

NIGERIA MARITIME UNIVERSITY

Okerenkoko, Warri South-West, Delta State

Department of Marine Geology

MGY 401

Geotechnics

Lecture Note

Advanced Soil Mechanics · Foundation Engineering · Site and Applied Practice

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Course information

This note covers three parts of MGY 401: advanced soil mechanics, foundation engineering, and site and applied practice. Rock mass classification (RQD, RMR and the Q-system) and ground improvement and special topics are covered in the other part of the course. The note builds on the soil classification and site investigation material of MGY 321.

Item	Detail
Course	MGY 401: Geotechnics
Level	400 level, first semester, 2026/2027 session
Department	Marine Geology
Institution	Nigeria Maritime University, Okerenkoko, Warri South-West, Delta State
Lecturer	Dr. Daniel Tuoyo Amiwero
Part covered by this note	Advanced soil mechanics; foundation engineering; site and applied practice

Learning outcomes

At the end of this part of the course, you should be able to:

1. Construct and use a flow net to estimate seepage, exit gradients and the safety of excavations against piping and heave.
2. Calculate the stress increase beneath foundations and embankments.
3. Interpret oedometer tests and calculate the magnitude and rate of consolidation settlement.
4. Interpret shear strength tests and distinguish drained and undrained behaviour.
5. Calculate lateral earth pressures on retaining walls and quay walls, and assess the stability of slopes.
6. Select a suitable foundation type, and calculate the bearing capacity and settlement of shallow foundations and the axial capacity of piles.
7. Plan a ground investigation for a foundation, interpret SPT and CPT results, and derive design parameters.
8. Describe the content of geotechnical reports and apply geotechnical principles to typical projects in the Niger Delta and coastal Nigeria.

Why geotechnics matters to a marine geologist

The Niger Delta is one of the most difficult places in the world to build. Its soft, compressible clays and peats, loose saturated sands, high water table, tides, scour and subsidence challenge every jetty, quay wall, road embankment, pipeline, building and offshore platform. Marine geologists in the oil and gas, ports, dredging and consulting industries plan and supervise the investigations, log the cores, run the tests and interpret the results on which these structures are designed. This note gives the soil mechanics and foundation engineering needed to do that work well, and to speak the language of the geotechnical engineers who use the results.

Section 1: Advanced soil mechanics

Soil mechanics provides the tools to predict how the ground will respond to the loads, excavations and changes in water pressure imposed by engineering works. MGY 321 introduced the classification of soils, phase relationships and the principle of effective stress. This section develops the main areas of soil mechanics needed for foundation design: seepage, stress distribution, consolidation, shear strength, earth pressure and slope stability.

1.1 Stresses in the ground

The **principle of effective stress** (Terzaghi) states that the behaviour of a saturated soil is governed by the effective stress, the total stress minus the pore water pressure:

$$\sigma' = \sigma - u$$

At depth z in level ground, the vertical stresses are calculated from the unit weights of the layers above (MGY 321, Section 1.5). The horizontal effective stress is a fraction of the vertical effective stress, given by the **coefficient of earth pressure at rest** (K_0):

$$\sigma'_h = K_0 \, \sigma'_v$$

For normally consolidated soils, K_0 is commonly estimated from the angle of shearing resistance using Jáky's formula:

$$K_0 = 1 - \sin\phi'$$

For a sand with $\phi' = 32°$, $K_0 = 1 - 0.53 = 0.47$. In overconsolidated clays, which were once buried more deeply, K_0 can exceed 1.

1.2 Seepage and flow nets

Water flows through soil from points of high total head to points of low total head. The flow obeys **Darcy's law**, $v = k\,i$, where k is the coefficient of permeability and i the hydraulic gradient. Typical values of k range from about 10^{-2} m/s for clean gravel to 10^{-4} to 10^{-6} m/s for sands, 10^{-7} m/s for silts and less than 10^{-9} m/s for clays.

In two dimensions, steady flow through a uniform soil satisfies Laplace's equation:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0$$

Its solution can be drawn as a **flow net** (Figure 1.1), made of two families of curves:

- **Flow lines**, the paths followed by the water;
- **Equipotential lines**, lines joining points of equal total head.

The two families cross at right angles and, in a correctly drawn net, form curvilinear "squares". The space between two flow lines is a **flow channel** and the head lost between two equipotentials is an **equipotential drop**. If the net has N_f flow channels and N_d equipotential drops, and the total head loss is H, the seepage per unit length of structure is:

$$q = k\,H\,\frac{N_f}{N_d}$$

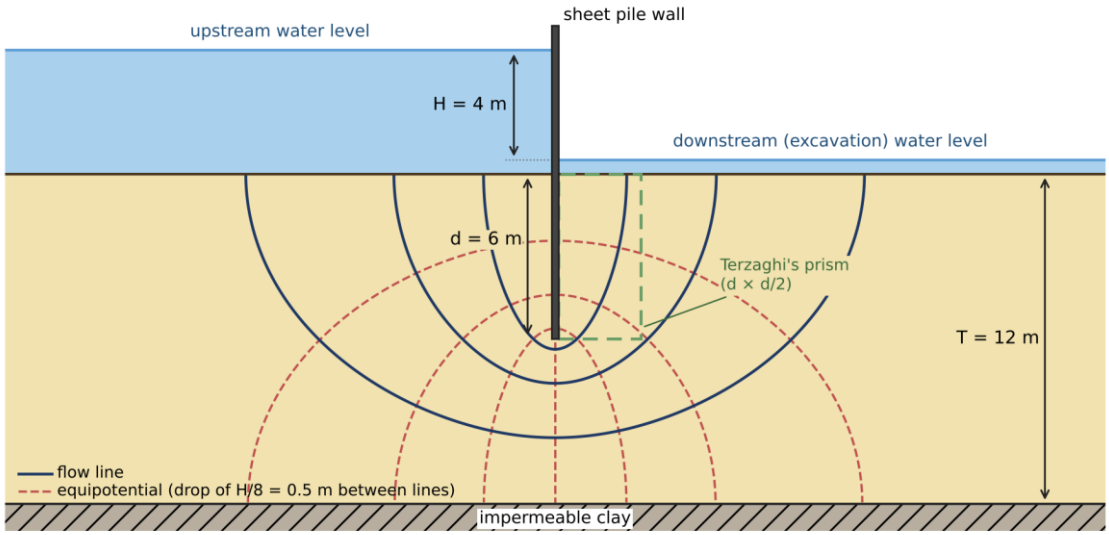


Figure 1.1. Flow net for seepage beneath a sheet pile wall driven to half the depth of a permeable sand layer. The net was computed numerically; it has about 4 flow channels and 8 equipotential drops.

Piping and heave. As water flows upwards towards the ground surface on the downstream side, it exerts an upward seepage force on the soil grains. When the upward hydraulic gradient reaches the **critical hydraulic gradient** (i_c), the effective stress falls to zero and the sand "boils":

$$i_c = \frac{\gamma'}{\gamma_w} = \frac{G_s - 1}{1 + e}$$

For most sands i_c is close to 1. Once boiling starts at the exit point, erosion can work back beneath the structure, forming a pipe; this is **piping**, the mechanism that destroyed the Teton Dam. The factor of safety against piping is the ratio i_c / i_{exit}, where i_{exit} is the gradient in the last square of the flow net at the exit; a factor of safety of 3 to 4 is commonly required. Terzaghi also checked the uplift of a prism of soil of depth d and width $d/2$ beside a sheet pile (**heave**), with a factor of safety:

$$F = \frac{\gamma' \, d}{\gamma_w \, h_a}$$

where h_a is the average excess head at the base of the prism.

Worked example. In Figure 1.1, a sheet pile wall penetrates $d = 6$ m into a 12 m thick sand layer with $k = 1 \times 10^{-5}$ m/s, $G_s = 2.65$, $e = 0.65$ and $\gamma_{sat} = 19.5$ kN/m^3. The difference in water level across the wall is $H = 4$ m.

$$q = 1 \times 10^{-5} \times 4 \times \frac{4}{8} = 2.0 \times 10^{-5} \text{ m}^3/\text{s per metre} \approx 1.7 \text{ m}^3 \text{ per day per metre}$$

The last equipotential drop, $4/8 = 0.5$ m, is lost over about 2.45 m beside the wall on the downstream side, so $i_{exit} = 0.5 / 2.45 = 0.20$. The critical gradient is:

$$i_c = \frac{2.65 - 1}{1 + 0.65} = 1.0$$

The factor of safety against piping is $1.0 / 0.20 = 5.0$. The numerical solution gives an average excess head of 1.34 m at the base of Terzaghi's prism, so the factor of safety against heave is:

$$F = \frac{(19.5 - 9.81) \times 6}{9.81 \times 1.34} = 4.4$$

Both are satisfactory. If the wall were driven less deeply, the flow path would be shorter, the exit gradient would rise and the factors of safety would fall.

1.3 Stress increase beneath loads

A foundation or embankment increases the stresses in the ground beneath it. The increase spreads out and decreases with depth, and settlement depends on how large it is in each compressible layer.

Point load. Boussinesq (1885) derived the vertical stress increase at depth z and horizontal distance r from a point load Q on the surface of an elastic half-space:

$$\Delta\sigma_z = \frac{3Q}{2\pi z^2}\left[\frac{1}{1 + (r/z)^2}\right]^{5/2}$$

Integrating this for loaded areas gives charts and tables for strip, rectangular and circular foundations. Lines of equal stress increase beneath a foundation form a **pressure bulb** (Figure 1.2a). Beneath a strip footing, the stress increase falls to about 20% of the applied pressure at a depth of about three times the width; beneath a square footing it falls to the same value at about 1.5 times the width. This is why ground investigation must reach well below foundation level, and deeper for wide structures such as rafts and embankments.

The 2:1 method. A simple approximation assumes that the load spreads at a slope of 2 vertical to 1 horizontal (Figure 1.2b). For a rectangular foundation of width B and length L carrying a pressure q, the average stress increase at depth z is:

$$\Delta\sigma_z = \frac{q\,B\,L}{(B + z)(L + z)}$$

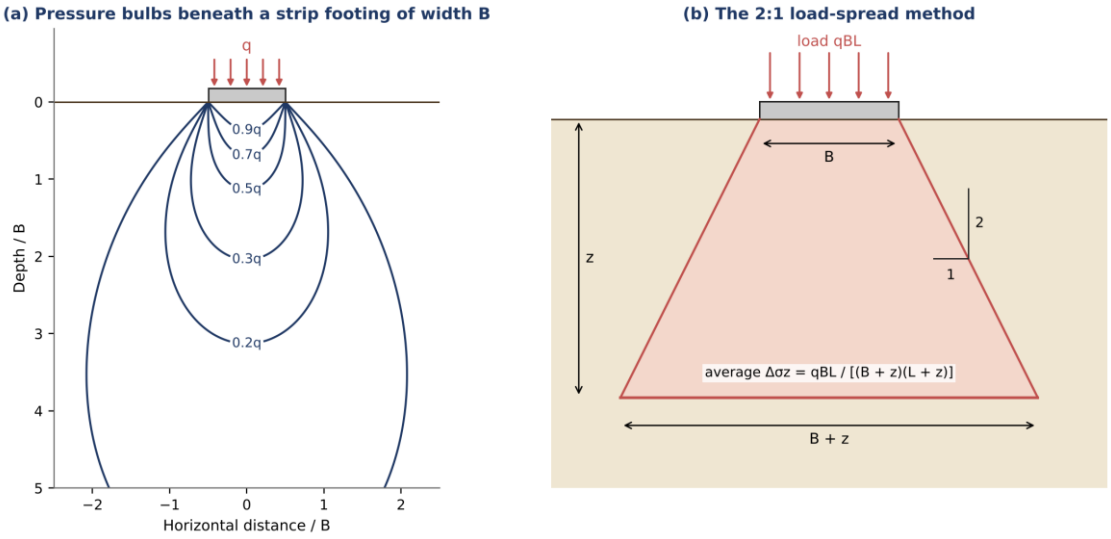


Figure 1.2. Stress increase beneath foundations. (a) Pressure bulbs beneath a strip footing, showing the stress increase as a fraction of the applied pressure q. (b) The 2:1 load-spread approximation.

Worked example. (a) A column load of 500 kN acts on the ground surface. Using Boussinesq's equation, the stress increase at 3 m depth directly beneath it is:

$$\Delta\sigma_z = \frac{3 \times 500}{2\pi \times 3^2} = 26.5 \text{ kPa}$$

At the same depth but 2 m to the side, the bracketed term is $[1/(1+0.444)]^{2.5} = 0.40$, so $\Delta\sigma_z = 10.6$ kPa. (b) A 2 m × 2 m footing carries 150 kPa. By the 2:1 method, the average stress increase at 3 m below the footing is:

$$\Delta\sigma_z = \frac{150 \times 2 \times 2}{(2+3)(2+3)} = 24 \text{ kPa}$$

1.4 Compressibility and consolidation

When a load is applied to a saturated clay, the water cannot escape quickly because the permeability is very low. The load is first carried by an increase in pore water pressure, the **excess pore pressure**. As water slowly drains out, the excess pore pressure dissipates, the effective stress increases and the clay compresses. This time-dependent process is **consolidation**, and it explains why buildings and embankments on the soft clays of the Niger Delta continue to settle for years.

The oedometer test. A disc of undisturbed clay, typically 75 mm in diameter and 20 mm thick, is confined in a steel ring between porous stones and loaded in stages, each usually doubling the previous load and left for 24 hours (Figure 1.3a). The void ratio at the end of each stage is plotted against the effective stress on a logarithmic scale (Figure 1.3b).

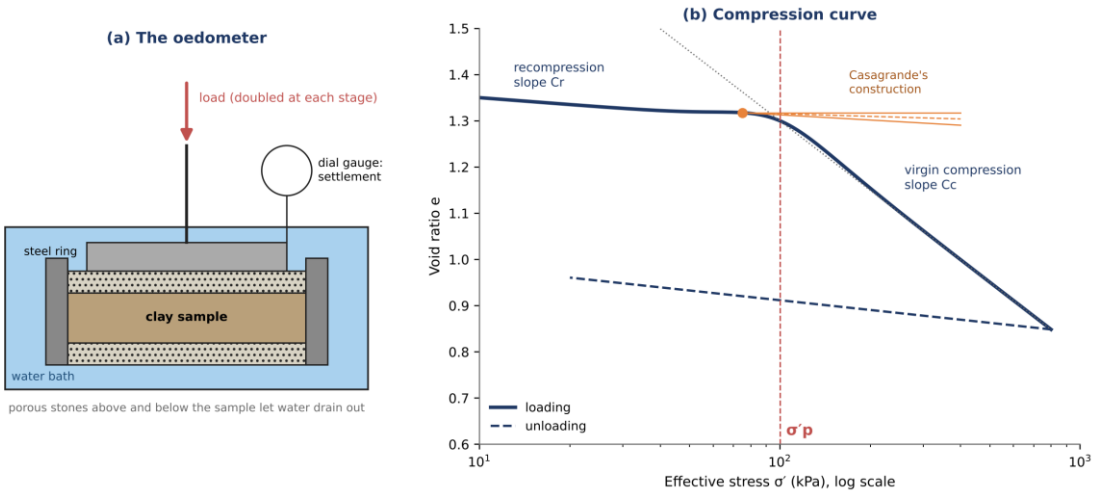


Figure 1.3. (a) The oedometer. (b) Results plotted as void ratio against the logarithm of effective stress, showing the recompression and virgin compression lines, the preconsolidation pressure found by Casagrande's construction, and the compression and recompression indices.

The curve has two parts. At low stresses, the clay is being reloaded to stresses it has carried before and is fairly stiff; at higher stresses, it follows the steeper **virgin compression line**. The stress at the change in slope is the **preconsolidation pressure** (σ'_p), the greatest effective stress the clay has ever carried. It is found graphically by Casagrande's (1936) construction. The slopes of the two lines are the **compression index** (C_c) and the **recompression index** (C_r, often about $C_c/5$ to $C_c/10$):

$$C_c = \frac{\Delta e}{\Delta \log_{10} \sigma'}$$

A clay whose present effective stress (σ'_0) equals σ'_p is **normally consolidated**; one that has been unloaded, by erosion for example, is **overconsolidated**. The **overconsolidation ratio** is:

$$\text{OCR} = \frac{\sigma'_p}{\sigma'_0}$$

Most young clays of the Niger Delta swamps are normally consolidated or only lightly overconsolidated, apart from a thin, stiffer crust near the surface that has dried out. Where oedometer results are not available, C_c can be estimated from the liquid limit (Skempton, 1944):

$$C_c \approx 0.009\,(\mathrm{LL} - 10)$$

Consolidation settlement. For a normally consolidated clay layer of thickness H and initial void ratio e_0, whose effective stress at mid-depth increases from σ'_0 to $\sigma'_0 + \Delta\sigma'$, the primary consolidation settlement is:

$$S_c = \frac{C_c\, H}{1 + e_0} \log_{10} \frac{\sigma'_0 + \Delta\sigma'}{\sigma'_0}$$

For an overconsolidated clay that stays below σ'_p, C_r replaces C_c. If the final stress exceeds σ'_p, the settlement is calculated in two parts: with C_r from σ'_0 to σ'_p, and with C_c from σ'_p to the final stress. Thick layers should be divided into sublayers and the settlements added.

Rate of consolidation. Terzaghi's theory of one-dimensional consolidation relates the progress of consolidation to a dimensionless **time factor** (T_v):

$$T_v = \frac{c_v\, t}{H_{dr}^2}$$

where c_v is the **coefficient of consolidation** (from the oedometer test), t is time and H_{dr} is the length of the longest drainage path: half the layer thickness if water can drain from both top and bottom (double drainage), or the full thickness if it can drain only one way. The **average degree of consolidation** (U) is the settlement at time t as a fraction of the final settlement. It depends only on T_v (Figure 1.4), and is closely approximated by:

$$T_v = \frac{\pi}{4} U^2 \quad (U < 0.6) \qquad T_v = 1.781 - 0.933 \log_{10}(100 - U\%) \quad (U > 60\%)$$

Useful values are $T_v = 0.197$ for 50% consolidation and $T_v = 0.848$ for 90%.

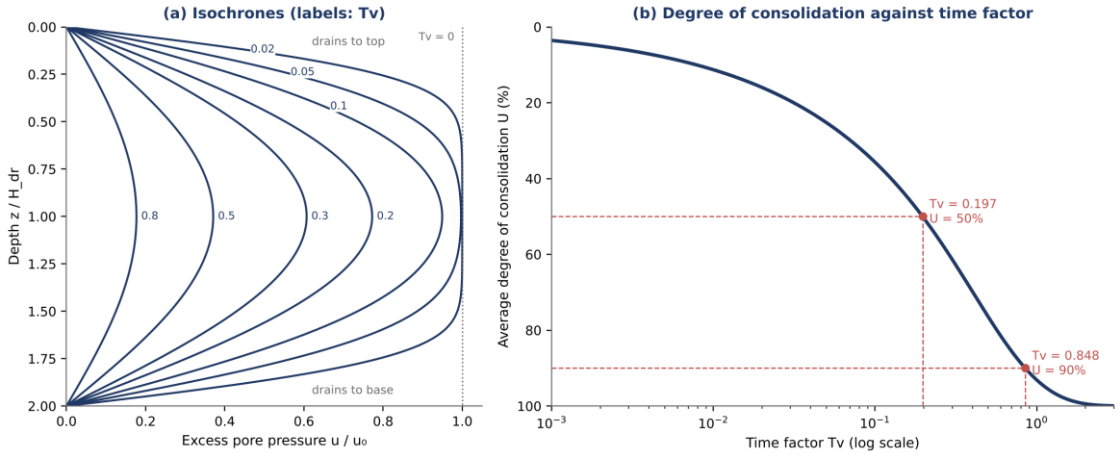


Figure 1.4. Terzaghi's theory of consolidation. (a) Isochrones showing the dissipation of excess pore pressure with time in a layer drained at top and bottom. (b) The average degree of consolidation against the time factor.

Because t is proportional to H_{dr}^2, halving the drainage path reduces the consolidation time to a quarter. This is the principle behind vertical drains, which are covered with ground improvement in the other part of the course.

Secondary compression. After the excess pore pressure has dissipated, clays and especially organic soils and peats continue to compress slowly by creep. This **secondary compression** is estimated from the coefficient of secondary compression (C_α):

$$S_s = \frac{C_\alpha \, H}{1 + e_p} \log_{10} \frac{t_2}{t_1}$$

where e_p is the void ratio at the end of primary consolidation and t_1 and t_2 are times after its end. In Niger Delta peats, secondary compression can be as large as primary consolidation.

Worked example. To raise a site above flood level, 2.0 m of sand fill ($\gamma = 19$ kN/m^3) is placed over a wide area of swamp. The ground consists of 6.0 m of normally consolidated soft clay ($\gamma_{sat} = 16$ kN/m^3, $e_0 = 1.80$, LL = 77%) over dense sand, with the water table at the ground surface. The coefficient of consolidation is 1.5 m^2/year. Estimate the consolidation settlement and the times for 50% and 90% consolidation.

The compression index from the liquid limit is $C_c = 0.009 \times (77 - 10) = 0.60$. At mid-depth of the clay (3.0 m):

$$\sigma'_0 = (16 - 9.81) \times 3.0 = 18.6 \text{ kPa} \qquad \Delta\sigma' = 2.0 \times 19 = 38 \text{ kPa}$$

$$S_c = \frac{0.60 \times 6.0}{1 + 1.80} \log_{10} \frac{18.6 + 38}{18.6} = 1.286 \times 0.483 = 0.62 \text{ m}$$

The clay drains to the fill above and the sand below, so $H_{dr} = 3.0$ m:

$$t_{50} = \frac{0.197 \times 3.0^2}{1.5} = 1.2 \text{ years} \qquad t_{90} = \frac{0.848 \times 3.0^2}{1.5} = 5.1 \text{ years}$$

The fill will settle by about 0.6 m, and it will take about five years for 90% of this to occur. Any building placed on the fill too soon will settle and crack, and pipes and roads connecting it to firmer ground will be damaged.

1.5 Shear strength

Soils fail in shear. The shear strength on a plane depends on the effective normal stress on that plane through the **Mohr–Coulomb** criterion:

$$\tau_f = c' + \sigma'_n \tan\phi'$$

where c' is the effective cohesion and ϕ' the effective angle of shearing resistance. The state of stress at a point is represented by a **Mohr circle**, drawn between the minor and major principal stresses σ'_3 and σ'_1. Failure occurs when the circle touches the failure envelope (Figure 1.5a). This gives the relationship between the principal stresses at failure:

$$\sigma'_1 = \sigma'_3 \tan^2\left(45° + \frac{\phi'}{2}\right) + 2c' \tan\left(45° + \frac{\phi'}{2}\right)$$

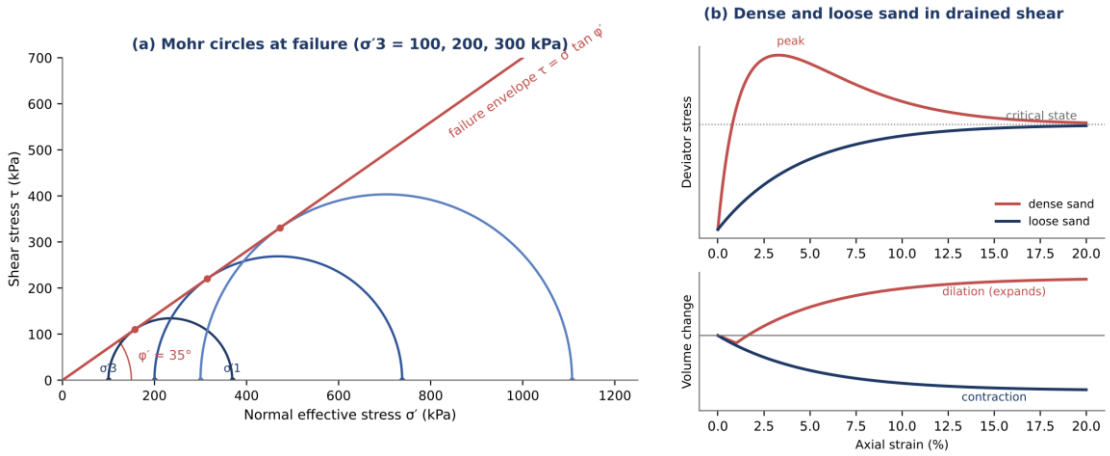


Figure 1.5. (a) Mohr circles at failure from three drained triaxial tests and the Mohr–Coulomb failure envelope. (b) Stress–strain and volume change behaviour of dense and loose sand in drained shear.

Laboratory tests.

- **Shear box (direct shear):** the sample is sheared along a horizontal plane under a constant normal stress. It is simple and well suited to sands and to measuring the strength along known planes of weakness.
- **Triaxial test:** a cylindrical sample in a rubber membrane is subjected to a cell pressure (σ_3) and then loaded axially to failure. Drainage and pore pressure are controlled. In the **unconsolidated undrained (UU)** test, no drainage is allowed, giving the undrained shear strength c_u. In the **consolidated undrained (CU)** test with pore pressure measurement, and the **consolidated drained (CD)** test, the effective strength parameters c' and ϕ' are obtained.
- **Unconfined compression test:** a UU test with zero cell pressure; for a saturated clay, c_u is half the unconfined compressive strength ($c_u = q_u/2$).
- **Vane test:** used in the field and laboratory for soft clays (MGY 321).

Drained and undrained behaviour. Sands drain quickly, so their strength is analysed in terms of effective stress with $c' = 0$ and ϕ' typically from 28° (loose) to 40° or more (dense). Clays drain slowly. Immediately after loading, as under a quickly built embankment, a saturated clay behaves undrained and its strength is the undrained shear strength c_u, independent of the total stress (the "$\phi_u = 0$" condition). In the long term, after excess pore pressures have dissipated, effective stress parameters apply. Both cases must be checked: for foundations on soft clay the short term usually governs, whereas for excavations and cut slopes in clay the long term often governs, because unloading causes the clay to soften as it takes in water.

Dense and loose sands. When sheared, a dense sand first expands (**dilates**) as the grains ride over each other, and reaches a peak strength before falling to a constant **critical state** strength (Figure 1.5b). A loose sand contracts and gains strength steadily to the same critical state. If a loose saturated sand is sheared quickly or shaken, its tendency to contract raises the pore pressure, which can reduce the effective stress to zero. This is **liquefaction**, a hazard for loose fills and the young sands of the delta under earthquake or wave loading.

Worked example. In a drained triaxial test on a dense sand ($c' = 0$), the sample fails when $\sigma'_3 = 100$ kPa and $\sigma'_1 = 370$ kPa. From the Mohr circle:

$$\sin\phi' = \frac{\sigma'_1 - \sigma'_3}{\sigma'_1 + \sigma'_3} = \frac{270}{470} = 0.574 \qquad \phi' = 35.1°$$

1.6 Lateral earth pressure

Retaining walls, basement walls, sheet piles, bridge abutments and quay walls must resist the horizontal pressure of the soil behind them. The pressure depends on how the wall moves (Figure 1.6):

- If the wall does not move, the soil is in the **at-rest** state, with K_0 (Section 1.1).
- If the wall moves slightly away from the soil, the pressure falls to a minimum, the **active** pressure, as a wedge of soil begins to slide down.
- If the wall is pushed into the soil, the pressure rises to a maximum, the **passive** pressure, as a wedge is pushed up.

For a smooth vertical wall retaining a horizontal surface of cohesionless soil, Rankine's theory gives the coefficients of active and passive earth pressure:

$$K_a = \frac{1 - \sin\phi'}{1 + \sin\phi'} = \tan^2\left(45° - \frac{\phi'}{2}\right) \qquad K_p = \tan^2\left(45° + \frac{\phi'}{2}\right)$$

The active pressure at depth z is $\sigma'_a = K_a \gamma z$, increasing linearly with depth, and the total active thrust on a wall of height H acts at $H/3$ above the base:

$$P_a = 1/2\, K_a\, \gamma\, H^2$$

Below the water table, the effective unit weight γ' is used for the soil pressure, and the **water pressure must be added separately**, because water exerts the same pressure in all directions ($K = 1$). Water pressure often exceeds the soil pressure, which is why quay walls and basement walls need drains or must be designed for the full water pressure, including the difference between high and low tide.

For a cohesive soil, cohesion reduces the active pressure: $\sigma'_a = K_a \gamma z - 2c'\sqrt{K_a}$. Near the surface this gives a theoretical tension, which in practice opens a crack to depth $z_c = 2c'/\left(\gamma\sqrt{K_a}\right)$ that can fill with water.

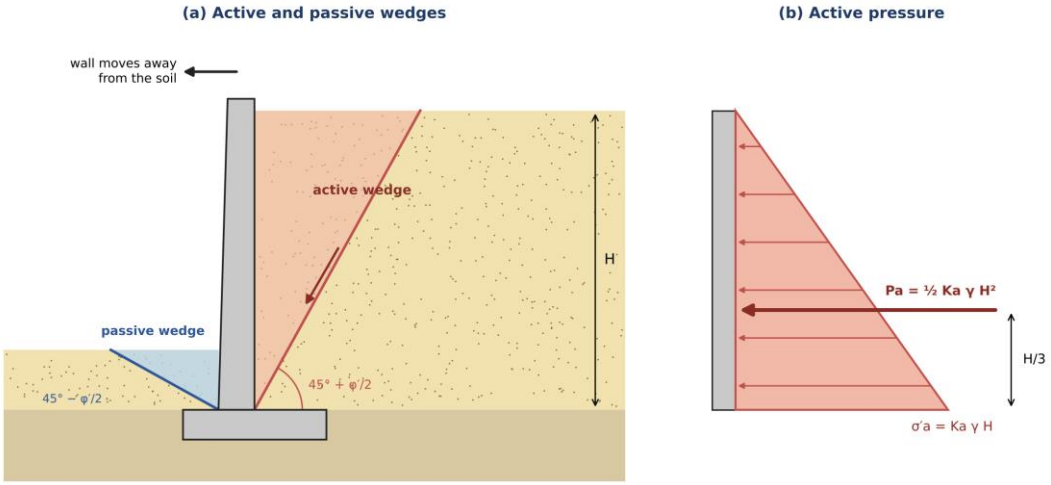


Figure 1.6. Earth pressure on a retaining wall. The active wedge behind the wall slides on a plane at 45° + φ′/2; the passive wedge in front of the toe on a plane at 45° − φ′/2. The active pressure increases linearly with depth, and its resultant acts at H/3.

Worked example. A 5.0 m high wall retains dry sand with $\phi' = 32°$ and $\gamma = 18\ \text{kN/m}^3$.

$$K_a = \tan^2(45° - 16°) = 0.307 \qquad K_p = \tan^2(45° + 16°) = 3.25$$

The active pressure at the base is $0.307 \times 18 \times 5.0 = 27.7$ kPa, and the thrust is:

$$P_a = 1/2 \times 0.307 \times 18 \times 5.0^2 = 69.1 \text{ kN per metre of wall}$$

acting 1.67 m above the base. The passive coefficient is more than ten times the active one, which is why a modest embedment in front of a wall provides a large resisting force.

1.7 Slope stability

MGY 201 introduced the factor of safety (F) of a slope as the ratio of resisting to driving forces. For engineering design, a factor of safety of about 1.3 to 1.5 is usually required for permanent slopes.

Infinite slope with seepage. For a long slope in cohesionless soil with the water table at the surface and seepage parallel to the slope, as in many riverbanks and creek banks of the delta at low tide:

$$F = \frac{\gamma'}{\gamma_{sat}} \cdot \frac{\tan\phi'}{\tan\beta}$$

Because γ'/γ_{sat} is roughly one half, seepage halves the factor of safety of a dry slope.

Worked example. A sandy bank slopes at $\beta = 15°$, with $\phi' = 30°$ and $\gamma_{sat} = 18$ kN/m^3. Dry, $F = \tan 30°/\tan 15° = 2.15$. With seepage parallel to the slope:

$$F = \frac{18 - 9.81}{18} \times \frac{0.577}{0.268} = 0.455 \times 2.15 = 0.98$$

The bank is stable when dry but fails when water seeps out of it after the tide falls or the river level drops quickly. This "rapid drawdown" condition explains many bank failures along the Niger and the creeks of the delta.

Method of slices. Slopes in clay and in layered ground usually fail on curved slip surfaces. The soil above a trial circular surface is divided into vertical slices (Figure 1.7). For each slice of weight W, base length l, base inclination α and pore pressure u on the base, the simplest (Fellenius, or ordinary) method gives:

$$F = \frac{\sum [c' l + (W\cos\alpha - u\, l)\tan\phi']}{\sum W \sin\alpha}$$

Bishop's simplified method (Bishop, 1955) allows for the forces between slices and is more accurate; it is solved by iteration and is the standard hand and computer method. Many trial surfaces are analysed, and the one with the lowest factor of safety is the critical slip surface. For short-term analysis of soft clay, c' is replaced by c_u and ϕ' by zero.

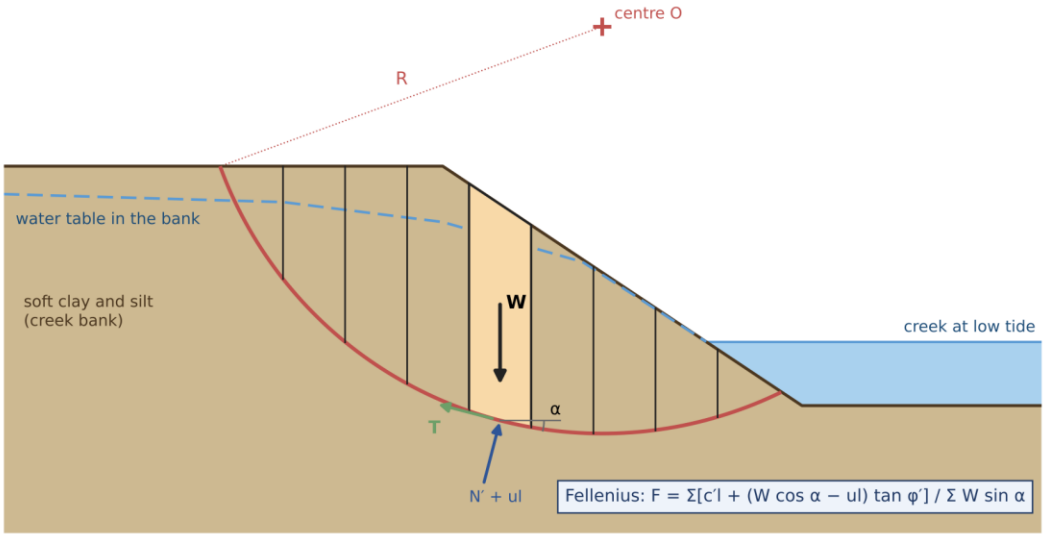


Figure 1.7. The method of slices for a circular slip surface through a creek bank. The forces on a typical slice are shown: its weight W, the normal and shear forces on its base, and the pore water pressure u.

1.8 Key points of Section 1

- Flow nets give seepage quantities and exit gradients; piping and heave must be checked where water flows upwards out of the ground.
- Stress beneath a foundation decreases with depth; the pressure bulb shows how deep the ground is affected.
- Consolidation settlement depends on C_c, σ'_p and the stress increase; its rate depends on c_v and the square of the drainage path.
- Shear strength follows the Mohr–Coulomb criterion; clays must be checked for undrained (short-term) and drained (long-term) conditions.
- Rankine's theory gives active and passive earth pressures; water pressure must be added separately.
- Seepage roughly halves the stability of a sandy slope; curved slip surfaces are analysed by the method of slices.

Section 2: Foundation engineering

A **foundation** transfers the loads of a structure safely to the ground. It must satisfy two requirements:

1. **Adequate safety against shear failure of the ground** (bearing capacity failure), with a suitable factor of safety, commonly 3 for shallow foundations and 2 to 3 for piles.
2. **Acceptable settlement**: both the total settlement and, more importantly, the differential settlement between parts of the structure must be small enough to avoid damage or loss of function.

In addition, the foundation must be deep enough to avoid seasonal swelling and shrinkage, erosion and scour, and must resist chemical attack, for example by sulphates in clays or by brackish water in the delta.

2.1 Types of foundation

Foundations are divided into shallow and deep (Figure 2.1).

Shallow foundations transfer load to the ground close to the surface, usually at a depth not greater than their width:

- **Pad (spread) footings** carry individual columns.
- **Strip footings** carry walls or rows of columns.
- **Raft (mat) foundations** cover the whole area of a building. They spread the load over a large area, reduce differential settlement and are used where the soil is weak or where pad footings would almost touch.

Deep foundations transfer load through weak upper layers to stronger ground at depth:

- **Piles** are long slender columns of concrete, steel or timber, driven or bored into the ground. They carry load by **end bearing** at the toe, by **shaft friction** along their sides, or by both.
- **Piers and caissons** are large-diameter deep foundations, used for bridge piers and heavy structures.

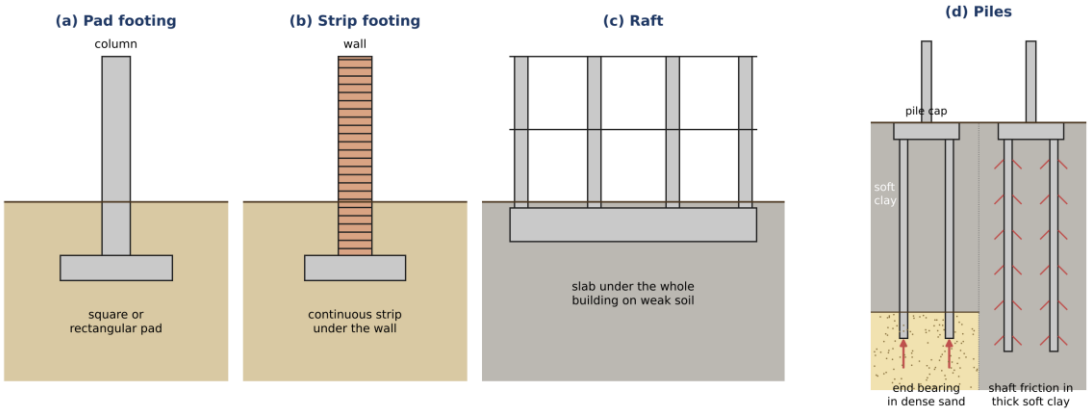


Figure 2.1. Types of foundation: pad and strip footings, a raft, and piles carrying load by end bearing on a strong stratum or by shaft friction in a thick soft layer.

The choice depends on the ground conditions, the loads, the tolerance of the structure to settlement, the presence of water, and cost. On the dense sands and laterites of much of Nigeria, pad and strip footings are usually adequate for buildings of a few storeys. On the soft clays and peats of the Niger Delta swamps, almost all significant structures are founded on piles, or on rafts placed on thick, well-compacted sand fill after the underlying clay has consolidated.

2.2 Modes of bearing capacity failure

As the load on a footing increases, the soil beneath it eventually fails in one of three ways (Figure 2.2):

- **General shear failure**: a well-defined slip surface develops from the edge of the footing to the ground surface, which bulges up beside the footing. The load reaches a clear peak. Typical of dense sands and stiff clays.
- **Local shear failure**: slip surfaces develop beneath the footing but do not reach the surface clearly; the footing settles considerably before failing. Typical of medium dense sands and firm clays.

- **Punching shear failure**: the footing punches vertically into the ground with little bulging, and there is no clear peak load. Typical of loose sands and soft clays, and of a firm layer over a soft one.

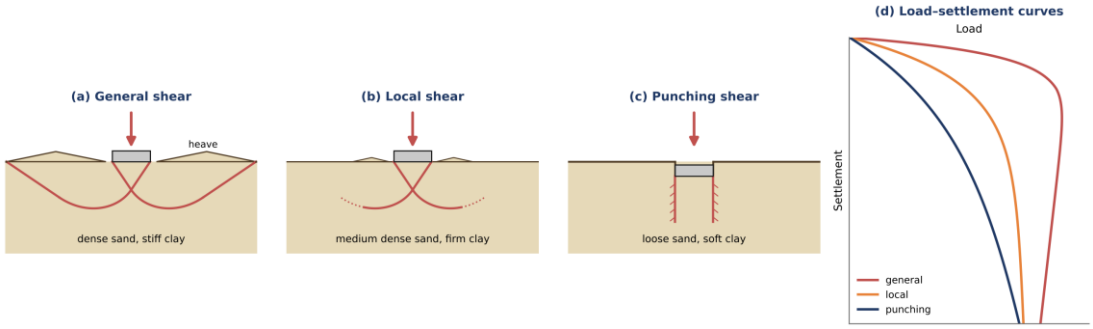


Figure 2.2. The three modes of bearing capacity failure, and the load–settlement curves associated with them.

2.3 Bearing capacity of shallow foundations

Terzaghi (1943) analysed general shear failure beneath a long strip footing. The soil beneath the footing forms a wedge that moves down with it (zone I), pushing the soil on either side outwards along curved slip surfaces (zone II) and up into passive wedges (zone III) (Figure 2.3).

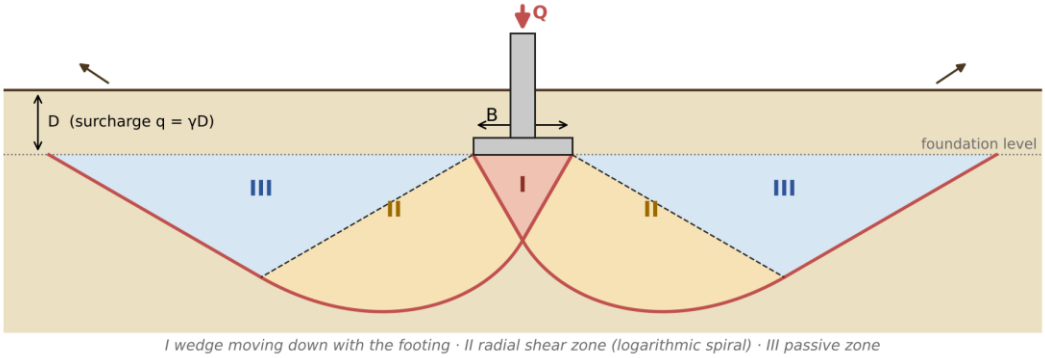


Figure 2.3. The failure mechanism beneath a strip footing: a wedge beneath the footing (I), radial shear zones bounded by logarithmic spirals (II), and passive Rankine zones (III). The soil above foundation level acts as a surcharge q = γD.

The **ultimate bearing capacity** (q_u) of a strip footing of width B at depth D is given by the general bearing capacity equation:

$$q_u = c' N_c + q N_q + 1/2 \; \gamma B N_\gamma$$

The three terms represent the contributions of the soil's cohesion, of the surcharge $q = \gamma D$ of soil above foundation level, and of the weight of the soil beneath the footing. The **bearing capacity factors** depend only on ϕ'. The values used here are:

$$N_q = e^{\pi \tan\phi'} \tan^2\left(45° + \frac{\phi'}{2}\right) \qquad N_c = (N_q - 1)\cot\phi' \qquad N_\gamma = 2\,(N_q + 1)\tan\phi'$$

The expression for N_γ is due to Vesić (1973); other authors give somewhat different values for this factor.

ϕ'	N_c	N_q	N_γ
0°	5.14	1.00	0.00

ϕ'	N_c	N_q	N_γ
10°	8.34	2.47	1.22
20°	14.83	6.40	5.39
25°	20.72	10.66	10.88
28°	25.80	14.72	16.72
30°	30.14	18.40	22.40
32°	35.49	23.18	30.21
34°	42.16	29.44	41.06
36°	50.59	37.75	56.31
38°	61.35	48.93	78.02
40°	75.31	64.20	109.41

The factors rise very steeply with ϕ', so a small error in the angle of shearing resistance leads to a large error in bearing capacity. This is one reason why a factor of safety of 3 is used.

Square and circular footings. For a square footing, Terzaghi multiplied the cohesion term by 1.3 and replaced the $1/2$ in the last term by 0.4; for a circular footing of diameter B, the factors are 1.3 and 0.3.

Undrained bearing capacity of clay. For a saturated clay loaded quickly, $\phi_u = 0$, so $N_q = 1$, $N_\gamma = 0$ and $N_c = 5.14$. The net ultimate bearing capacity of a strip footing is then $5.14\,c_u$. For square and circular footings, Skempton's factor of about 1.2 is applied, giving about $6.2\,c_u$:

$$q_{u,net} = 5.14\,c_u \quad \text{(strip)} \qquad q_{u,net} \approx 6.2\,c_u \quad \text{(square or circular)}$$

Effect of the water table. Below the water table, the effective unit weight γ' (about half of γ_{sat}) must be used. If the water table is at or above foundation level, γ' is used in the N_γ term, and if it is at the ground surface, also in the surcharge q. A rise of the water table to foundation level can reduce the bearing capacity of a footing on sand by a quarter or more, which matters in the delta where the water table rises close to the surface in the wet season.

Net and allowable bearing pressure. The **net ultimate bearing capacity** is the increase in pressure that causes failure, $q_{u,net} = q_u - \gamma D$. The **net allowable bearing pressure** is found by dividing by the factor of safety F, usually 3:

$$q_{a,net} = \frac{q_{u,net}}{F}$$

Worked example 1: strip footing on sand. A strip footing 1.5 m wide is placed at a depth of 1.0 m in a sand with $\phi' = 32°$, $c' = 0$ and $\gamma = 18$ kN/m^3. The water table is deep. For $\phi' = 32°$, $N_q = 23.18$ and $N_\gamma = 30.21$.

$$q_u = (18 \times 1.0) \times 23.18 + 1/2 \times 18 \times 1.5 \times 30.21 = 417 + 408 = 825 \text{ kPa}$$

$$q_{u,net} = 825 - 18 = 807 \text{ kPa} \qquad q_{a,net} = \frac{807}{3} = 269 \text{ kPa}$$

If the water table rose to foundation level, the second term would become $1/2 \times 8.19 \times 1.5 \times 30.21 = 186$ kPa, and q_u would fall to 603 kPa, a reduction of 27%.

Worked example 2: square footing on clay. A 2.0 m square footing at 1.0 m depth rests on a firm clay with $c_u = 40$ kPa:

$$q_{u,net} = 6.2 \times 40 \approx 1.2 \times 5.14 \times 40 = 247 \text{ kPa} \qquad q_{a,net} = \frac{247}{3} = 82 \text{ kPa}$$

On a typical Niger Delta swamp clay with $c_u = 15$ kPa, the net allowable bearing pressure would be only about 31 kPa, too low for anything but the lightest structures, and the settlement would be large. This is why piles are normally used.

2.4 Settlement of shallow foundations

The settlement of a foundation has three components:

- **Immediate (elastic) settlement**, which occurs as the load is applied, from distortion of the soil without change in volume (in saturated clays) or from compression of the soil skeleton (in sands);
- **Primary consolidation settlement** in clays, which develops over months or years (Section 1.4);
- **Secondary compression**, by creep, especially in organic clays and peats.

Allowable settlement. Structures are damaged mainly by differential settlement, which distorts the frame and cracks walls. Classic guidance (Skempton and MacDonald, 1956; Bjerrum, 1963) is:

Criterion	Sand	Clay
Maximum settlement, isolated footings	25 to 40 mm	65 mm
Maximum settlement, rafts	40 to 65 mm	65 to 100 mm
Maximum differential settlement	25 mm	40 mm
Angular distortion: safe limit for buildings where cracking is not acceptable	1/500	1/500
Angular distortion: cracking of panel walls expected	1/300	1/300
Angular distortion: structural damage expected	1/150	1/150

The angular distortion is the differential settlement between two points divided by the distance between them.

Immediate settlement from elastic theory. The settlement of a footing of width B carrying a pressure q on soil with Young's modulus E_s and Poisson's ratio ν is:

$$S_i = \frac{q\,B\,(1-\nu^2)}{E_s}\,I_s$$

where I_s is an influence factor that depends on the shape and rigidity of the footing: for a square footing, about 1.12 at the centre and 0.56 at the corner of a flexible footing, and about 0.82 for a rigid footing. For example, a rigid 2.0 m square footing carrying 150 kPa on a clayey sand with $E_s = 15$ MPa and $\nu = 0.3$ settles:

$$S_i = \frac{150 \times 2.0 \times (1-0.09)}{15\,000} \times 0.82 = 0.015 \text{ m} = 15 \text{ mm}$$

Footings on sand from the SPT. Because undisturbed samples of sand cannot easily be taken, the settlement of footings on sand is usually estimated from in situ tests. A simple method

(Meyerhof, 1965) gives the net bearing pressure (kPa) that causes a settlement of about 25 mm, in terms of the corrected SPT value N_{60} (Section 3.2):

$$q_a \approx 12\,N_{60} \quad (B \le 1.2\text{ m}) \qquad q_a \approx 8\,N_{60}\left(\frac{B+0.3}{B}\right)^2 \quad (B > 1.2\text{ m})$$

with B in metres. Allowing for the depth of the footing increases these values slightly.

Worked example 3. The sand in worked example 1 has $N_{60} = 15$. For the 1.5 m wide strip footing:

$$q_a \approx 8 \times 15 \times \left(\frac{1.5+0.3}{1.5}\right)^2 = 120 \times 1.44 = 173\text{ kPa}$$

This is less than the 269 kPa allowed by bearing capacity, so **settlement governs** the design. The footing can safely carry a net load of about $1.5 \times 173 = 260$ kN per metre. On sands, settlement rather than bearing capacity usually controls the size of footings wider than about 1 m.

Footings and rafts on clay. The consolidation settlement is calculated from oedometer results and the stress increase beneath the foundation (Sections 1.3 and 1.4), dividing the clay into sublayers. Because the stress increase decreases with depth, a narrow footing affects only the upper part of a clay layer, whereas a wide raft or embankment stresses the full thickness and settles much more for the same bearing pressure.

2.5 Pile foundations

Piles are used where the upper soils are too weak or compressible, where loads are heavy or concentrated, where scour or erosion may remove the soil around a shallow foundation, where uplift or lateral loads must be resisted, and for structures in water such as jetties, bridges and offshore platforms.

Types of pile.

- **Driven (displacement) piles**: precast concrete, steel H-sections or steel tubes, driven by hammer or vibrator. Driving displaces and densifies the surrounding soil. Steel tubular piles are standard for jetties and offshore platforms.
- **Bored (replacement) piles**: a hole is drilled, often supported by casing or drilling fluid, and filled with reinforced concrete. They cause little noise or vibration, but loosen the soil slightly.
- **Driven cast-in-place piles**: a steel tube with a closed end is driven, reinforcement is placed and concrete is poured as the tube is withdrawn.

Axial capacity. The ultimate capacity of a single pile (Q_u) is the sum of the **base resistance** (Q_b) and the **shaft resistance** (Q_s) (Figure 2.4a):

$$Q_u = Q_b + Q_s = q_b\,A_b + \sum f_s\,A_s$$

where A_b is the area of the base, A_s the shaft area in each layer, q_b the unit base resistance and f_s the unit shaft friction. The allowable load is $Q_a = Q_u/F$, with F typically 2.5 when capacity is calculated, and as low as 2 when it is confirmed by load tests.

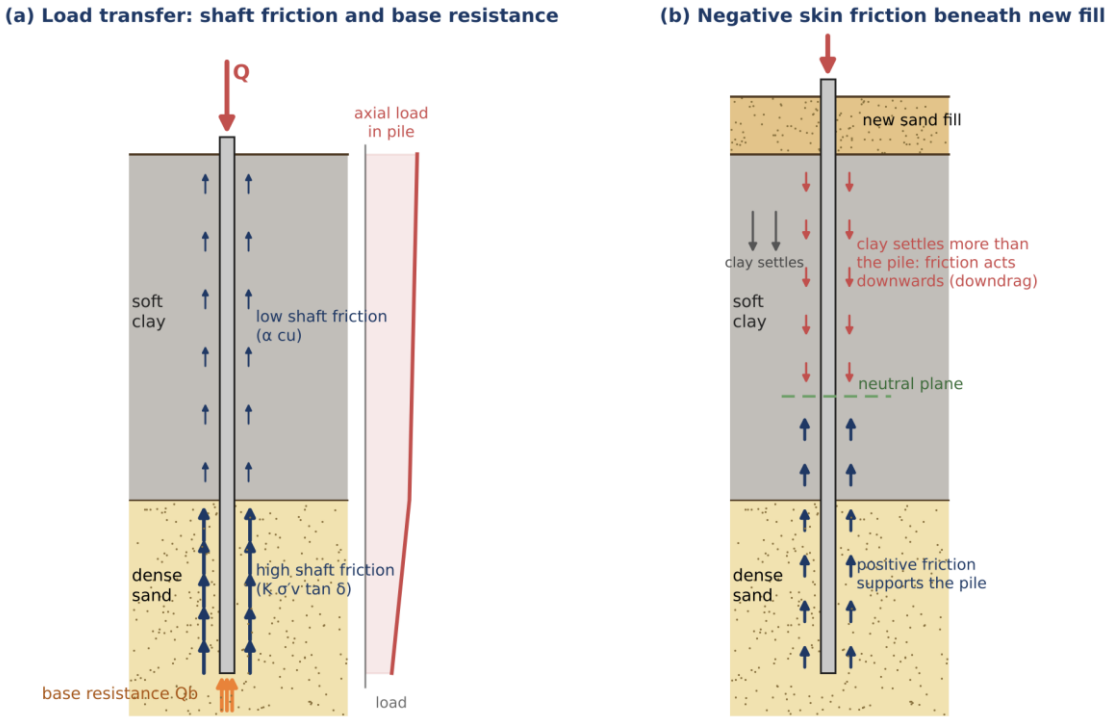


Figure 2.4. (a) Load transfer in a pile through soft clay into dense sand: shaft friction along the pile and base resistance at the toe. (b) Negative skin friction: fill placed on soft clay makes the clay settle more than the pile, so the clay drags the pile down.

In clay (undrained). The base resistance is $q_b = 9\,c_u$. The shaft friction is a fraction α of the undrained shear strength (the **α-method**): $f_s = \alpha\,c_u$. The American Petroleum Institute recommendations for offshore piles (API RP 2A, 2000), widely used in the delta, take:

$$\alpha = 0.5\,\psi^{-0.5} \quad (\psi \le 1) \qquad \alpha = 0.5\,\psi^{-0.25} \quad (\psi > 1), \qquad \alpha \le 1, \qquad \psi = \frac{c_u}{\sigma'_v}$$

so that α approaches 1 in soft, normally consolidated clays and falls to about 0.5 in stiff clays.

In sand (drained). The shaft friction is $f_s = K\,\sigma'_v \tan\delta$, where K is the coefficient of horizontal stress (taken as about 1.0 for displacement piles in API RP 2A) and δ is the angle of friction between pile and soil. The base resistance is $q_b = N_q\,\sigma'_v$ at the toe. Both are subject to limiting values. For dense sand, API RP 2A gives $\delta = 30°$, a limiting shaft friction of 95.7 kPa, $N_q = 40$ and a limiting base resistance of 9.6 MPa. More recent practice uses methods based directly on CPT results.

Worked example 4. A 0.4 m square precast concrete pile is driven through 12 m of soft clay ($c_u = 18$ kPa, $\gamma_{sat} = 16$ kN/m^3) and 6 m into dense sand ($\gamma_{sat} = 19.5$ kN/m^3). The water table is at the ground surface. Estimate the allowable load. The perimeter is 1.6 m and the base area 0.16 m^2.

Effective stresses. At the middle of the clay, $\sigma'_v = 6.19 \times 6 = 37.1$ kPa; at the top of the sand, $6.19 \times 12 = 74.3$ kPa; at the toe, $74.3 + 9.69 \times 6 = 132.4$ kPa; at the middle of the sand layer, 103.4 kPa.

Clay shaft. $\psi = 18/37.1 = 0.49$, so $\alpha = 0.5 \times 0.49^{-0.5} = 0.72$ and $f_s = 0.72 \times 18 = 12.9$ kPa:

$$Q_{s,clay} = 12.9 \times 1.6 \times 12 = 248 \text{ kN}$$

Sand shaft. $f_s = 1.0 \times 103.4 \times \tan 30° = 59.7$ kPa (below the limit of 95.7 kPa):

$$Q_{s,sand} = 59.7 \times 1.6 \times 6 = 573 \text{ kN}$$

Base. $q_b = 40 \times 132.4 = 5296$ kPa (below the limit of 9.6 MPa):

$$Q_b = 5296 \times 0.16 = 847 \text{ kN}$$

$$Q_u = 248 + 573 + 847 = 1668 \text{ kN} \qquad Q_a = \frac{1668}{2.5} = 667 \text{ kN}$$

Nearly 85% of the capacity comes from the 6 m of sand. Driving the pile a few metres deeper into the sand would increase the capacity considerably, whereas the soft clay contributes little.

Negative skin friction. If fill is placed on soft clay around piles, or if the water table is lowered, the clay consolidates and moves down relative to the pile. The friction on the upper part of the pile then acts downwards, adding load instead of supporting it (Figure 2.4b). This **downdrag** is estimated as $f_n = \beta\, \sigma'_v$, with β about 0.2 to 0.3 for soft clay. In worked example 4, if 2 m of sand fill (38 kPa) were placed around the pile, the average effective stress in the clay would rise to about 75 kPa, and with $\beta = 0.25$ the downdrag force would be about $0.25 \times 75 \times 1.6 \times 12 =$ 360 kN, more than half of the allowable load. Downdrag is one of the commonest causes of excessive settlement of piled structures in the delta.

Pile groups. Piles are normally used in groups joined by a pile cap. In clay, the capacity of a closely spaced group may be less than the sum of the individual capacities, because the group can fail as a block. The settlement of a group is larger than that of a single pile, because the stresses from all the piles overlap and reach deeper soil.

Testing and installation control. Calculated capacities are uncertain, so piles are tested. In a **static load test**, a pile is loaded in stages, usually to 1.5 to 2 times its working load, and its settlement is measured. In **dynamic testing**, gauges on the pile head record the stress wave from a hammer blow, and the record is analysed to estimate the capacity. During driving, the set (penetration per blow) and the hammer energy are recorded for every pile.

Lateral loads. Jetty and offshore piles must also resist large horizontal loads from waves, currents, berthing ships and wind. Their design uses the stiffness and strength of the upper few pile diameters of soil, which in the delta are often the weakest soils of the profile and may be removed by scour.

2.6 Foundations in the Niger Delta and the marine environment

Buildings on soft ground. Two main solutions are used. In the first, piles are driven or bored through the soft clay to the dense sands below, typically 15 to 30 m deep. In the second, the site is raised with dredged or imported sand fill, the clay is allowed to consolidate (often speeded up by vertical drains or preloading), and the building is placed on a raft or on footings in the fill. The second is cheaper but slower and depends on good settlement monitoring.

Jetties and quay walls. Jetties in the creeks and rivers of the delta are usually built on steel tubular or precast concrete piles driven from barges. Their design must allow for scour around the piles, the soft clay near the bed, corrosion of steel in brackish water, the tidal range and lateral loads from berthing vessels. Quay walls of steel sheet piles retain the land behind them and must resist the active earth pressure and the difference in water level between high and low tide (Section 1.6).

Offshore foundations. Fixed oil and gas platforms off the Nigerian coast stand on steel jacket structures pinned to the seabed by long, open-ended steel tubular piles, often more than 1.5 m in diameter and many tens of metres long, designed by the API methods of Section 2.5. Suction

caissons, large upturned steel cans sucked into the seabed, anchor floating structures. Mobile jack-up drilling rigs stand on large conical footings called spudcans; a dangerous situation arises where a layer of sand overlies soft clay, because the spudcan can suddenly **punch through** the sand into the clay, tilting the rig.

2.7 Key points of Section 2

- Foundations must have an adequate factor of safety against bearing failure and acceptable total and differential settlement.
- Shallow foundations include pads, strips and rafts; deep foundations include piles, piers and caissons.
- The general bearing capacity equation combines cohesion, surcharge and self-weight terms; the factors rise steeply with ϕ'.
- On sand, settlement usually governs the size of footings; on soft clay, bearing capacity and consolidation both restrict shallow foundations.
- Pile capacity is the sum of shaft and base resistance; negative skin friction adds load where soft clay settles around piles.
- In the Niger Delta, piles or rafts on consolidated sand fill are the usual solutions; marine structures must also allow for scour, corrosion, tides and lateral loads.

Section 3: Site and applied practice

The calculations of Sections 1 and 2 are only as good as the ground model and parameters they use. This section covers how a ground investigation is planned for a foundation, how in situ and laboratory tests are interpreted, how design parameters and factors of safety are chosen, how the work is reported and monitored, and how these principles are applied to typical projects in the Niger Delta.

3.1 Planning a ground investigation for foundations

A ground investigation should be planned around the questions the design must answer: what the soil layers are, how thick and how variable they are, where the groundwater is, and what their strength, compressibility and permeability are. It usually proceeds in stages: a desk study and walkover (MGY 321), a preliminary investigation to establish the general conditions, a main investigation for design, and, if needed, supplementary work during construction.

Number and spacing of exploration points. Points should cover the whole footprint of the structure and its critical parts, such as heavily loaded columns, tanks, cuttings and the line of a quay wall (Figure 3.1). Guidance such as Eurocode 7 suggests spacings of roughly 15 to 40 m for buildings and industrial structures, with closer spacing where the ground is variable. At least three points are needed even for a small building, so that the variation in level of the layers can be seen. Boreholes are often combined with cheaper and faster CPTs between them.

Depth of exploration. The investigation must extend below all the ground significantly stressed by the structure (Section 1.3):

- for pad and strip footings, to at least 1.5 to 3 times the footing width below foundation level;
- for rafts and embankments, to at least 1.5 times the width of the loaded area, which in the delta usually means through the full thickness of the soft clay;

- for piles, to at least about 5 m, and at least three pile diameters, below the expected toe level, to prove that no weaker layer lies beneath the bearing stratum.

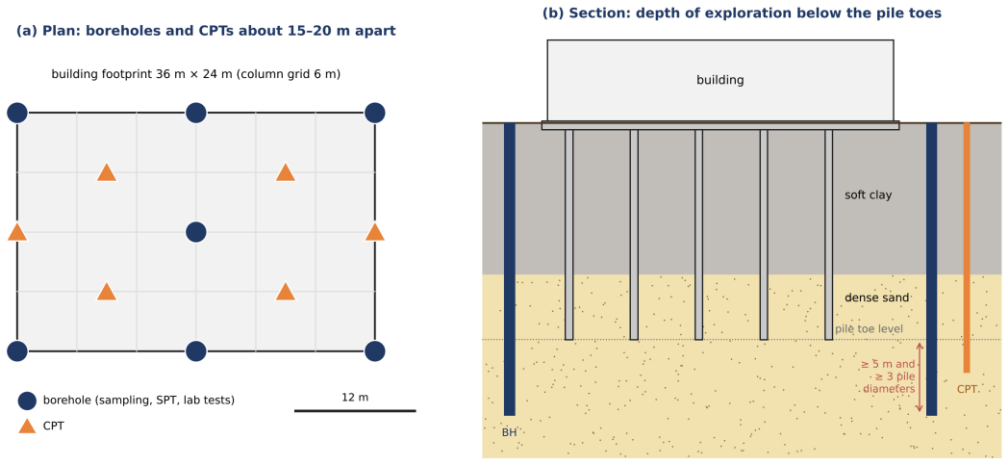


Figure 3.1. Planning exploration points for a piled building on soft ground: (a) plan showing boreholes and CPTs across the footprint; (b) section showing the required depth of exploration below the pile toes.

3.2 Interpreting in situ tests

Standard penetration test (SPT). The measured N-value depends on the energy delivered by the hammer, which varies between equipment. It is first corrected to a standard energy of 60% of the theoretical free-fall energy:

$$N_{60} = N \times \frac{E_r}{60}$$

where E_r is the energy ratio of the hammer in per cent. In sands, the N-value also increases with overburden stress, so it is normalised to an effective vertical stress of 100 kPa (about one atmosphere):

$$(N_1)_{60} = C_N \, N_{60} \qquad C_N = \left(\frac{100}{\sigma'_v}\right)^{0.5}$$

with σ'_v in kPa (Liao and Whitman, 1986). C_N is usually limited to a maximum of about 1.7. The normalised value is correlated with the angle of shearing resistance of sand, for example (Wolff, 1989):

$$\phi' \approx 27.1 + 0.3\,(N_1)_{60} - 0.00054\,(N_1)_{60}^2$$

In clays, the undrained shear strength is roughly $c_u \approx 4$ to $6\,N_{60}$ kPa (Stroud, 1974), but the SPT is a crude test in soft clay, where N-values of 0 to 2 are common.

Worked example. In a sand at 6 m depth, where $\sigma'_v = 70$ kPa, the measured N-value is 18. The hammer has an energy ratio of 50%.

$$N_{60} = 18 \times \frac{50}{60} = 15 \qquad (N_1)_{60} = 15 \times \left(\frac{100}{70}\right)^{0.5} = 17.9$$

$$\phi' \approx 27.1 + 0.3 \times 17.9 - 0.00054 \times 17.9^2 = 32.3^\circ$$

Cone penetration test (CPT). The CPT gives a continuous record of the **cone resistance** (q_c) and the **sleeve friction** (f_s) (Figure 3.2). Modern **piezocones** also measure the pore pressure behind the cone (u_2), which is used to correct the cone resistance to q_t and to detect thin drainage layers. The **friction ratio** is:

$$R_f = \frac{f_s}{q_c} \times 100\%$$

Sands give high cone resistance (typically 5 to 30 MPa) and low friction ratio (about 1% or less). Soft clays give low cone resistance (often less than 1 MPa) and higher friction ratio (typically 3% to 6%), and peats give the highest friction ratios. Charts of cone resistance against friction ratio (Robertson, 1990) are used to identify the soil type. In clays, the undrained shear strength is:

$$c_u = \frac{q_t - \sigma_{v0}}{N_{kt}}$$

where σ_{v0} is the total vertical stress and N_{kt} is a cone factor, usually between 10 and 20 and often about 15. It should be calibrated against vane or laboratory tests on the same clay.

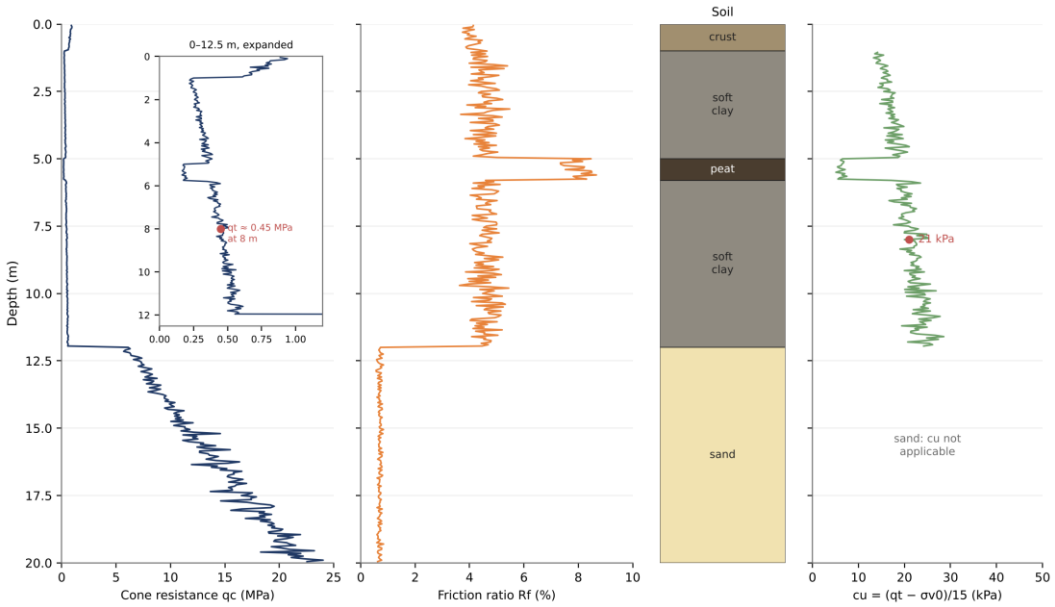


Figure 3.2. An illustrative CPT profile typical of the Niger Delta: a stiffer crust, soft clay with a peat band, and medium dense to dense sand. The soil type is interpreted from the cone resistance and friction ratio.

Worked example. At 8 m depth in the soft clay of Figure 3.2, $q_t = 0.45$ MPa. The clay has $\gamma = 16$ kN/m^3, so $\sigma_{v0} = 16 \times 8 = 128$ kPa. With $N_{kt} = 15$:

$$c_u = \frac{450 - 128}{15} = 21 \text{ kPa}$$

Other in situ tests. The **field vane** measures c_u directly in soft clay; its results are often reduced by a correction factor that depends on plasticity. The **pressuremeter** expands a probe in a borehole to measure stiffness and strength. The **plate load test** loads a steel plate at foundation level, but tests only the ground to a depth of about twice the plate width. Geophysical methods, such as seismic surveys and resistivity, help to correlate layers between boreholes.

3.3 Laboratory testing for design

Design need	Main laboratory tests
Classification and correlation	Water content, Atterberg limits, particle size, density, specific gravity (MGY 321)

Design need	Main laboratory tests
Undrained strength (short-term stability, bearing capacity, pile shaft in clay)	Unconsolidated undrained triaxial, unconfined compression, laboratory vane
Effective strength (long-term stability, earth pressure)	Consolidated undrained triaxial with pore pressure measurement, drained triaxial, shear box
Settlement and rate of settlement	Oedometer: C_c, C_r, σ'_p, c_v, C_α
Seepage and drainage	Constant-head and falling-head permeability tests; k from oedometer
Fill and road materials	Compaction and California bearing ratio tests (compaction is covered in MGY 321)
Durability of concrete and steel	Sulphate, chloride and pH tests on soil and groundwater

Good laboratory results depend on good samples. Soft clays are easily disturbed by poor sampling, transport or storage, which reduces their apparent strength and makes the preconsolidation pressure hard to identify.

3.4 Design parameters and factors of safety

Selecting parameters. Test results scatter because the ground is variable and tests are imperfect. Design values are chosen as a **cautious estimate** of the value that governs the behaviour of the ground, not simply the average or the lowest value. For example, the average shear strength along a long slip surface matters more than one weak result, whereas the base resistance of a single pile depends on the soil just beneath its toe. Results are plotted against depth, and the ground model is used to group them by layer.

Factors of safety. In traditional practice, a single global factor of safety is applied to account for uncertainty in loads, parameters and methods:

Design check	Typical global factor of safety
Bearing capacity of shallow foundations	3
Axial capacity of piles (calculated / confirmed by load tests)	2.5 / 2
Stability of permanent slopes and embankments	1.3 to 1.5
Sliding of retaining walls	1.5
Overturning of retaining walls	2
Piping (on the exit gradient)	3 to 4

Limit state codes such as **Eurocode 7** apply separate **partial factors** to the actions (loads) and to the soil parameters instead. For example, in one of its combinations the design value of $\tan\phi'$ is the characteristic value divided by 1.25, and the design value of c_u is the characteristic value divided by 1.4. Separate checks are made for the ultimate limit state (collapse) and the serviceability limit state (settlement and deformation).

3.5 Geotechnical reports

The results of a ground investigation are usually presented in two reports:

- The **factual report** contains the description of the site and the work done, borehole and trial pit logs, CPT plots, laboratory test results and groundwater observations, without interpretation.
- The **interpretative (geotechnical design) report** presents the ground model, with cross-sections, selected design parameters for each layer, the assessment of hazards, foundation recommendations with bearing pressures or pile capacities, predicted settlements, and requirements for construction and monitoring.

A good report states clearly the limitations of the investigation, such as the spacing of boreholes and the conditions that may lie between them. Many disputes and failures arise when the ground encountered during construction differs from that assumed, so the report should also describe what to look out for on site.

3.6 Construction control and monitoring

The ground is never known completely, so geotechnical work continues during construction:

- **Supervision**: logging excavations, checking that formations match the ground model, and recording the driving of every pile.
- **Testing**: pile load tests, compaction control of fill, and checks on concrete and materials.
- **Instrumentation**: **piezometers** measure pore water pressure; **settlement plates** and surveys measure settlement of fills and structures; **inclinometers** measure horizontal movement in slopes and beside excavations; **extensometers** measure compression at different depths.

Where the ground is uncertain, the **observational method** may be used: the design is based on the most probable conditions, the behaviour is monitored closely, and planned changes are made if the measurements exceed trigger values. Fills on soft clay are a classic example: the rate of filling is controlled by measured pore pressures and settlements so that the clay gains strength by consolidation before the next layer is placed.

3.7 Applied practice: typical projects

A jetty in a delta creek. The investigation is made from a pontoon or jack-up barge, with boreholes, CPTs and sampling at each line of piles, and a bathymetric survey of the creek bed. The usual profile is very soft clay or silt at the bed, over sand. The design must allow for scour around the piles during floods and tidal flows, the weak lateral support of the soft upper soil, corrosion of steel piles in the splash and tidal zones, and the tidal range (Figure 3.3). Piles are driven into the dense sand, and their capacity is confirmed by driving records and dynamic tests.

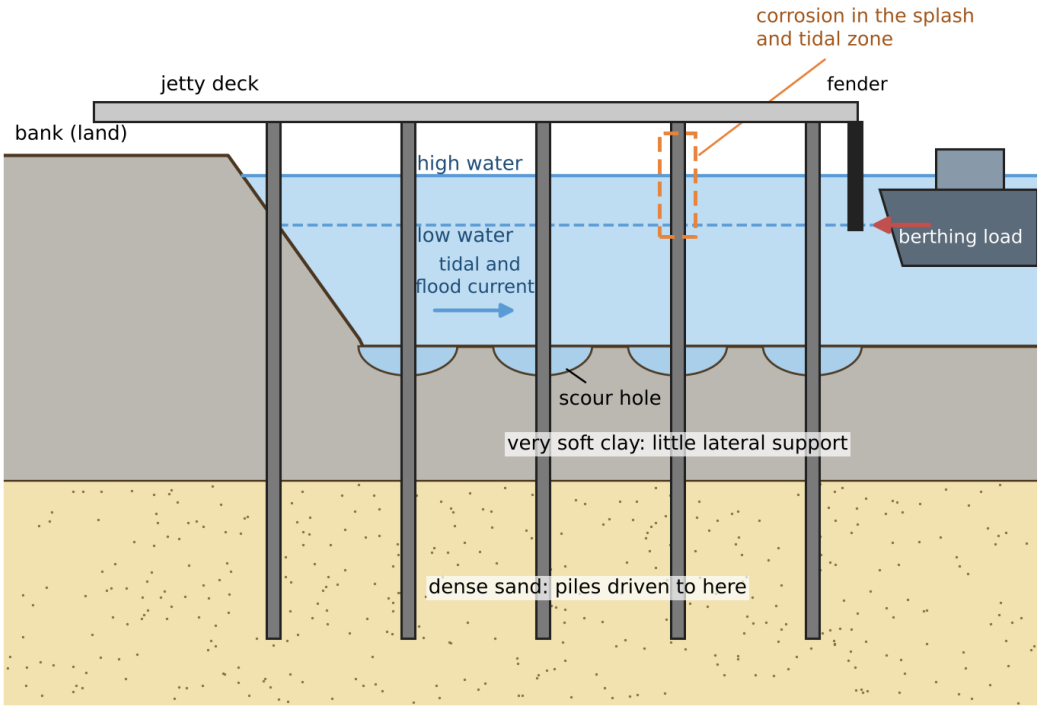


Figure 3.3. Geotechnical issues for a piled jetty in a Niger Delta creek: tidal range, scour around the piles, soft clay with little lateral support, corrosion in the splash zone, and piles driven into the dense sand below.

A road embankment across a swamp. Embankments on soft clay can fail by bearing failure or rotational slip if built too quickly, and will settle by large amounts for years (Section 1.4). Solutions include building in stages with waiting periods while the clay consolidates and gains strength, using geotextiles or geogrids at the base, light-weight fill, vertical drains and preloading, or carrying the road on a piled viaduct over the worst ground. Monitoring of settlement and pore pressure is essential.

A multi-storey building on reclaimed land. In coastal cities such as Lagos, Warri and Port Harcourt, many sites are underlain by sand fill over soft clay or peat. Shallow foundations in the fill may appear satisfactory but settle as the underlying clay consolidates. Piles must be taken through the soft layers into dense sand, and the effect of downdrag from the fill must be included (Section 2.5).

Building failures. Investigations of building collapses in Nigeria have repeatedly pointed to inadequate or absent soil investigation, poor foundation design and construction, substandard materials and unapproved changes to buildings. Many of these failures could have been prevented by a proper ground investigation, foundation design based on it, and supervision of construction by qualified professionals.

3.8 Professional practice

Geotechnical work affects public safety, so it must be carried out by competent, registered professionals: engineers registered with the Council for the Regulation of Engineering in Nigeria (COREN), and geoscientists registered with the Council of Nigerian Mining Engineers and Geoscientists (COMEG). Professional practice also requires:

- honest reporting of what was found, including uncertainties and limitations;
- working within one's competence and seeking specialist advice where needed;

- safety on site, especially when drilling over water, near buried pipelines and power cables (common in the delta), in deep trial pits and near unstable slopes;
- care for the environment, including proper disposal of drilling fluids and protection of groundwater.

3.9 Key points of Section 3

- Exploration points must cover the footprint and reach below all significantly stressed ground; for piles, at least about 5 m below the toe.
- SPT values are corrected for hammer energy and overburden before use; CPT results give soil type and, in clays, undrained strength.
- Design parameters are cautious estimates of the values that govern behaviour; global factors of safety or partial factors allow for uncertainty.
- Factual and interpretative reports record the investigation and the ground model; construction must be supervised and monitored.
- Jetties, embankments and buildings on the soft soils of the delta need piles, staged construction, drainage or monitoring, and careful allowance for scour, downdrag and settlement.

Summary, review questions and glossary

Summary

Topic	Main ideas	Niger Delta and marine applications
Stresses and seepage	Effective stress; K_0; flow nets; critical gradient; piping and heave	Cofferdams and excavations below the water table
Stress distribution	Boussinesq; pressure bulbs; 2:1 method	Depth of investigation beneath rafts and fills
Consolidation	Oedometer; C_c, C_r, σ'_p, OCR; settlement; T_v, c_v; secondary compression	Long-term settlement of fills and buildings on swamp clays and peats
Shear strength	Mohr–Coulomb; triaxial and shear box tests; drained and undrained; dilation; liquefaction	Undrained stability of embankments on soft clay
Earth pressure	At rest, active and passive; Rankine; water pressure	Quay walls, sheet pile walls, basements
Slope stability	Infinite slope with seepage; method of slices	Creek and riverbank failures; rapid drawdown
Shallow foundations	Failure modes; bearing capacity equation; net and allowable pressure; settlement limits; settlement on sand and clay	Rafts on sand fill; footings on laterite and sand
Piles	Shaft and base resistance; α-method; API method for sand; negative skin friction; testing	Jetties, offshore platforms, buildings on soft ground

Topic	Main ideas	Niger Delta and marine applications
Site practice	Planning; SPT and CPT interpretation; laboratory tests; parameters; factors of safety; reports; monitoring; professional practice	Over-water investigations; staged embankments; reclaimed land

Summary of key equations

Quantity	Equation	Section
Coefficient of earth pressure at rest	$K_0 = 1 - \sin\phi'$	1.1
Seepage from a flow net	$q = kH\dfrac{N_f}{N_d}$	1.2
Critical hydraulic gradient	$i_c = \dfrac{\gamma'}{\gamma_w} = \dfrac{G_s - 1}{1 + e}$	1.2
Boussinesq point load	$\Delta\sigma_z = \dfrac{3Q}{2\pi z^2}\left[\dfrac{1}{1 + (r/z)^2}\right]^{5/2}$	1.3
2:1 load spread	$\Delta\sigma_z = \dfrac{qBL}{(B + z)(L + z)}$	1.3
Compression index from liquid limit	$C_c \approx 0.009(\mathrm{LL} - 10)$	1.4
Consolidation settlement (normally consolidated)	$S_c = \dfrac{C_c H}{1 + e_0}\log_{10}\dfrac{\sigma'_0 + \Delta\sigma'}{\sigma'_0}$	1.4
Time factor	$T_v = \dfrac{c_v t}{H_{dr}^2}$	1.4
Secondary compression	$S_s = \dfrac{C_\alpha H}{1 + e_p}\log_{10}\dfrac{t_2}{t_1}$	1.4
Mohr–Coulomb	$\tau_f = c' + \sigma'_n\tan\phi'$	1.5
Rankine coefficients	$K_a = \tan^2(45° - \phi'/2),\ K_p = \tan^2(45° + \phi'/2)$	1.6
Active thrust	$P_a = 1/2\, K_a\gamma H^2$	1.6
Infinite slope with seepage	$F = \dfrac{\gamma'}{\gamma_{sat}}\cdot\dfrac{\tan\phi'}{\tan\beta}$	1.7
Bearing capacity (strip)	$q_u = c'N_c + qN_q + 1/2\,\gamma BN_\gamma$	2.3
Undrained bearing capacity	$q_{u,net} = 5.14\, c_u$ (strip), $\approx 6.2\, c_u$ (square)	2.3
Elastic settlement	$S_i = \dfrac{qB(1 - \nu^2)}{E_s} I_s$	2.4
Allowable pressure on sand (25 mm)	$q_a \approx 8N_{60}\left(\dfrac{B + 0.3}{B}\right)^2$	2.4

Quantity	Equation	Section
Pile capacity	$Q_u = q_b A_b + \sum f_s A_s$	2.5
SPT corrections	$N_{60} = N\frac{E_r}{60}, (N_1)_{60} = N_{60}\left(\frac{100}{\sigma'_v}\right)^{0.5}$	3.2
Undrained strength from CPT	$c_u = \frac{q_t - \sigma_{v0}}{N_{kt}}$	3.2

Review questions

Part A: Short answers

1. State the principle of effective stress and explain why it is central to soil mechanics.
2. What are flow lines and equipotential lines? List the rules for drawing a flow net.
3. Explain piping and heave, and how the factor of safety against each is assessed.
4. What is a pressure bulb, and how does it guide the depth of a ground investigation?
5. Describe the oedometer test and sketch a typical plot of void ratio against the logarithm of effective stress.
6. Define the preconsolidation pressure and the overconsolidation ratio, and explain their importance for settlement.
7. Why does consolidation take so long in thick clay layers, and how can it be accelerated?
8. Distinguish between UU, CU and CD triaxial tests and state what each is used for.
9. Explain why a foundation on soft clay is usually most critical in the short term, while a cutting in clay may be most critical in the long term.
10. Describe the behaviour of dense and loose sands in shear. What is liquefaction?
11. Explain the difference between at-rest, active and passive earth pressure.
12. Why must water pressure be considered separately in the design of a quay wall?
13. Describe the three modes of bearing capacity failure.
14. Why does settlement, rather than bearing capacity, often govern the design of footings on sand?
15. Distinguish between driven and bored piles, and between end-bearing and friction piles.
16. What is negative skin friction, and when does it occur?
17. How should the depth of boreholes be chosen for a piled building?
18. Why is the SPT N-value corrected for hammer energy and overburden stress?
19. What information is given by a piezocone test?
20. What is the observational method? Give an example of its use.

Part B: Calculations

21. A sheet pile cofferdam produces a flow net with 5 flow channels and 12 equipotential drops. The head difference is 5 m and $k = 4 \times 10^{-5}$ m/s. Calculate the seepage per metre length of wall per day.
22. A 3 m × 3 m footing carries 200 kPa. Using the 2:1 method, calculate the stress increase at depths of 1.5 m, 3 m and 6 m below the footing, and comment on the result.
23. A 4 m thick clay layer has $e_0 = 1.10$, $C_c = 0.35$, $C_r = 0.06$, $\sigma'_0 = 60$ kPa and $\sigma'_p = 90$ kPa at mid-depth. A building increases the stress at mid-depth by 50 kPa. Calculate the consolidation settlement.

24. The clay in question 23 drains only at its top surface and has $c_v = 2\ \text{m}^2/\text{year}$. How long will it take to reach 60% consolidation?
25. Two drained triaxial tests on a clay give failure at $\sigma'_3 = 100$ kPa, $\sigma'_1 = 320$ kPa and at $\sigma'_3 = 200$ kPa, $\sigma'_1 = 560$ kPa. Determine c' and ϕ'.
26. An 8 m high quay wall retains sand with $\phi' = 30°$, $\gamma = 18\ \text{kN/m}^3$ above the water table and $\gamma_{sat} = 20\ \text{kN/m}^3$ below it. The water table behind the wall is 3 m below the top and the water in front of the wall is at the base level of the wall. Draw the pressure diagrams and calculate the total horizontal thrust from soil and water.
27. A 2.5 m square footing is placed at 1.2 m depth in sand with $\phi' = 34°$ and $\gamma = 18\ \text{kN/m}^3$. Calculate the ultimate and net allowable bearing pressures, using Terzaghi's shape factors and a factor of safety of 3.
28. A 0.6 m diameter bored pile is 15 m long in stiff clay with $c_u = 80$ kPa. Taking $\alpha = 0.5$, calculate its ultimate and allowable capacities.
29. At 10 m depth in a sand ($\sigma'_v = 95$ kPa) the measured SPT N-value is 24 with a hammer of energy ratio 72%. Calculate N_{60}, $(N_1)_{60}$ and the estimated ϕ'.
30. A CPT in soft clay records $q_t = 0.62$ MPa at 12 m depth. The clay has $\gamma = 16.5\ \text{kN/m}^3$. Estimate c_u with $N_{kt} = 15$.

Part C: Essay questions

31. Discuss the causes, magnitude and rate of settlement of structures founded on the soft clays of the Niger Delta, and the methods used to control them.
32. Compare shallow and piled foundations for a four-storey building on reclaimed land in a coastal Nigerian city.
33. Describe the geotechnical investigation and design of a piled jetty in a Niger Delta creek.
34. Discuss the role of the engineering geologist in the investigation, design and construction monitoring of a road embankment across a swamp.
35. Explain how ground investigation, foundation design and site supervision could reduce building failures in Nigeria.

Glossary

Term	Meaning
Active pressure	The minimum lateral earth pressure, reached when a wall moves away from the soil
Bearing capacity	The pressure a foundation can apply before the ground fails in shear
Coefficient of consolidation	A soil property (c_v) that controls the rate of consolidation
Compression index	The slope of the virgin compression line on a plot of void ratio against log effective stress
Cone penetration test (CPT)	An in situ test in which a cone is pushed into the ground while its resistance is recorded
Consolidation	The gradual compression of a saturated soil as excess pore pressure dissipates
Critical hydraulic gradient	The upward gradient at which the effective stress in a soil becomes zero
Differential settlement	The difference in settlement between two parts of a structure

Term	Meaning
Downdrag (negative skin friction)	Downward load on a pile caused by soil settling around it
Drained condition	Loading slow enough for excess pore pressures to dissipate
Equipotential line	A line joining points of equal total head
Excess pore pressure	Pore pressure above the steady-state value, caused by loading
Flow net	A diagram of flow lines and equipotential lines for seepage
Heave	Upward movement of soil at the base of an excavation caused by seepage or unloading
Liquefaction	Loss of strength of a loose saturated sand as pore pressure rises
Mohr–Coulomb criterion	The linear relationship between shear strength and normal effective stress
Observational method	Design and construction adjusted in response to monitoring
Oedometer	A laboratory apparatus for measuring the compressibility of soil
Overconsolidation ratio	The preconsolidation pressure divided by the present effective stress
Passive pressure	The maximum lateral earth pressure, reached when a wall is pushed into the soil
Piezocone	A cone penetrometer that also measures pore water pressure
Pile	A long slender foundation member that carries load to depth
Piping	Internal erosion of soil by seepage, starting at the exit point
Preconsolidation pressure	The greatest effective stress a soil has carried in the past
Pressure bulb	The zone beneath a foundation bounded by a line of equal stress increase
Raft	A foundation that covers the whole area of a structure
Scour	Erosion of the bed around structures by flowing water
Secondary compression	Compression by creep after the dissipation of excess pore pressure
Time factor	A dimensionless measure of time in consolidation theory
Undrained condition	Loading too quick for drainage, so that the volume of a saturated soil does not change
Undrained shear strength	The shear strength of a saturated clay under undrained loading (c_u)

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