

Foundation Engineering I: Lecture 1

Introduction to Foundation Engineering and Subsurface Exploration

Week 1 | Contact time: 4 hours (2 h theory + 2 h tutorial)

1.1 Course Plan for the First Semester

Week	Lecture	Topic
1	1	Introduction to foundation engineering; subsurface exploration
2	2	Lateral earth pressure I: at-rest and Rankine theory (granular soils)
3	3	Lateral earth pressure II: c - ϕ soils, Coulomb theory, sloping and layered backfill
4	4	Retaining walls, I: types, proportioning, overturning and sliding
5	—	First monthly examination (Lectures 1–4)
6	5	Retaining walls II: bearing pressure, eccentricity and complete stability check
7	6	Retaining walls III: joints, drainage, filters, and mechanically stabilized earth
8	7	Sheet pile walls I: types, construction, cantilever walls in sand
9	8	Sheet pile walls II: cantilever walls in clay, anchored walls, anchors
10	—	Second monthly examination (Lectures 5–8)
11	9	Bearing capacity, I: failure modes and Terzaghi theory
12	10	Bearing capacity II: general equation, water table, local shear, factor of safety
13	11	Bearing capacity III: eccentric and inclined loads, in-situ test methods
14	12	Stresses and settlement of shallow foundations; proportioning of footings
15	—	Semester examination (comprehensive)

1.2 What Is a Foundation?

A structure consists of two parts. The **superstructure** is the part above ground level. The **substructure**, or foundation, transfers the loads of the superstructure to the ground. The **foundation soil** is the part of the ground that receives additional stress because of the structure. Foundation engineering applies soil mechanics, structural analysis and engineering judgement to design this transfer safely and economically.

Every foundation must satisfy two independent criteria:

1. **Stability (strength) criterion.** The soil must not fail in shear under the applied load. A factor of safety is applied to the ultimate bearing capacity. Typical values are 2.5–3.0 for footings.
2. **Serviceability (deformation) criterion.** Total and differential settlements must stay below the limits that the structure tolerates. Typical limits are 25 mm total settlement

for isolated footings on sand and $1/300$ angular distortion for framed buildings (Skempton and MacDonald 1956).

The design is governed by whichever criterion gives the larger footing. On loose sand and soft clay, the settlement criterion usually governs. On dense sand and stiff clay, the shear criterion often governs. **Figure 1** summarizes the requirements.

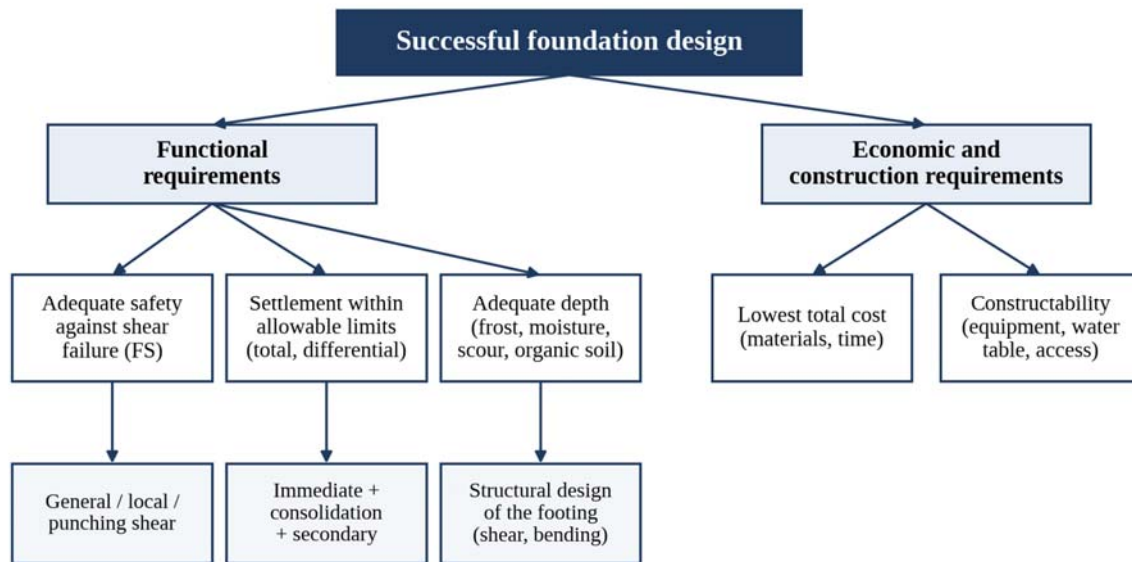


Figure 1: Requirements for a successful foundation design.

1.3 Classification of Foundations

Foundations are classified by the ratio of embedment depth D_f to width B and by the load-transfer mechanism (See **Table 1** and **Figure 2**)

1. **Shallow foundations** transfer the load mainly through the base. Terzaghi defined them by $D_f/B \leq 1$. Later studies extend the shallow behaviour to D_f/B of about 3–4. Types: spread (isolated) footing, strip (wall) footing, combined footing, strap footing, and mat (raft).
2. **Deep foundations** transfer the load through the shaft (skin friction) and the tip (end bearing) to deeper, stronger strata. Types: driven piles, bored piles, drilled shafts (piers) and caissons. They are the subject of the second semester.
3. **Earth-retaining structures** resist lateral earth pressure: gravity and cantilever walls, sheet pile walls, braced cuts and mechanically stabilized earth (MSE) walls. They form the first half of this semester.

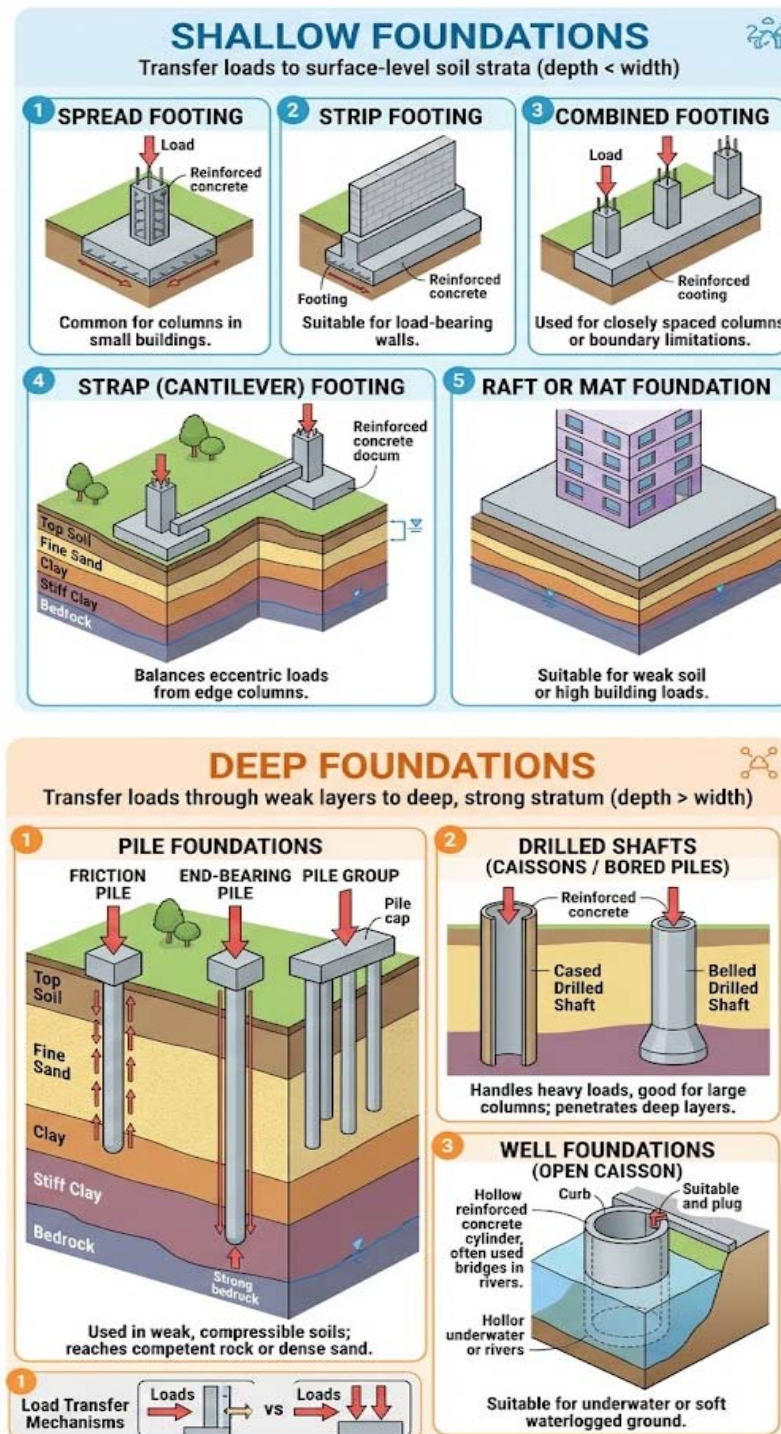


Figure 2: Main types of shallow and deep foundations.

Table 1: Guide to the selection of the foundation type.

Situation	Usual choice	Reason
Firm soil near surface, light to moderate loads	Spread or strip footing	Cheapest; simple construction
Columns close together, low bearing capacity	Mat (raft)	Footings would overlap; reduces differential settlement
Exterior column on property line	Combined or strap footing	Avoids an eccentrically loaded footing
Weak soil over firm stratum at depth	Piles or drilled shafts	Load carried to the firm stratum
Scour risk (bridge piers, river works)	Piles below scour depth	Shallow soil may be removed by flow
Uplift or large lateral loads	Piles, anchors	Tension and lateral resistance needed

1.4 Ground Conditions Relevant to Water Resources Projects in Iraq

Most water resources structures in Iraq (barrages, regulators, pumping stations, canal structures, river protection works) are built on the Mesopotamian alluvial plain. The plain consists of fluvial deposits of the Tigris and Euphrates: interbedded silty clay, clayey silt and fine sand. Three features control foundation design:

1. **Shallow groundwater.** Near rivers and irrigation canals the water table is often within a few metres of the surface. It reduces effective stress, bearing capacity and passive resistance. It also governs dewatering.
2. **Soft to medium clays.** Upper clay layers are often normally consolidated or lightly overconsolidated. Consolidation settlement then governs the design of footings and embankments.
3. **Gypseous soils.** In the western and central regions, soils with significant gypsum content lose strength and collapse when soaked. Leakage from canals and pipes can dissolve gypsum and create cavities.

These features explain why water table effects, settlement and drainage receive special attention in foundation engineering.

1.5 Subsurface Exploration

1.5.1 Purpose

Subsurface exploration (site investigation) supplies the data for design. Its objectives are:

1. to identify the soil layers, their thickness and their lateral extent;

2. to locate the groundwater table and measure its seasonal variation;
3. to obtain disturbed and undisturbed samples for laboratory tests;
4. to perform in-situ tests (SPT, CPT, vane, plate load);
5. to detect special problems: collapsible, expansive, gypseous or organic soils, cavities, contamination;
6. to recommend the foundation type, depth and allowable bearing pressure.

1.5.2 Stages of Exploration

Figure 3 shows the six stages. The cost of a site investigation is small compared with the cost of the structure. Insufficient investigation is the most common cause of foundation problems.

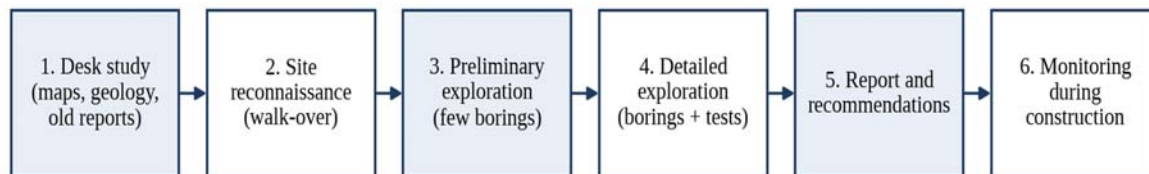


Figure 3: Stages of a subsurface exploration programme.

1.5.3 Boring Methods

Table 2: Common boring methods.

Method	Principle	Suitable for	Limitations
Hand / power auger	Rotating helical auger removes soil	Shallow depths, cohesive soils	Collapses below water table in sand
Wash boring	Water jet loosens soil; cuttings flushed up	Soft soils, sand, silt	Samples disturbed; soil layers mixed
Rotary drilling	Rotating bit with drilling fluid	All soils and rock; deep holes	Needs drilling mud; higher cost
Percussion (cable tool)	Heavy tool dropped repeatedly	Gravel, boulders	Very disturbed samples
Hollow-stem auger	Auger acts as casing	Sampling below water table	Limited in dense gravel

1.5.4 Depth and Spacing of Borings

A boring must go deep enough to cover all layers that contribute to settlement or failure. Two practical rules are used together.

1. **Stress criterion.** Continue the boring to the depth where the stress increase $\Delta\sigma$ caused by the foundation falls to about 10% of the applied pressure q , and below that to the

depth where $\Delta\sigma \leq 0.05\sigma'_v$. **Figure 4** shows that $\Delta\sigma = 0.1q$ at about $2B$ below a square footing and about $6B$ below a strip footing.

2. **Rule of thumb for buildings (ASCE 1972).** $D_b = 3S^{0.7}$ for light steel or narrow concrete buildings and $D_b = 6S^{0.7}$ for heavy steel or wide concrete buildings, where D_b is in metres and S is the number of storeys. (see **Table 3**)
3. If rock is reached, core at least 3 m into it to confirm that it is bedrock and not a boulder.

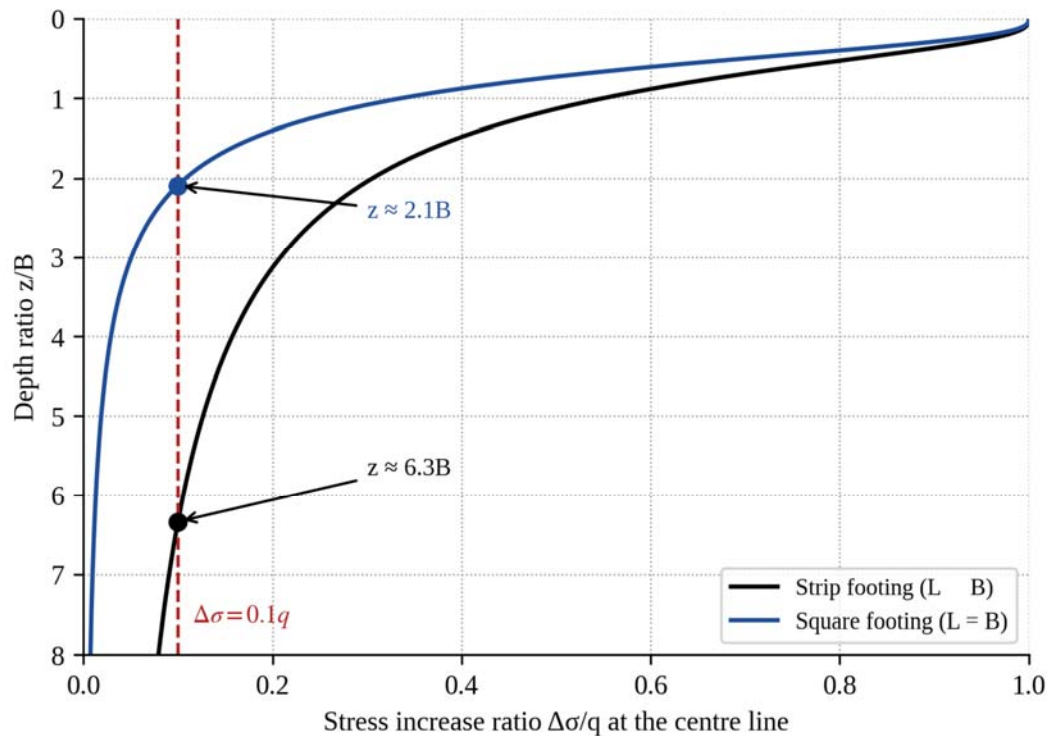


Figure 4: Decay of vertical stress increase below the centre of strip and square footings (Boussinesq solution) and the $0.1q$ depth criterion.

Table 3: Approximate spacing of borings (after Sowers and Sowers 1970).

Type of project	Spacing of borings (m)
Multistorey buildings	10–30
One-storey industrial plants	20–60
Highways	250–500
Residential subdivision	250–500
Dams and dikes	40–80

1.5.5 Soil Sampling

Disturbed samples keep the soil grains but not the structure. They serve for classification, grain size, Atterberg limits and water content. Undisturbed samples keep the structure and water

content. They serve for consolidation, triaxial and unconfined compression tests. Thin-walled (Shelby) tubes give undisturbed samples in soft to stiff clay. The degree of disturbance depends on the area ratio:

$$A_R(\%) = \frac{D_o^2 - D_i^2}{D_i^2} \times 100 \quad (1)$$

where D_o and D_i are the outside and inside diameters of the sampler. A sample is considered undisturbed when $A_R \leq 10\%$. The standard split spoon ($D_o = 50.8$ mm, $D_i = 34.9$ mm) gives $A_R \approx 112\%$, so its samples are disturbed. In rock, core recovery and the rock quality designation are recorded:

$$\text{Recovery ratio} = \frac{\text{length of core recovered}}{\text{length of core run}}, \quad RQD = \frac{\sum \text{pieces} \geq 100 \text{ mm}}{\text{length of core run}} \quad (2)$$

1.6 Standard Penetration Test (SPT)

The SPT is the most widely used in-situ test in Iraq. A split-spoon sampler is driven into the soil at the bottom of the borehole by a 63.5 kg hammer falling 760 mm. The number of blows is recorded for three successive increments of 150 mm. The first increment is a seating drive. The **standard penetration number N** is the sum of the blows for the second and third increments (300 mm). **Figure 5** shows the test.

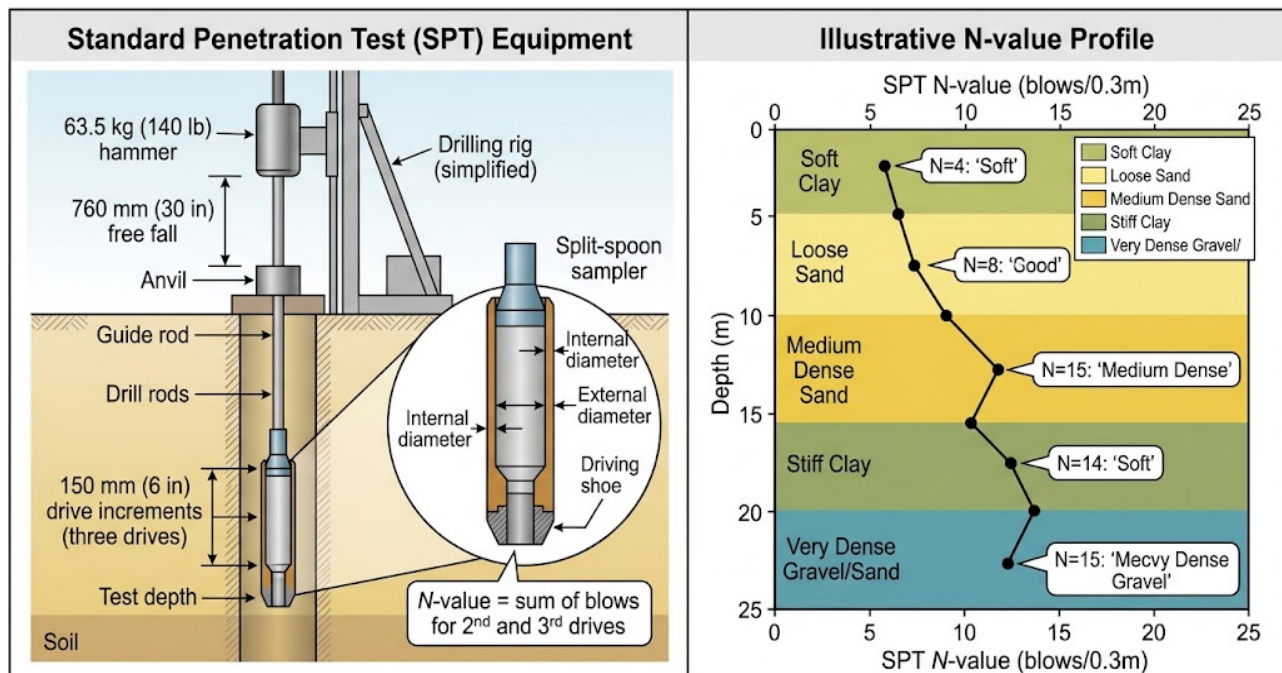


Figure 5: Standard penetration test: equipment (left) and an illustrative N-value profile (right).

1.6.1 Corrections

The energy delivered to the rods varies with the hammer type and the equipment. Results are normalized to 60% of the theoretical free-fall energy: (see **Table 4** and **Figure 6**)

$$N_{60} = \frac{N \eta_H \eta_B \eta_S \eta_R}{60} \quad (3)$$

where η_H = hammer efficiency (%), η_B = borehole diameter correction, η_S = sampler correction, and η_R = rod length correction. For sands, N_{60} is also normalised to an effective overburden pressure of $p_a = 100$ kPa:

$$(N_1)_{60} = C_N N_{60}, \quad C_N = \left(\frac{p_a}{\sigma'_v} \right)^{0.5} \leq 2.0 \quad (4)$$

Table 4: SPT correction factors (after Seed et al. 1985 and Skempton 1986).

Correction	Condition	Value
η_H (hammer efficiency)	Safety hammer, rope and pulley	60%
	Donut hammer, rope and pulley	45%
	Automatic trip hammer	70–80%
η_B (borehole diameter)	60–120 mm / 150 mm / 200 mm	1.0 / 1.05 / 1.15
η_S (sampler)	Standard sampler / with liner in dense sand	1.0 / 0.8
η_R (rod length)	>10 m / 6–10 m / 4–6 m / 0–4 m	1.0 / 0.95 / 0.85 / 0.75

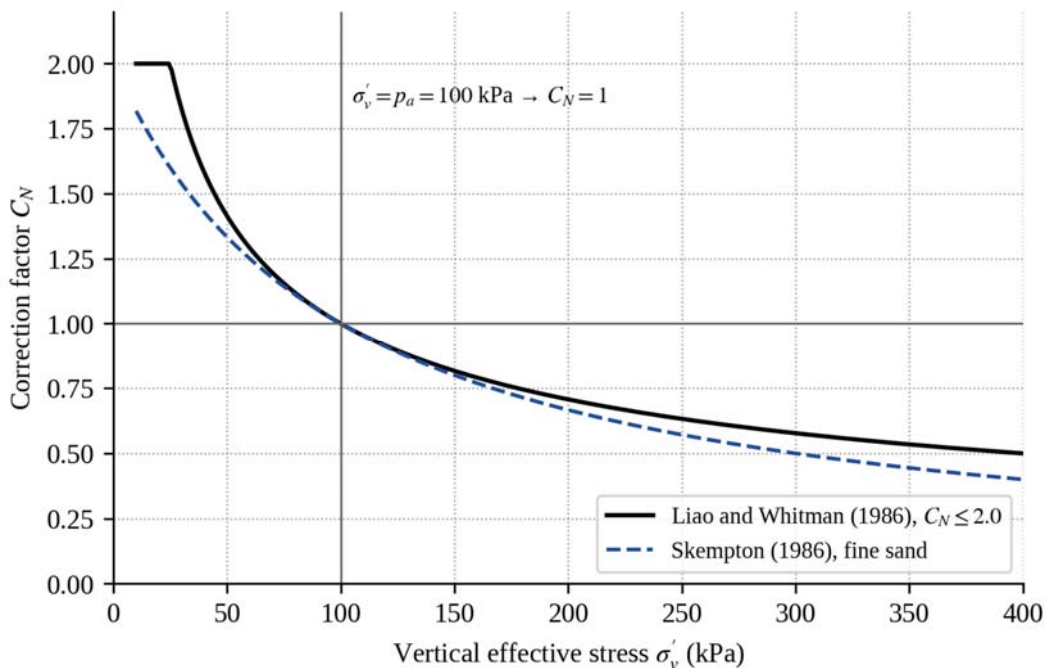


Figure 6: Overburden correction factor C_N versus vertical effective stress.

1.6.2 Correlations

Table 5: Descriptive terms from N_{60} (after Terzaghi and Peck).

Sand: N_{60}	Relative density	Clay: N_{60}	Consistency
0–4	Very loose	< 2	Very soft
4–10	Loose	2–4	Soft
10–30	Medium dense	4–8	Medium
30–50	Dense	8–15	Stiff
> 50	Very dense	15–30	Very stiff
		> 30	Hard

Commonly used correlations are listed below. They are approximate and must be checked against laboratory tests on samples from the same site.

$$D_r \approx \sqrt{\frac{(N_1)_{60}}{60}} \quad (\text{Skempton 1986, sands}) \quad (5)$$

$$\phi' (^{\circ}) \approx 27.1 + 0.3N_{60} - 0.00054N_{60}^2 \quad (\text{Wolff 1989}) \quad (6)$$

$$c_u \approx KN_{60}, \quad K = 3.5 \text{ to } 6.5 \text{ kPa} \quad (\text{Stroud 1974, clays}) \quad (7)$$

1.7 Cone Penetration Test (CPT)

A cone with a 60° apex and a base area of 10 cm^2 is pushed into the ground at 20 mm/s . The test records the cone (tip) resistance q_c and the sleeve friction f_s continuously. The piezocone (CPTu) also records pore pressure. The friction ratio $R_f = f_s/q_c$ identifies the soil: clays give high R_f , sands give low R_f . The undrained shear strength of clay is estimated by: (see **Figure 7**)

$$c_u = \frac{q_c - \sigma_{vo}}{N_k} \quad (8)$$

where σ_{vo} is the total overburden stress and N_k is the cone factor, usually 15–20. The CPT gives a continuous profile and good repeatability. It cannot penetrate gravel or cemented layers and gives no sample.

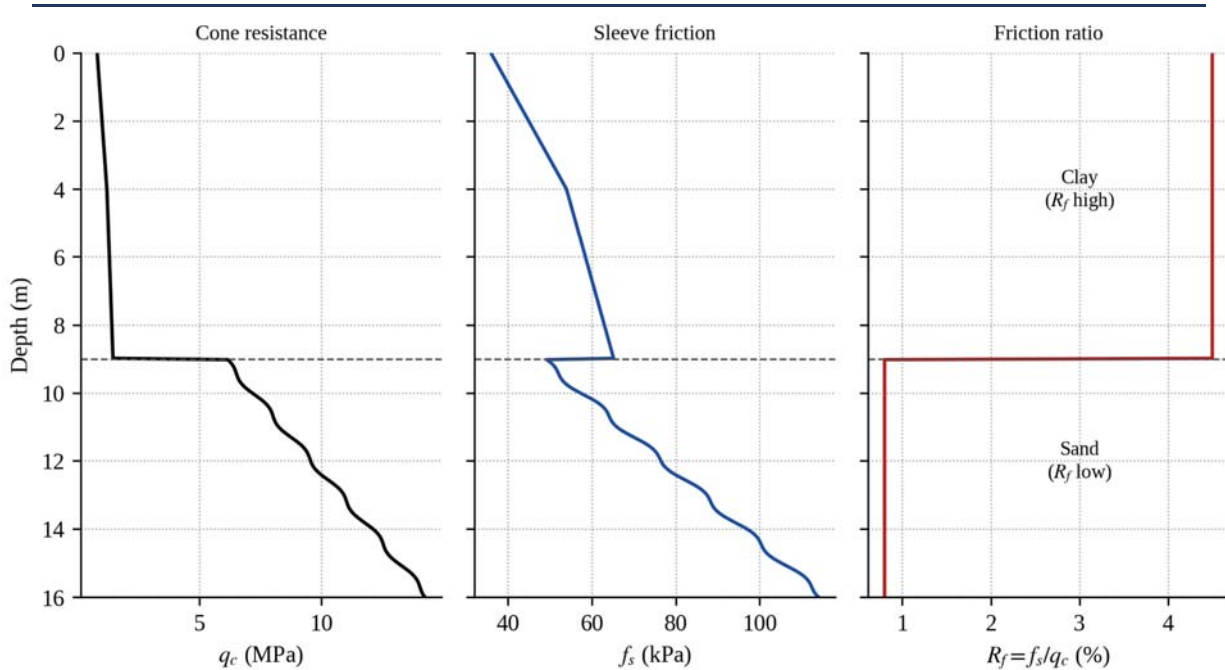


Figure 7: Illustrative CPT record: a soft clay over a medium dense sand. The change in friction ratio marks the boundary.

1.8 Other In-Situ Tests and Groundwater Observation

1. **Field vane shear test.** Measures c_u of soft clay. For a vane with $H/D = 2$: $T = \pi c_u D^2 (H/2 + D/6)$. The field value is reduced by Bjerrum's factor λ that depends on the plasticity index.
2. **Pressuremeter test.** Expands a cylindrical probe in a borehole and gives the soil modulus and limit pressure.
3. **Plate load test.** Loads a rigid plate at foundation level.
4. **Geophysical methods.** Seismic refraction and electrical resistivity give continuous sections between borings.
5. **Groundwater observation.** Record the water level in each borehole at least 24 hours after drilling. Use piezometers for long-term readings and for artesian conditions.

1.9 The Geotechnical Report and the Borehole Log

The report presents the findings and recommendations. It contains: scope and site description; geology; exploration methods; boring logs; laboratory and field results; groundwater conditions; recommended foundation type, depth and allowable bearing pressure; expected settlements; and construction recommendations (dewatering, excavation support, sulphate-resistant cement in gypseous soils). **Figure 8** shows a typical log.

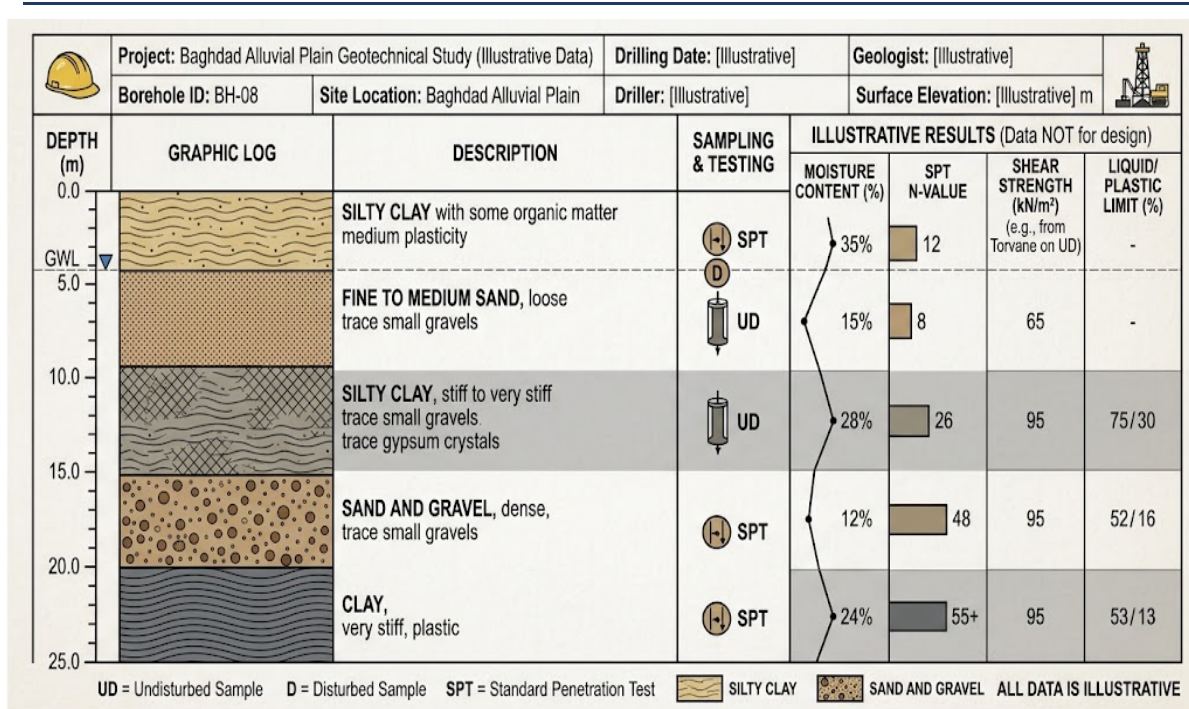


Figure 8: Illustrative borehole log of the type produced for sites on the Baghdad alluvial plain (UD = undisturbed sample).

1.10 Soil Parameters Needed for Foundation Design

The investigation must deliver the parameters that the later lectures use. **Table 6** links each parameter to its source and to the lecture where it is applied.

Table 6: Parameters required from the site investigation.

Parameter	Symbol	Source (laboratory / field)	Used in
Unit weight (bulk, saturated)	$\gamma, \gamma_{\text{sat}}$	Undisturbed samples; density tests	All lectures
Effective friction angle	ϕ'	Direct shear, CD/CU triaxial; SPT, CPT correlations	Earth pressure, bearing capacity
Effective cohesion	c'	CD/CU triaxial	Earth pressure, bearing capacity
Undrained shear strength	c_u	UU triaxial, unconfined, vane, CPT	Short-term stability in clay
Compression index, recompression index	C_c, C_r	Oedometer test	Consolidation settlement (L12)
Preconsolidation pressure	σ'_c	Oedometer test	OCR, K_0 , settlement
Modulus of elasticity, Poisson ratio	E_s, ν_s	Triaxial, pressuremeter, correlations	Immediate settlement
Groundwater level	z_w	Boreholes, piezometers	Effective stress, uplift, dewatering
Gypsum and sulphate content	$\text{SO}_3\%$	Chemical tests	Collapse risk; cement type

1.10.1 Unified Soil Classification System (Review)

Borehole logs describe soils with the Unified Soil Classification System (USCS). Coarse-grained soils have more than 50% retained on the No. 200 (0.075 mm) sieve. Fine-grained soils have 50% or more passing it. The second letter describes the grading (W, P) or the plasticity (L, H). **Table 7** recalls the group symbols that appear most often in Iraqi site reports.

Table 7: USCS group symbols and their typical foundation behaviour.

Group symbol	Description	Typical foundation behaviour
GW, GP	Well / poorly graded gravel	High strength, low compressibility, free draining
SW, SP	Well / poorly graded sand	Good bearing when dense; settlement governs when loose
SM, SC	Silty sand, clayey sand	Moderate strength; sensitive to water
ML	Silt of low plasticity	Poor when saturated; erodible; frost and piping susceptible
CL	Clay of low plasticity (lean clay)	Consolidation settlement; time-dependent
CH	Clay of high plasticity (fat clay)	Large settlement; swelling and shrinkage
OL, OH, Pt	Organic silt and clay, peat	Unsuitable; remove or bypass with piles

Limits of the methods

Correlations between N or q_c and soil parameters are empirical. Scatter of $\pm 30\%$ or more is common.

The SPT N value is sensitive to the equipment and the operator. Report the hammer type and energy.

Borings describe the ground only at their locations. Interpolation between borings assumes continuous layers. Alluvial deposits often contain lenses that a widely spaced programme misses.

Tutorial: Worked Examples**Example 1**

An SPT was performed at a depth of 7.0 m in sand using a donut hammer ($\eta_H = 45\%$), a 150 mm borehole and a standard sampler. The field value is $N = 18$. The vertical effective stress at the test depth is 85 kPa. Determine N_{60} , $(N_1)_{60}$, the relative density and the friction angle.

Solution

Correction factors (Table 4). $\eta_H = 45\%$ (donut hammer), $\eta_B = 1.05$ (150 mm borehole), $\eta_S = 1.0$ (standard sampler), $\eta_R = 0.95$ (rod length 6–10 m; the rod length is taken equal to the test depth, 7.0 m).

Energy correction, Eq. (3):

$$N_{60} = \frac{N \eta_H \eta_B \eta_S \eta_R}{60} = \frac{18(45)(1.05)(1.0)(0.95)}{60} = 13.5$$

Overburden correction, Eq. (4):

$$C_N = \left(\frac{100}{85} \right)^{0.5} = 1.085 \quad (\leq 2.0); \quad (N_1)_{60} = C_N N_{60} = 1.085(13.5) = 14.6$$

Relative density, Eq. (5):

$$D_r = \sqrt{\frac{(N_1)_{60}}{60}} = \sqrt{\frac{14.6}{60}} = 0.49 = 49\%$$

Table 5 gives the same description: $N_{60} = 13.5$ lies in the range 10–30, so the sand is **medium dense**.

Friction angle, Eq. (6):

$$\phi' = 27.1 + 0.3(13.5) - 0.00054(13.5)^2 = 31.0^\circ$$

Answer: $N_{60} = 13.5$; $(N_1)_{60} = 14.6$; $D_r \approx 49\%$ (medium dense); $\phi' \approx 31^\circ$.

Example 2

An 8-storey reinforced concrete building has wide floor plates. (a) Estimate the boring depth by the ASCE rule of thumb. (b) Check the depth with the stress criterion (Figure 4) for a 3 m $\times$ 3 m footing carrying $q = 250$ kPa. (c) State the spacing of the borings (Table 3).

Solution

(a) Rule of thumb (Section 1.5.4, item 2). A wide concrete building takes the heavy-building formula, with $S = 8$ storeys:

$$D_b = 6S^{0.7} = 6(8)^{0.7} = 6(4.29) = 25.7 \text{ m}$$

For comparison, a light steel or narrow concrete building would need only $D_b = 3S^{0.7} = 3(4.29) = 12.9 \text{ m}$.

(b) Stress criterion (Section 1.5.4, item 1). The boring must reach the depth where the stress increase falls to 10% of the footing pressure:

$$\Delta\sigma = 0.1q = 0.1(250) = 25 \text{ kPa,} \quad \text{i. e.} \quad \Delta\sigma/q = 0.1$$

Enter Figure 4 at $\Delta\sigma/q = 0.1$ (the red dashed line). Go down to the square-footing curve ($L = B$). The curve crosses the line at $z/B \approx 2.1$:

$$z = 2.1B = 2.1(3.0) = 6.3 \text{ m below the footing base}$$

The stress criterion gives only 6.3 m because it treats one footing alone. Under a building the stress bulbs of the adjacent footings overlap. The footings then act like one wide loaded area, and the stressed zone goes much deeper. The rule of thumb includes this effect, so the larger depth governs.

(c) Spacing (Table 3). For a multistorey building, place the borings 10–30 m apart.

Answer: borings about 26 m deep below the ground surface, at 10–30 m spacing. If rock is met above this depth, core at least 3 m into it (Section 1.5.4, item 3).

Example 3

A thin-walled tube has $D_o = 76.2 \text{ mm}$ and $D_i = 73.0 \text{ mm}$. A 1.5 m rock core run recovered 1.32 m of core; the pieces longer than 100 mm add up to 0.96 m. Find the area ratio, the recovery ratio and the RQD, and describe the rock quality.

Solution

Area ratio, Eq. (1):

$$A_R = \frac{D_o^2 - D_i^2}{D_i^2} \times 100 = \frac{76.2^2 - 73.0^2}{73.0^2} \times 100 = \frac{5806.4 - 5329.0}{5329.0} \times 100 = 9.0\%$$

$A_R = 9.0\% \leq 10\%$, so the tube gives **undisturbed** samples.

Recovery ratio and RQD, Eq. (2):

$$\text{Recovery ratio} = \frac{1.32}{1.50} = 0.88 \text{ (88\%)}, \quad RQD = \frac{0.96}{1.50} = 0.64 \text{ (64\%)}$$

Rock quality from the RQD (Deere 1964): 0–25 % very poor; 25–50 % poor; 50–75 % fair; 75–90 % good; 90–100 % excellent.

Answer: $A_R = 9.0\%$ (undisturbed); recovery = 0.88; RQD = 64 %, which describes **fair** rock.

Example 4

A field vane ($D = 65$ mm, $H = 130$ mm) required a maximum torque of 40 N·m to shear a soft clay. Find c_u . At the same depth a CPT gave $q_c = 0.62$ MPa with $\sigma_{vo} = 85$ kPa. Estimate c_u from the CPT with $N_k = 17$ and compare the two values.

Solution

Field vane (Section 1.8). $H/D = 130/65 = 2$, so $T = \pi c_u D^2 (H/2 + D/6)$ applies. Solve for c_u ($D = 0.065$ m, $H/2 = 0.065$ m, $D/6 = 0.0108$ m):

$$c_u = \frac{T}{\pi D^2 (H/2 + D/6)} = \frac{40}{\pi (0.065)^2 (0.065 + 0.0108)} = 39\,740 \text{ N/m}^2 = 39.7 \text{ kPa}$$

CPT, Eq. (8), with $q_c = 0.62$ MPa = 620 kPa:

$$c_u = \frac{q_c - \sigma_{vo}}{N_k} = \frac{620 - 85}{17} = 31.5 \text{ kPa}$$

For the usual range $N_k = 15$ – 20 (Section 1.7), the CPT gives $c_u = 535/20$ to $535/15 = 26.8$ – 35.7 kPa.

Comparison. The CPT value is 21 % lower than the vane value ($31.5/39.7 = 0.79$). The field vane value is reduced by Bjerrum's factor λ before design (Section 1.8). For an assumed $\lambda = 0.85$, the corrected vane value is $0.85 \times 39.7 = 33.8$ kPa, which lies inside the CPT range.

Answer: c_u (vane) = 39.7 kPa; c_u (CPT, $N_k = 17$) = 31.5 kPa.

Example 5

Use the borehole log of Figure 8. The water table (GWL) is at 4.2 m. Take $\gamma = 18.0 \text{ kN/m}^3$ above the water table and $\gamma_{\text{sat}} = 19.0 \text{ kN/m}^3$ below it for all layers. The SPT used a safety hammer ($\eta_H = 60\%$), a 100 mm borehole and a standard sampler; take the rod length equal to the test depth. (a) For the two granular layers, use the SPT at the middle of each layer: $N = 8$ at 7.0 m (fine to medium sand) and $N = 48$ at 17.5 m (sand and gravel). Compute σ'_v , N_{60} , $(N_1)_{60}$, D_r and ϕ' . (b) For the silty clay, $N = 26$ at 12.5 m. Estimate c_u with Eq. (7) and compare it with the shear strength in the log.

Solution

(a) **Effective vertical stress** below the water table (z in m):

$$\sigma'_v = 18.0(4.2) + (19.0 - 9.81)(z - 4.2) = 75.6 + 9.19(z - 4.2) \text{ kPa}$$

$\eta_H = 60\%$, $\eta_B = 1.0$ and $\eta_S = 1.0$, so $N_{60} = N \eta_R$ (Eq. 3). $\eta_R = 0.95$ at 7.0 m (6–10 m) and 1.0 at 17.5 m (> 10 m). C_N , $(N_1)_{60}$, D_r and ϕ' follow from Eqs. (4), (5) and (6):

Depth (m)	Layer	N	σ'_v (kPa)	η_R	N_{60}	C_N	$(N_1)_{60}$	D_r (%)	ϕ' (°)
7.0	Fine to medium sand	8	101.3	0.95	7.6	0.99	7.5	35	29.3
17.5	Sand and gravel	48	197.8	1.00	48.0	0.71	34.1	75	40.3

Check with Table 5: $N_{60} = 7.6$ is **loose** and $N_{60} = 48$ is **dense**. Both agree with the descriptions in the log. $C_N < 1$ at 17.5 m because $\sigma'_v = 198 \text{ kPa}$ exceeds 100 kPa.

(b) **Silty clay at 12.5 m.** $\eta_R = 1.0$, so $N_{60} = 26$. Eq. (7):

$$c_u = KN_{60} = (3.5 \text{ to } 6.5)(26) = 91 \text{ to } 169 \text{ kPa}$$

The log gives 95 kPa, at the lower end of this range. Table 5 describes $N_{60} = 26$ as very stiff; the log says stiff to very stiff. The correlation is approximate, so laboratory tests on the undisturbed (UD) samples must confirm c_u .

Design values. Fine to medium sand (4.2–10 m): $\phi' \approx 29^\circ$. Sand and gravel (15–20 m): Eq. (6) gives $\phi' \approx 40^\circ$. Eq. (6) uses N_{60} , which is not corrected for the large overburden at 17.5 m ($C_N = 0.71$), so a cautious design value is lower, about $36\text{--}38^\circ$, until laboratory or CPT data confirm it. Silty clay: $c_u \approx 95 \text{ kPa}$.

Key Terms (English – Arabic)

English term	المصطلح العربي
Foundation / substructure	الأساس / المنشأ السفلي
Superstructure	المنشأ العلوي
Shallow foundation	أساس ضحل
Deep foundation	أساس عميق
Mat (raft) foundation	أساس حصيري
Site investigation	التحريات الموقعية
Borehole / boring	حفرة / ثقب استكشافي
Undisturbed sample	نموذج غير مشوش
Standard penetration test	فحص الاختراق القياسي
Cone penetration test	فحص الاختراق المخروطي
Groundwater table	منسوب المياه الجوفية
Borehole log	سجل الحفرة

References for Further Reading

Das, B.M. and Sivakugan, N. (2019). *Principles of Foundation Engineering*, 9th ed. Cengage Learning, Boston.

Bowles, J.E. (1996). *Foundation Analysis and Design*, 5th ed. McGraw-Hill, New York.

Coduto, D.P., Kitch, W.A. and Yeung, M.R. (2016). *Foundation Design: Principles and Practices*, 3rd ed. Pearson, Boston.

Terzaghi, K., Peck, R.B. and Mesri, G. (1996). *Soil Mechanics in Engineering Practice*, 3rd ed. Wiley, New York.

Skempton, A.W. and MacDonald, D.H. (1956). The allowable settlements of buildings. *Proceedings of the Institution of Civil Engineers*, 5(6), 727–768.

Homework No. 1

Answer all questions. Show every step, draw the pressure or force diagrams, and state units. Take $\gamma_w = 9.81 \text{ kN/m}^3$ unless stated otherwise.

Q1. List the two criteria that a foundation must satisfy. Explain which one usually governs for (a) a footing on loose sand and (b) a footing on dense gravel.

Q2. A Shelby tube has $D_o = 89 \text{ mm}$ and wall thickness 1.65 mm . Compute the area ratio and state whether the samples can be considered undisturbed.

-
- Q3. An SPT in sand at 9 m depth gave $N = 22$ with a safety hammer ($\eta_H = 60\%$), 100 mm borehole and standard sampler. $\gamma = 17.5 \text{ kN/m}^3$ above the water table at 3 m and $\gamma_{\text{sat}} = 19.5 \text{ kN/m}^3$ below. Find N_{60} , $(N_1)_{60}$, D_r and ϕ' .
- Q4. A vane 75 mm in diameter and 150 mm high needed a torque of 52 N·m. Compute c_u . If the plasticity index is 40% and Bjerrum's $\lambda = 0.85$, what is the design c_u ?
- Q5. Using the Boussinesq solution (or the chart in **Figure 4**), find the depth at which $\Delta\sigma = 0.1q$ below the centre of a 2 m wide strip footing and of a $2 \text{ m} \times 2 \text{ m}$ square footing. Explain the difference.
- Q6. A CPT in a clay layer gave $q_c = 1.1 \text{ MPa}$ at 6 m depth. The water table is at 1.5 m; $\gamma_{\text{sat}} = 19 \text{ kN/m}^3$. Estimate c_u with $N_k = 15$ and $N_k = 20$. Classify the consistency.