualification of Contractor			Yes	Yes	No	No
Proper Construction Equipment's			Yes	No	No	No
Maintaining Proper Construction Log			Yes	No	No	No

As a general guideline, it also states that a pile shall be designed for a minimum overall factor of safety of 2.0 against bearing capacity failure (end bearing, side resistance or combined) when the design is based on the results of a load test conducted at the site. Otherwise, it shall

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be designed for a minimum overall factor of safety of 3.0. If a normal level of field quality control cannot be assured, higher minimum factors of safety shall be used. Factor of safety chart is given in Table 3.2.

3.10 Group Pile Action

In practice piles are placed in a group under a pile cap. Individual action of a single pile is different from group action when piles are closely spaced since the influence zones are overlapped. Ideally, the piles in a group should be spaced so that the load-bearing capacity of the group is not less than the sum of the bearing capacity of the individual piles.

The group action of bored pile is almost similar to driven pile. Group pile capacity should be determined as the product of group efficiency, number of piles in the group and the capacity of a single pile. Pile group capacity can be obtained by

Pile group capacity = Efficiency of the pile group × Individual Pile Capacity × no of piles

For an example if a group of pile consists 4 piles each having the capacity of 300 kPa then the pile group should the capacity of 4×300=1200 kPa. Due to the overlapping of influence zone if the group efficiency is 0.70 then the actual group capacity is 0.70×4×300=840 kPa. It is clearly seen that, in design high efficiency is desirable. The efficiency of pile group depends on the spacing of piles but the spacing of the piles cannot be infinitely increased since they are connected with pile cap and increasing the spacing also increases the cost of pile cap.

There are some general guidelines included in BNBC (2015) as follows:

a) A group efficiency value of 1.0 should be used ***except for friction piles driven in the cohesive soils***. *(failure of the pile group as a whole should be considered in cohesive soil)*
b) The minimum center to center pile ***spacing of 2.5B*** is recommended.
c) The ***nominal dimensions and length of all piles in a group should be similar***.
d) ***All piles shall be braced to provide lateral stability in all directions***. Three or more piles connected by a rigid cap shall be considered as being braced (stable), provided that the piles are located in a radial direction from the centroid of the group, not less

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than 60° apart circumferentially. A two pile group in a rigid cap shall be considered to be braced along the axis connecting the two piles. However, Individual piles are considered stable if the pile tops are laterally braced in two directions by construction, such as a structural floor slat, grade beams, struts or wall.

e) The use of a ***single pile as foundation is not recommended*** unless the diameter is 600mm (2ft) or more.

From BNBC (2015) guideline it can be concluded that for a pile group in sand with spacing 2.5B or greater, group efficiency can be assumed as 1.0 and hence capacity of a pile group is nothing but equal to the single pile capacity multiplied by the number of piles in that group. In cohesive soil, due to group failure action, one can calculate the group capacity similar to the single pile capacity as described in Section 3.8.1 and 3.8.3 or 3.7 by only considering the pile group as a whole. In this case, adhesion factor is taken as unity since the failure occurs from soil to soil interactions not soil to pile interactions.

Example 3.2

Find out the allowable capacity of a single pile in a pile group consisting four 10m long precast driven pile of 400mm×400mm square section with 1m center to center spacing in a square arrangement. Pile is to be driven into a uniform saturated medium soft clay layer of 15m with average undrained shear strength of 60 kPa measured from unconfined compressive strength test in laboratory.

Solution

<u>Individual Pile Action</u>

Given

Length of Pile, L = 10 m

Width of Pile, B = 400mm = 0.4m

Skin friction area, $(A_s)_{total}$ = perimeter×length= $4BL = 4 \times 0.4 \times 10 = 16.00\ m^2$

End bearing area, $A_b = B^2 = 0.4^2 = 0.16\ m^2$

Using Eq. (3.6) and (3.7) the unit skin friction cab be obtained. In this case

$\alpha = 1 - \left(\frac{c_u - 25}{90}\right)$ for $25\ kPa < c_u < 70 kPa$

$$\alpha_{avg} = 1 - \left(\frac{60 - 25}{90}\right) = 0.61$$

$$(f_s)_{avg} = \alpha_{avg}(c_u)_{avg} = 0.61 \times 60 = 36.6\ kPa$$

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Eq. (3.11) can be used to calculate the unit end bearing as follows

$f_b = (c_u)_b N_c \leq 4000kPa$

Where $N_c = 6\left(1 + 0.2\frac{L}{B}\right) \leq 9.$

$$N_c = 6 \times \left(1 + 0.2 \times \frac{10}{0.4}\right) = 36 > 9 \ \therefore N_c = 9 \text{ (generally } \tfrac{L}{B} \text{ is >2.5 hence } N_c = 9)$$

$f_b = (c_u)_b N_c = 60 \times 9 = 540kPa$

Neglecting the weight of pile, Eq. (3.1) can be rewritten as

Ultimate Pile Capacity, $Q_{ult} = Q_s + Q_b = (A_s)_{total}(f_s)_{avg} + A_b f_b$

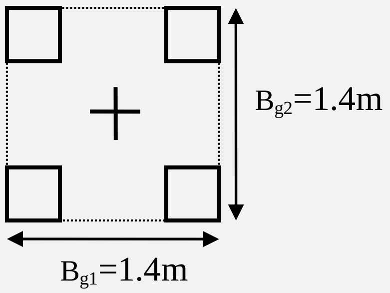
$$= 16 \ \times 36.6 + 0.16 \ \times 540 \ \ = 585.6 + 86.4 = 672 \ kN$$

If a normal level of field quality control cab be assured, BNBC (2015) suggests a minimum overall factor of safety 3.0.

Allowable Pile Capacity, $Q_{allow} = \frac{Q_{ult}}{FS} = \frac{672}{3.0} = 224 \ kN$

<u>Group Pile Action in Square/Rectangular Arrangement of Piles as a whole</u>

This pile carries load mainly in skin resistance ($Q_s > 0.8Q_b$) and is in cohesive soil.



Length of Pile group, L_g = 10m

Center to center distance, c=1m

No of piles in x-direction, n_1=2

No of piles in y-direction, n_2=2

Total no of piles, n = $n_1 \times n_2$=4

Width of Pile group in x-direction, B_{g1}= [(n_1-1)c+B]=[(2-1)×1+0.4]=1.4m

Width of Pile group in y-direction, B_{g2}= [(n_2-1)c+B]=[(2-1)×1+0.4]=1.4m

Skin friction area, $(A_s)_{total,g}$ = perimeter×length

$$= 2\left(B_{g1} + B_{g2}\right)L_g = 2 \times (1.4 + 1.4) \times 10 = 56 \ m^2$$

End bearing area, $(A_b)_g$ = $\left(B_{g1} \times B_{g2}\right) = (1.4 \times 1.4) = 1.96 \ m^2$

$(f_s)_{avg,g} = \alpha_{avg,g}(c_u)_{avg} = 1.00 \times 60 = 60 \ kPa$

Here adhesion factor α is considered as 1.00 since for group action failure occurs between soil to soil (soil to soil interaction) which is different from soil to pile interaction.

$(f_b)_g = (c_u)_b(N_c)_{group} \leq 4000kPa$

Where

$$(N_c)_g = 6\left(1 + 0.2\frac{L}{B}\right) \leq 9 \text{ for square footing}$$

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$$(N_c)_g = 6 \times \left(1 + 0.2 \times \frac{10}{1.4}\right) = 14.6 > 9 \ \therefore N_c = 9$$

(generally $\frac{L}{B}$ is >2.5 hence usually $(N_c)_g = 9$)

$(f_b)_g = (c_u)_b (N_c)_g = 60 \times 9 = 540 kPa$

Neglecting the weight of pile, Eq. (3.1) can be rewritten as

Ultimate Pile Capacity, $(Q_{ult})_g = (Q_s)_g + (Q_b)_g = (A_s)_{total,g}(f_s)_{avg,g} + (A_b)_g (f_b)_g$

$$= 56 \ \times 60 + 1.96 \ \times 540 \ \ = 3360 + 1058.4 = 4418.4 \ kN$$

Allowable Pile Capacity, $(Q_{allow})_g = \frac{(Q_{ult})_g}{FS} = \frac{4418.4}{3.0} = 1472.8 \ kN$

Allowable Pile Capacity for a single pile in group from group action,

$$Q_{allow} = \frac{(Q_{allow})_g}{no\ of\ piles} = \frac{1472.8}{4} = 368.2 \ kN$$

Since this capacity is greater than the capacity of a single pile in individual action. Hence

Allowable capacity of a single pile, $Q_{allow} = 224 \ kN$ **(Ans)**

Problems

3.1 Define Driven pile and bored pile

3.2 Write short note on

(a) Load transfer mechanism of piles

(b) Group pile action

3.3 Write down the factors that should be considered while determining the geotechnical design axial capacity of pile foundation.

3.4 Write down the name of the methods which are available to determine the ultimate bearing capacity (skin friction and/or end bearing) of a single vertical pile.

3.5 Repeat Example 3.1 for precast driven pile. Also find the group capacity of four piles having center to center distance of 1.5 m.

3.6 Find out the allowable capacity of a single pile in a pile group consisting six 15m long bored pile of 500mm diameter with 1m center to center spacing in rectangular arrangement. Pile is to be cast into a uniform saturated medium soft clay layer with average undrained shear strength of 55 kPa.

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