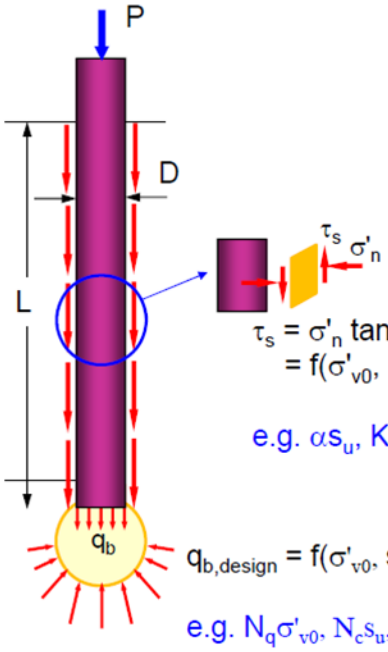
Pile Foundations

Axial pile capacity – input soil parameters

- Soil 'strength'
  - s_u, q_{ucs}, ϕ' by deduction only (q_c as primary)
  - also by correlation with SPT blowcount N
- What is most appropriate measure of strength for pile design?
- How do pile type, geometry and construction methods affect design parameters?



$\tau_s = \sigma'_n \tan\delta$
$= f(\sigma'_{v0}, K_0, s_u, q_c, \phi' \text{ (or } \delta\text{) etc})$

e.g. αs_u, $K\sigma'_{v0} \tan\delta$, $\beta\sigma'_{v0}$, $f_s q_c$

$q_{b,design} = f(\sigma'_{v0}, s_u, q_c, \phi' \text{ etc})$

e.g. $N_q\sigma'_{v0}$, $N_c s_u$, $f_b q_c$

Notes for OCF1 Foundation Design 2023

1.0 Background

The Netherlands is a predominantly flat country, with surface levels at or below sea level. About 60% of the Dutch shallow subsurface consists of fluvial and coastal lowlands of Holocene age (the Holocene is the current geological epoch which began at the end of the last ice age approximately 10,000 years ago), found around or below Dutch ordnance level NAP (roughly mean sea level). The remaining part of the country consists almost entirely of Pleistocene terrains: sandy soils sloping upwards to the south and the east, with an average elevation of between 10 and 20 m above NAP. The depth of these older Pleistocene deposits dips towards the West of the country, See Figure 1.1. The older deposits are aged and in parts over-consolidated by glacial action. In simple terms the strength and stiffness of aged and over-consolidated soils is much higher than for recent soils. Therefore, in the Western parts where soft soils are up to 20-25m thick the vast majority of building are supported by pile foundations. In the Eastern (higher) part of the country, shallow foundations are typically used for with pile foundations being used only for deeper or heavier structures.

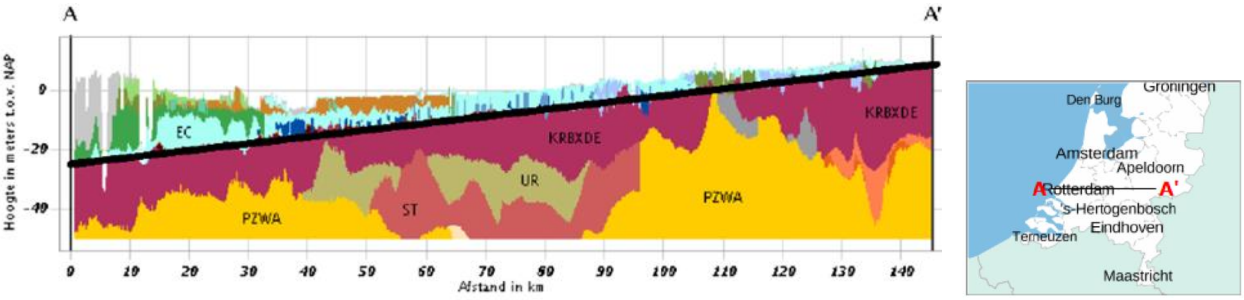


Figure 1.1 Soil profile from the West to East of the Netherlands. The black line shows the separation of the Pleistocene (older) and Holocene (recent) deposits (TNO 2016).

A pile is a structural member used to transmit loads through water and soft soils to deeper bearing stratum. They have been used since Neolithic times to protect dwellings from flooding and to allow the construction of fortified dwellings. The Romans used timber piles (trees with the branches removed) in bridge construction more than 2000 years ago. Wooden piles have been used extensively in the Netherlands since the 15th century, See Figure 1.2. Klaassen et al. (2012) suggests that 25,000,000 wooden piles were used to support structures in the Netherlands prior to the adoption of precast concrete piles after 1960.

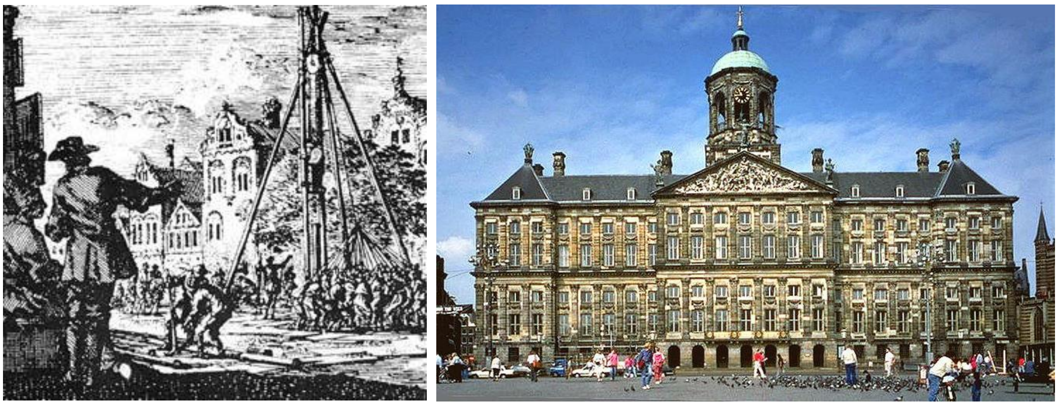


Figure 1.2 (a) Wooden pile driving in Amsterdam, (b) Palace of the dam constructed on 13,659 wooden piles (from van Baars 2016).

2.0 Pile Types

2.1 Introduction

The load carrying capacity and stiffness response of a pile foundation depends on (i) the soil conditions and (ii) the installation of the pile and (iii) pile properties. Pile installation effects are critically dependent on the form of construction chosen and in this section we describe the process of constructing piles.

Piles can be broadly classified by their form of construction, as either:

- Displacement – i.e. pre-fabricated units made in a factory, transported to site and driven or jacked into the soil and
- Replacement – where an excavation is made using one of a range of possible techniques, See Table 1 and the pile is constructed in place.

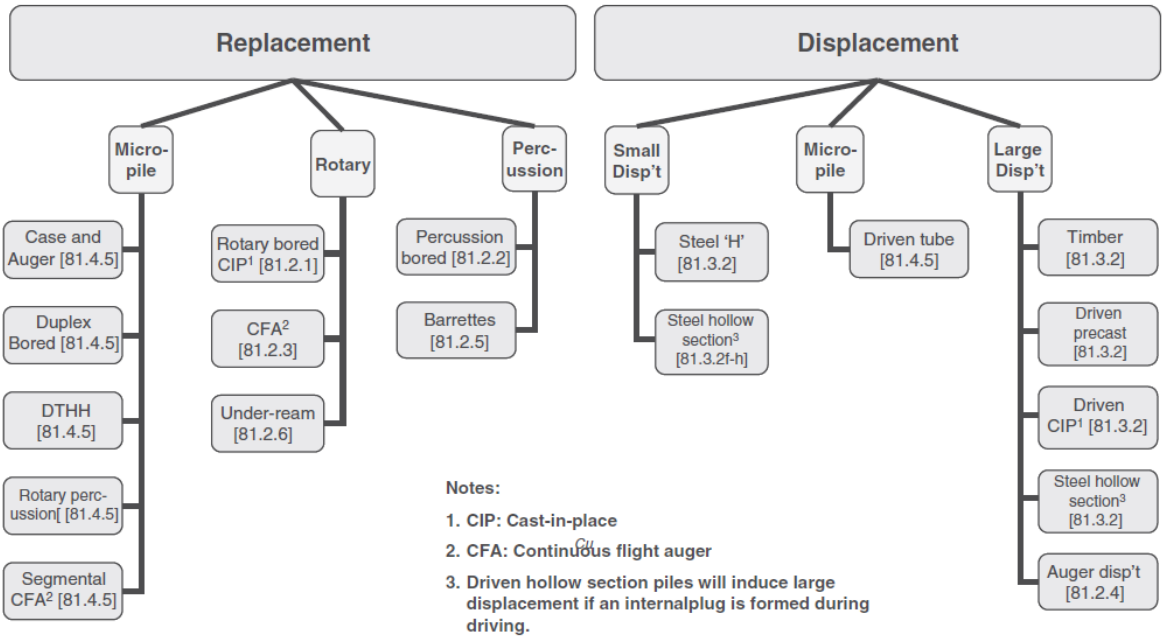


Table 2.1 General Classification of Pile Types (after ICE 2012)

The pile types most commonly used pile types in the Netherlands (Reinders et al 2016) include:

1. Precast square concrete driven, displacement type piles (>50% of total)
2. Cast in situ concrete piles, replacement (approximately. 30%)
   - Driven with a steel casing, at the final penetration and after pouring concrete into the casing, the casing is retrieved by upward driving or upward vibrating. (Vibro-pile)
   - Screwed casing with screw plate at the tip. At final penetration and after pouring concrete into the casing, the casing is retrieved (Screw-injection Pile, SiP).
   - Continuous Flight Auger, CFA piles
   - Bored piles. The borehole stability is achieved using bentonite support fluid. After achieving the required depth, concrete replaces the bentonite.

With the remained of the market being largely;

3. Steel piles, displacement type
   - Closed ended tubular driven pile
   - Open ended tubular driven pile

- Steel H-pile with grouting
- Steel screwed closed ended tubular pile
- Steel screwed closed ended tubular pile with grouting along the perimeter
- Open ended tubular pile, installed by percussion drilling.
-

4. Micropiles, replacement type
   - Cast in-situ with drilling system
   - Cast in-situ with anchor tubes and drill bit
   - Cast in-situ with anchor tubes and flight auger blades
   - Cast in-situ with temporary casing

2.2 Driven pre-cast concrete or steel displacement piles

Precast concrete piles are generally made of reinforced concrete (occasionally pre-stressed) and typically come in square, solid cross-section and sizes from 0.25m x 0.25m to 0.5m x 0.5m are generally used. The piles can be cast as single elements, usually aroun 20 t0 30m long although exceptionally up to 40m long, 0.5m square piles have been installed in the Port of Rotterdam. They can also be formed in smaller lengths and joined in the ground. Steel tubes with or without end plates can be installed. When the piles are driven (with a hammer, in Dutch "heien") as solid cross-section concrete or steel piles with a closed-end they are known as full displacement piles. These piles displace a volume of soil equal to their own volume and therefore they increases stresses and densify of the ground during installation.

Figure 2.1 (a) Precast concrete piles, (b) steel open-ended tube piles and (c) Steel H-Piles

In cases where long pile penetrations are required, for example to resist tension or lateral loads, steel tubes can be driven without an end plate (open-ended). These piles develop much lower end bearing resistance and thus are much easier to drive. These open-ended piles are known as partial displacement piles as they displace a smaller volume of soil.

Other open cross-sections such as H-Piles are commonly used for example in supporting bridge structures. The resistance of H-Piles and other open ended steel piles is dependent on whether soil plugs form during the flanges of the steel plates during installation, See Figure 2.1c and 2.2 thus causing additional volumetric displacement.

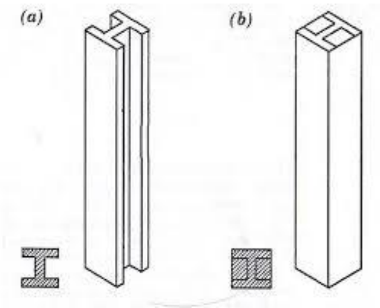


Figure 2.2 Steel H-Pile driven in (a) Unplugged condition (b) and Plugged condition

In order to mitigate noise and vibration problems new techniques like press-in piles and vibrated-in piles are being used more frequently. For the press-in method the piles are generally expected to have higher capacity and stiffness than piles driven in place. On the other hand, for vibration based methods uncertainty remains about the reduction of soil strength and stiffness caused by installation.

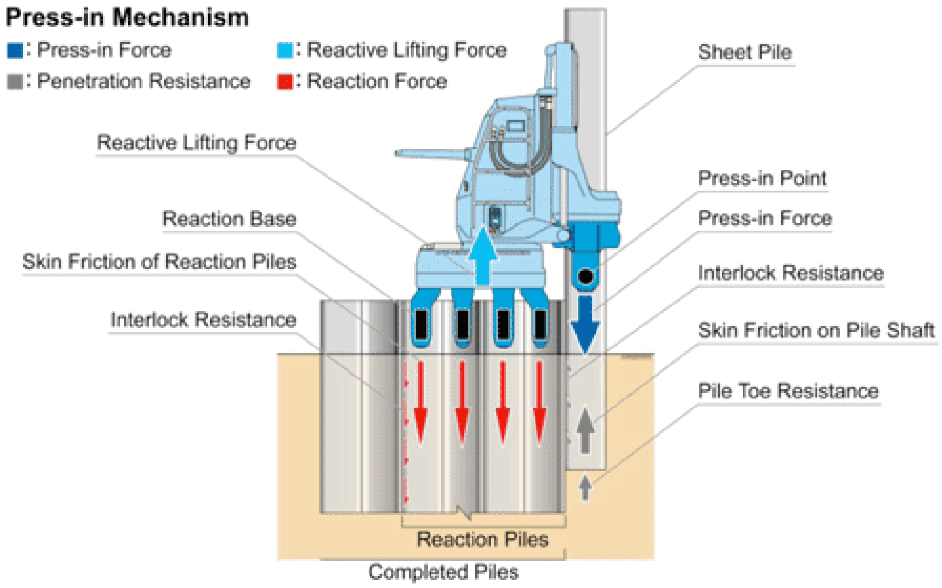


Figure 2.3a Press-in piling for steel piles from Giken Ltd. https://www.giken.com/en/press-in_method

Figure 2.3b Press-in piling for precast concrete piles (used for example in project Sebastiaansbrug Delft, 2019, drukpaal.nl)

Some advantages and disadvantages of displacement piles are summarised below:

Advantages of Displacement Piles	Disadvantages of Displacement Piles
• Densify the soil • Increase stress and stiffness of soil around the pile tip • Good quality control since piles are pre-formed in factory conditions • There is no time lag for concrete curing in the ground • The size and position of reinforcement is known • Different shapes and lengths can be constructed • Useful to support structures in water	• Can be difficult to drive in certain soils such as glacial till and soils with cobbles and boulders • Noise levels and/or vibrations levels are high (except for press-in) • Very long piles are difficult to transport • Additional reinforcement might be required for driving • If the pile reach refusal early, the pile must be cut-off leading to waste • Need to be careful with handling

Video references	
https://www.youtube.com/watch?v=bFzK1GcmmTg	Pile driving construction checks, templates, cushions, lifting, driving, rebound, level checks, driving criteria
https://www.youtube.com/watch?v=Qjb8gBwl0hU	See the installation and handling of piles for onshore wind turbine bases

2.3 Cast in-situ concrete or replacement piles

There are numerous pile types of this type on the market. Summarised below are examples of the systems used in the Netherlands and some common worldwide solutions.

Vibro or driven cast-in-place, DCiP

The process of forming DCiP piles is described in Flynn and McCabe (2016) and illustrated in Figure 2.4. A hollow steel tube with a sacrificial circular steel base plate is top-driven into the soil. The base plate which has a diameter slightly greater than the steel tube prevents ingress of soil and water during driving. When the tube reaches the required depth of penetration, the hammer is retracted and the pile is constructed by pouring concrete into the installation tube. When concreting is complete, the hammer is reattached and the tube is extracted from the soil (using a combination of some hammer blows and vibration to remove the pile – hence the name). The pile base is sacrificially remaining in place. Reinforcement may be placed either before or after concreting. The pile is then left to cure in-situ for a number of days, with the steel plate remaining at the base. The pile shaft should be relatively uneven thus providing high shear resistances. One thing that is uncertain is the final shaft diameter, it should be a minimum diameter equal to the steel tube used for installation and a maximum of the base plate diameter, assuming an open hole was formed as the larger base plate penetrated the soil. In reality it is probably somewhere between the two. Generally, the size of the base plate is larger than the tube for easier installation and removal. If the difference between the base plate and shaft diameter is too large then the shaft friction might be affected. A variation is possible with installation of a precast pile in the tube and backfill of the remaining space with concrete. This type is called a vibro-combination pile.

See a video of the construction process: https://www.vroom.nl/en/products/8-vibrated-hbf-type-pile

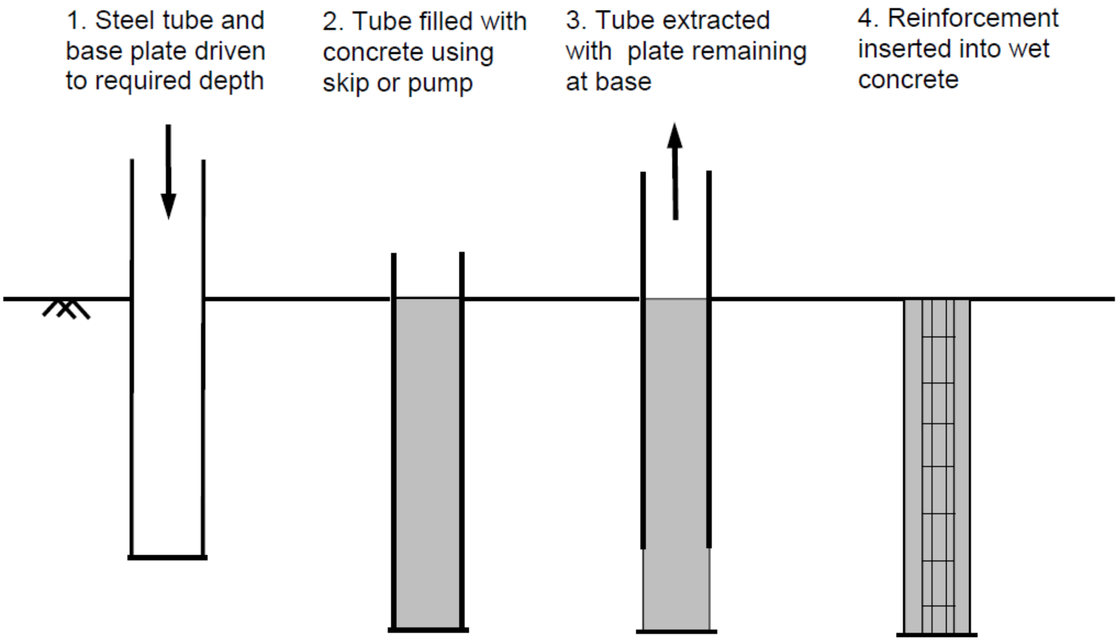


Figure 2.4 Process for construction of Vibro or DCiP piles (Flynn and McCabe 2016)

Screw-Injection Piles

For sites where noise and vibration should be minimised the Screw injection Pile, SiP provides a solution. In the first stage, Figure 2.5a, a steel tube with an expanded, sacrificial end tip is rotated (screwed into the ground) with the machine providing a vertical (crowd) force. Grout can be injected at the pile tip during installation in hard ground. When the tip reaches the required depth the reinforcement cage is placed and concrete is poured. The casing is extracted using an oscillating motion (Fundex type). Alternatively, the casing can be left in place (Tubex type).

Installation process for SiP https://www.youtube.com/watch?v=IkX3HNNvWH8

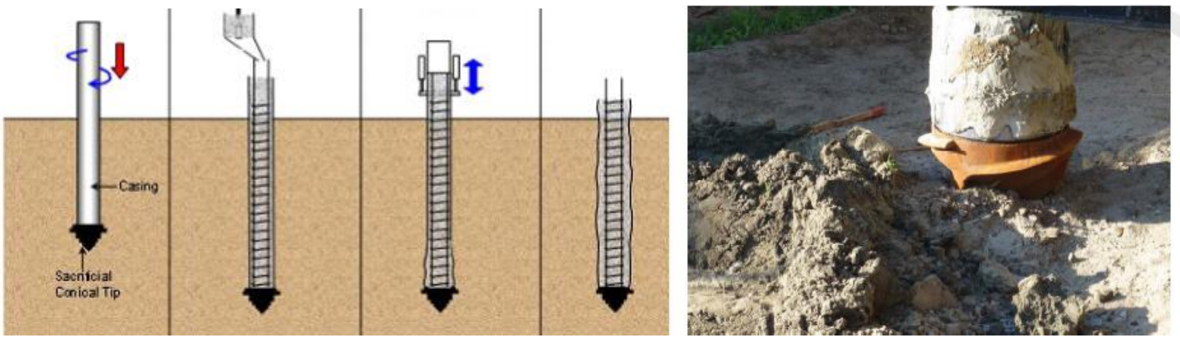


Figure 2.5 (a) Construction of SiP Piles (b) Sacrificial end plate (courtesy of Fundex Ltd.)

Auger Cast in Place

A range of augering systems are used to form replacement piles. Where temporary casings are utilised, See Figure 2.6 the system has the advantage that a dry bore is formed and the reinforcing and concrete can be placed in well-controlled conditions, where loose/weak soils are fully supported. The system has the following advantages, (i) Flexibility in terms of diameter and length, (ii) ability to penetrate very hard ground (iii) continuous reinforcements with no joints and (iv) relatively low noise and vibration.

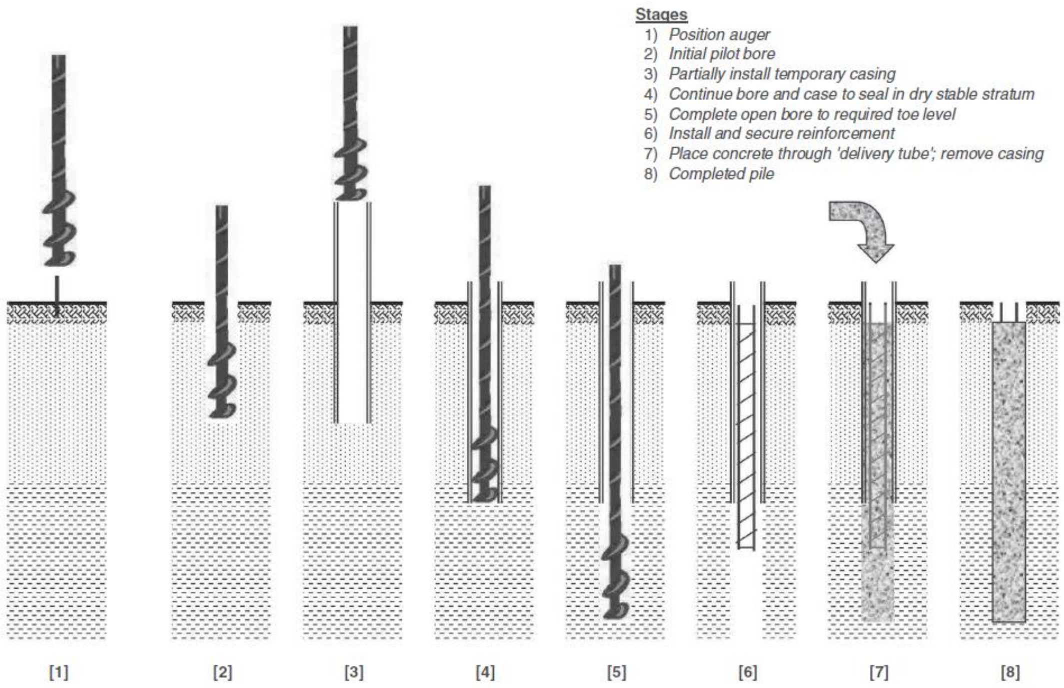


Figure 2.6 Construction of Auger Pile with Casing (ICE Manual of Geotechnical Engineering)

Continuous Flight Auger, CFA Pile

A very popular alternative form of auger pile is the CFA pile. In this system a hollow-stem auger is screwed into the ground, See Figure 2.7. When the tip reaches the target penetration concrete is injected at pressure through the tip as the auger is slowly withdrawn. When the auger is removed and the concrete reaches ground level, a reinforcing cage can be placed, using some crowd force from the piling rig to push downwards.

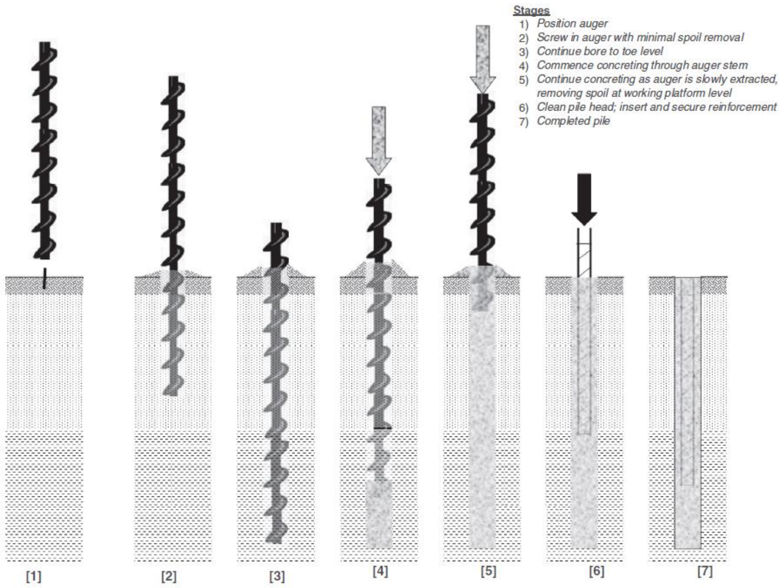


Figure 2.7 Construction of Continouos Flight Auger Pile (ICE Manual of Geotechnical Engineering)

Drilled Displacement Piles

Prezzi and Basu (2005) provide a comprehensive overview of the range of drilled displacement piling systems commonly used in the European and US markets. They note that the soil displacement produced during installation varies from system to system such that they vary from a full to partial displacement pile. A summary of common types used in the Netherlands and Belgium is given in Table 2.2. They note the importance of installation monitoring for all replacement piles. Modern piling rigs have a number of monitoring systems built in to measure auger speed, pitch, concrete flow rates etc.

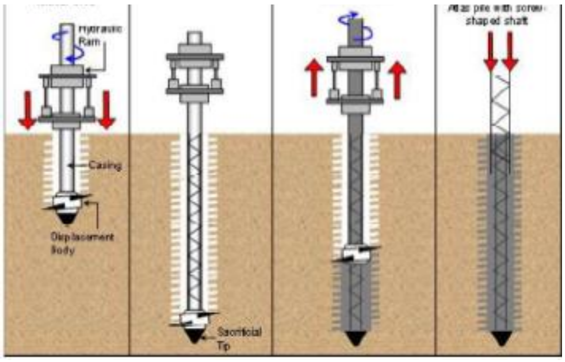
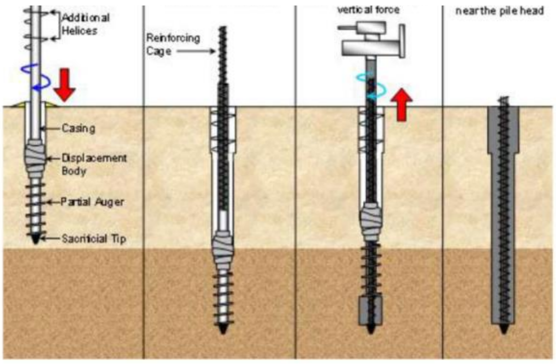
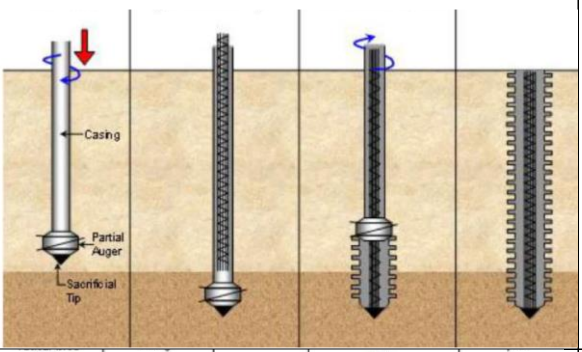
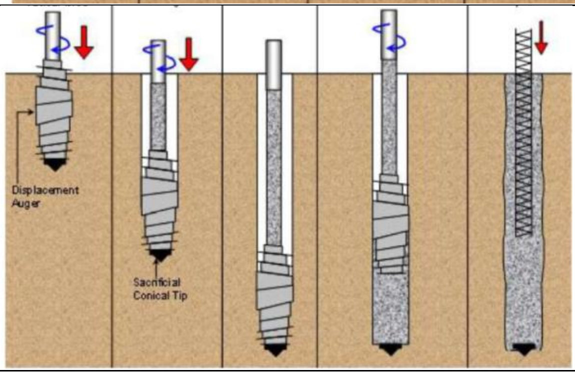
Drilled Displacement Piles Systems	Features
	**Atlas Pile:** A sacrificial tip is attached to a steel casing through a watertight connection. Installation consists of continuous clockwise rotation with a vertical force applied. When the penetration depth is reached the steel shoe is detached from the casing. A reinforcement cage is placed and high-slump concrete is poured through a hopper to form a screw shaped shaft. Typical shaft diameters are 0.31m to 0.56m and base diameter is 0.45m to 0.81m. Shaft lengths can be up to 25m.
	**De Waal Pile:** A drilling tool consisting of a sacrificial tip, a partial flight auger and a displacement body is used to install the De Waal pile. The drilling tool is rotated clockwise using torque and a vertical force to achieve the required penetration depth. The tip is released and the reinforcing cage is added. Concrete is injected as the casing is extracted. The resulting shaft is smooth and an enlarged head is created by the helices near the ground surface.
	**Olivier Pile:** Installation is similar to the Atlas system, whereby a sacrificial tip is attached to a partial flight auger that is in-turn attached to a casing. The casing is rotated into the ground using torque and vertical force. At the final depth, the tip is released and the reinforcement cage is inserted. Concrete is placed using a funnel and the casing and partial flight auger are removed by counter-clockwise rotation. The resulting shaft is in the shape of a screw.
	**Omega Pile:** Drilling is achieved using a displacement auger attached to a sacrificial tip connected to a casing. Concrete is injected under pressure into the casing before the final penetration depth is reached. Once the final depth is achieved the tip is released and the auger and casing is rotated slowly out. The reinforcement is then inserted into the wet concrete.

Table 2.2 Description of common drilled displacement pile systems (after Prezzi and Basu 2012)

2.5 Choice of pile foundation

The ICE geotechnical engineering manual provides useful guidance on factors to consider when choosing foundations options, See Figure 2.8.

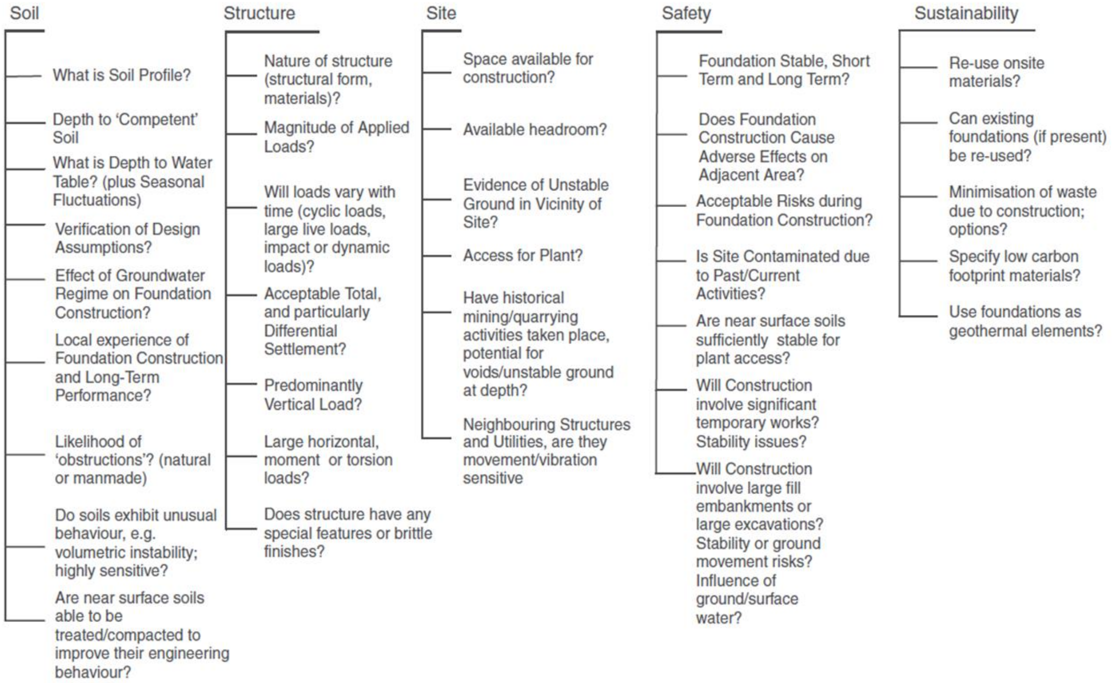


Figure 2.8 Factors to consider in foundation choice (ICE Manual of Geotechnical Engineering)

In reality, the choice usually comes down to cost, environmental impact and local experience.

For reflection consider technical options for the following cases:

1. A site where 6m of soft clay overlies bedrock, static axial compression loads are applied from a commercial development, normal settlement criteria for buildings apply and no adjacent sensitive structures are present.
2. Same site as 1 but structure is an electricity pylon and tension and compression loads are applied.
3. A commercial development in Amsterdam where 15m of soft clay overlies 1st and 2nd sand layers. Static axial compression loads are applied, normal settlement criteria for buildings apply and there are adjacent sensitive structures present at short distance.
4. A highway bridge pier with predominantly vertical loading and 5% horizontal loading due to traffic braking forces. 5 m soft clay over dense sand.
5. A 5 MW offshore wind turbine with significant horizontal and lateral loads from environmental (wind and wave forces).

3.0 Ground Investigation for piling projects

3.1 Introduction

Soil can best be described as heterogeneous – the material will be variable in both lateral and vertical extent, See Figure 3.1. Even when the soil type is relatively constant – parameters such as strength and stiffness vary with depth because of varying stress level. Therefore each site we consider has unique properties that we need to quantify as well as possible prior to construction.

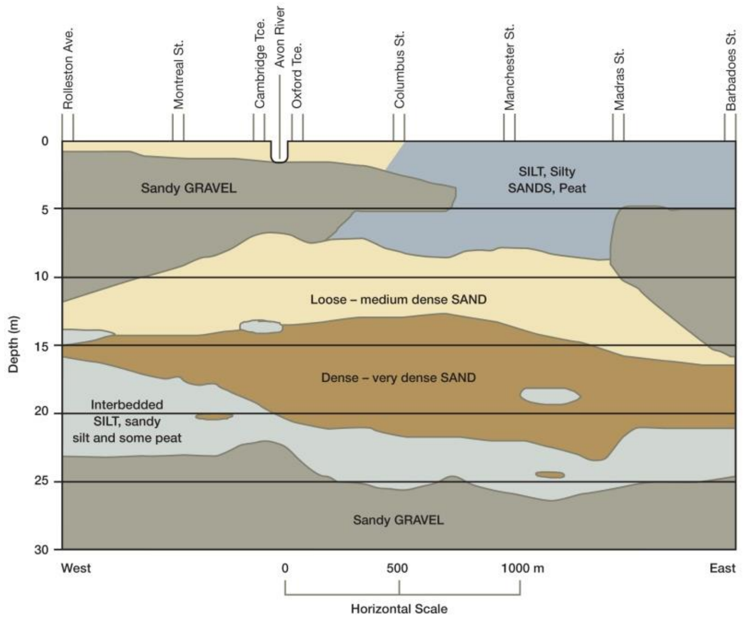


Figure 3.1 Horizontal and vertical soil variability, Christchurch, New Zealand (Royal Commission of Enquiry Report)

Site investigation is a component of an overall ground investigation. The ground investigation contains many components and includes; a desk study, site investigation, field and laboratory tests. For this course we concentrate on the main components of a site investigation performed for pile design purposes. Given pile design in the Netherlands is predominantly based on the results of Cone Penetration Test, CPT the operation and interpretation of CPT tests will be our focus.

3.2 Historical Development of Cone Penetration Testing[1]

The history of the Cone Penetration Test, CPT is described by (reference) who reports the test was invented by Pieter Barensten from Rijkwaterstaat in 1930. The original purpose was to measure the thickness and bearing capacity of a hydraulic fill deposit in Vlaardingen using a cone inserted into the ground, See Figure 3.2a. The concept was adapted by Delft Laboratory of Soil Mechanics (DLSM, now Deltares) in 1932 who introduced a dead-weight loading system, See Figure 3.2b that allowed a reaction force of up to 100 kN to be provided that allowed for application of a smooth pushing force during penetration. The 60° cone with a diameter of 35 mm was pushed through an outer casing pipe, Figure 3.2c.

[1] Based on content Robertson and Cabal (2014) and Lo Presti and Meisina (2020)

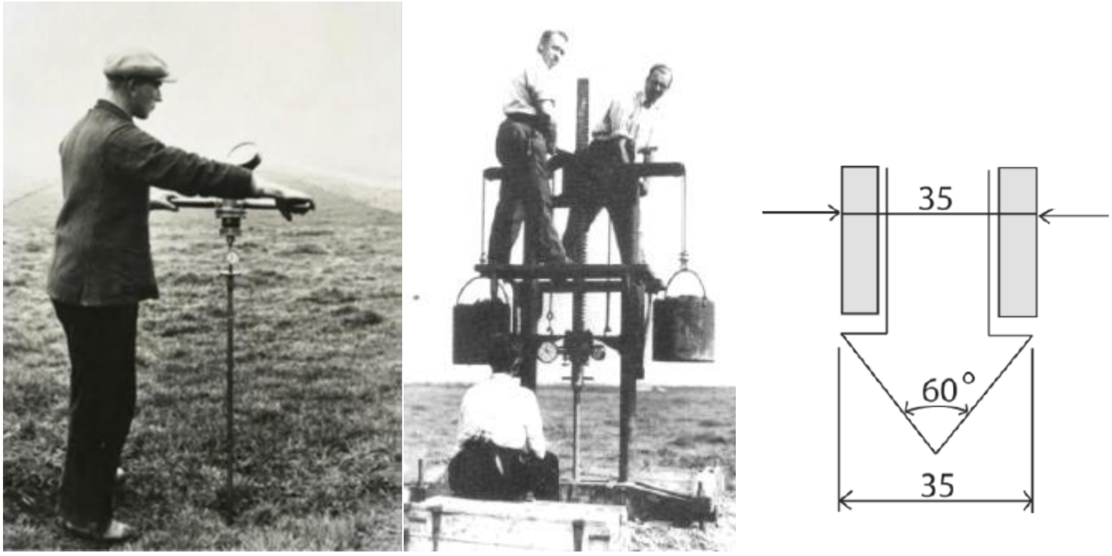


Figure 3.2 (a)The first CPT (1930), (b) DLSM Loading system and (c) Cone Geometry (images from Robertson and Cabal 2014)

In order to increase the maximum depth that could be penetrated and provide a smooth pushing force of up to 100kN the deep CPT was developed by Delft Laboratory of Soil Mechanics in1932. The cone was pushed through an outer casing pipe. Vermeiden (1948) improved this design through the addition of a sleeve to prevent soil ingress between the cone and casing. Begemann (1953) added a separate friction sleeve and recorded the first separate measurements of end resistance (q_c) and sleeve friction (f_s) at 20cm intervals. The usefulness of the friction ratio, $F_r = f_s/q_c$ %, as a method of classifying soils (identifying soil type) was immediately identified, See Figure 3.3 whilst Begemann (1965) already used the results directly to derive the load resistance of piles.

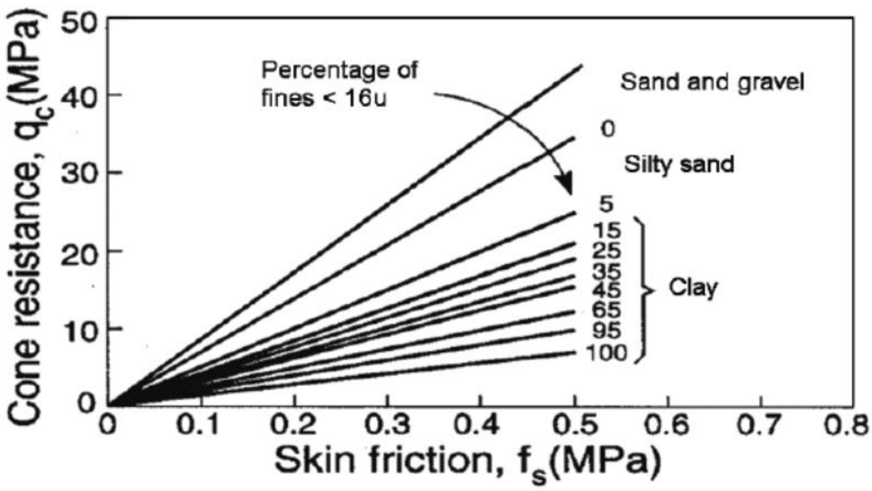


Figure 3.3 Use of cone end resistance and friction sleeve measurements to classify soil (Begemann 1965)

The electric cone was developed by Fugro in 1965. The advantages over the mechanical cone were; continuous penetration, reliable measurements and elimination of friction and rod weight error. A significant advance was achieved in 1974 with the introduction of a pore pressure probe near the pile tip, See Figure 3.4. This allows the excess pore pressure to be measured during installation providing the ability to identify the drainage state of the soil (undrained vs drained response) at the penetration rate of the penetrometer, 2cm/sec.

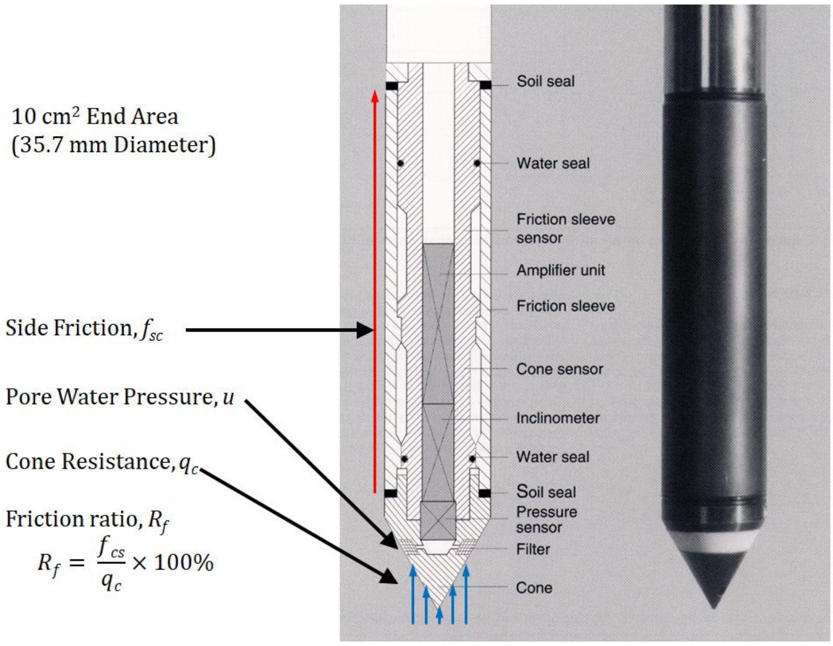


Figure 3.4 Details of the Fugro (1965) electric cone with pore pressure sensor at tip

In recent years a number of new sensors have been added, these include geophones capable of measuring the shear wave velocity of the soil that can be related to the small stress shear stiffness, G_0, See Figure 3.5. Additional sensors have been added for temparture, conductivity etc.

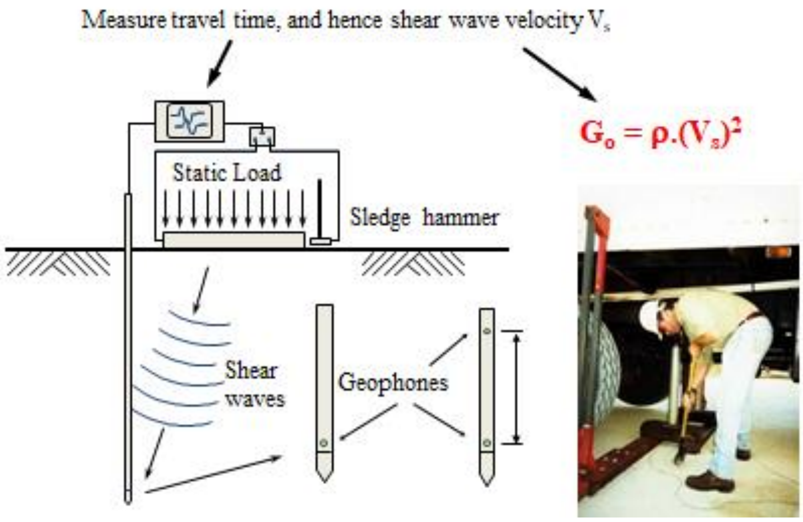
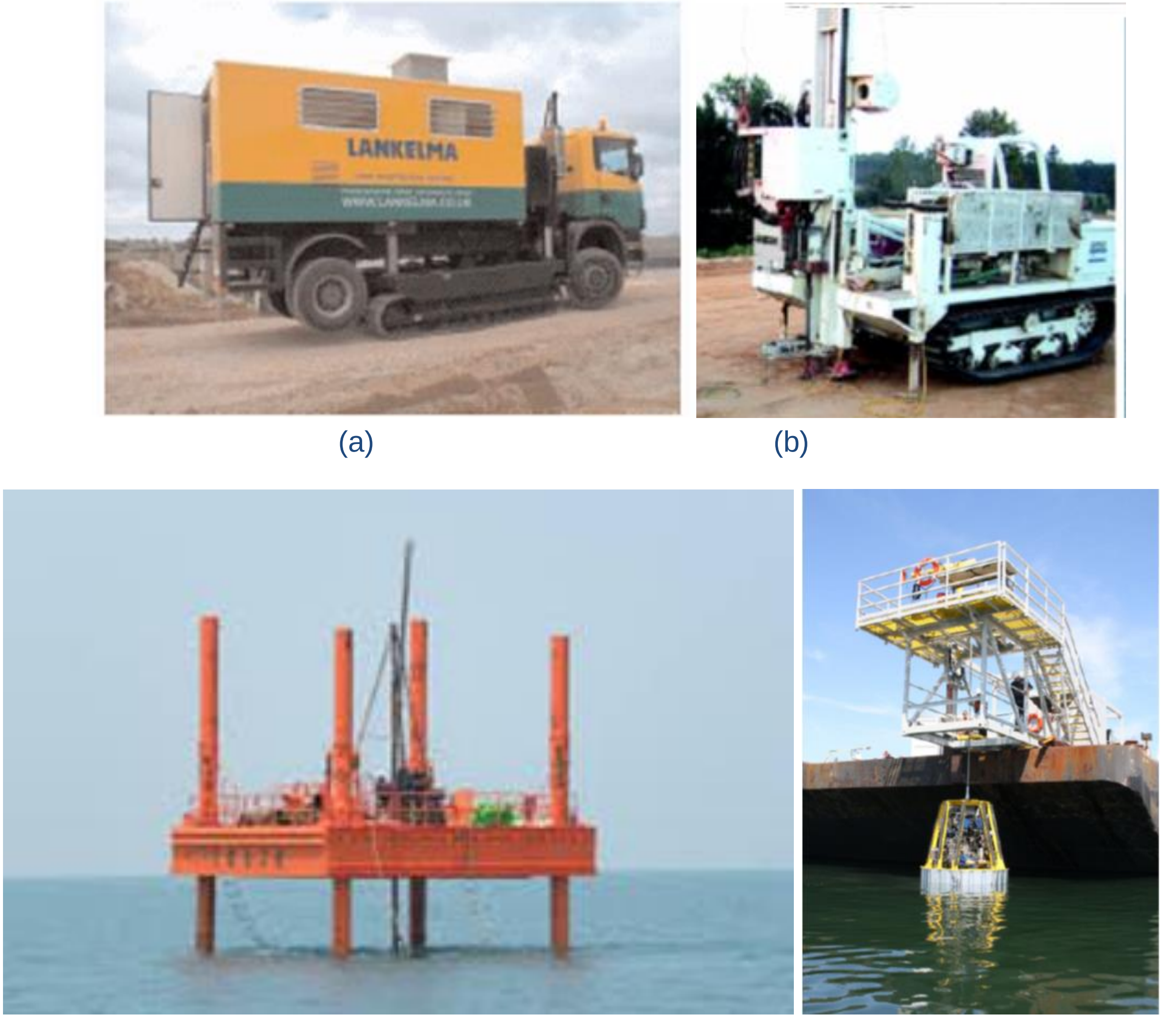


Figure 3.5 The use of geophones to infer shear modulus, G_0

Modern CPTs are undertaken using truck-mounted systems, Figure 3.6a with a normal ballasted weight of 20 tonnes. For sites with soft near surface soils, lighter, track-mounted rigs can be provided, See Figure 3.6b or even with smaller rigs for in-house operations. For near shore and offshore investigations in shallow to medium water depths, jack-up rigs can be used, Figure 3.6c. These involve penetrating four legs into the sea bed and performing a normal CPT test with casings into the river or sea bed. For deeper water sites sea-bed mounted systems can be deployed, See Figure 3.6d.



(a) (b)

(c) (d)

Figure 3.6 (a) Standard CPT rig, (b) Light-weight CPT rig, (c) Jack-up rig (d) sea-bed mounted rig

3.3 Terminology, correction factors and normalisations for CPT results

During installation the cone is pushed into ground at 1-2 cm/sec and the force acting on the cone, Q_c the pore pressure, u and the friction sleeve resistance, f_s is recorded. The force Q_c divided by the area of the cone, A_c gives the cone resistance, q_c. See Figure 3.7. The usual location for measurement of the porewater pressure is just behind the cone tip is called, u_2.

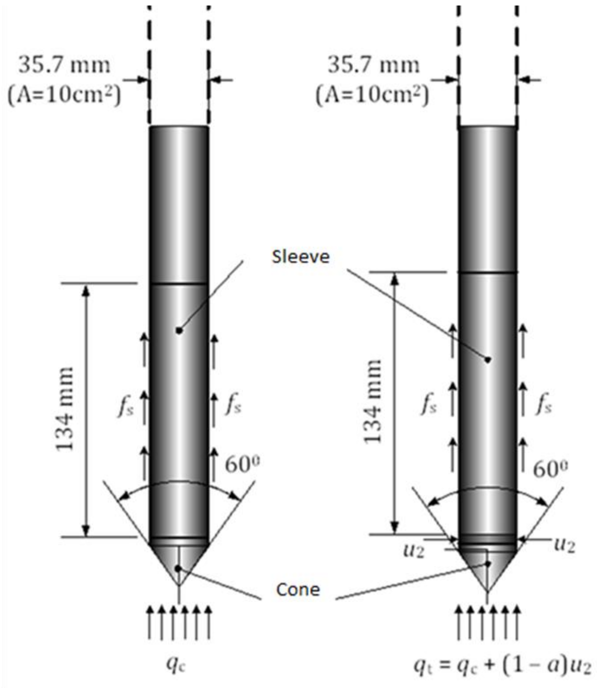


Figure 3.7 Definition of terms related to CPT installation measurements

In soft clay and silt, the measured q_c value should be corrected for pore pressure effects using a correction factor a, determined for the individual cone using a laboratory calibration procedure:

$$q_t = q_c + u_2\,(1-a) \qquad [3.1]$$

Where a, the correction factor is typically between 0.70 and 0.85. In sandy soils $q_t = q_c$.

In some applications for example in soil profiling it is necessary to correct the measured cone resistance for the effects of vertical total stress, σ_{vo} giving the net cone resistance, q_n:

$$q_n = q_t - \sigma_{vo} \qquad [3.2]$$

In some applications it is useful to have the cone resistance, Q_t in a non-dimensional form:

$$Q_t = (q_t - \sigma_{vo})/\ \sigma'_{vo} \qquad [3.3]$$

Where σ'_{vo} is the effective vertical stress.

A more accurate version of Eqn. 3.3 includes an exponent n which varies with stress level and soil type:

$$Q_{tn} = \left(\frac{q_t - \sigma_{vo}}{P_{atm}}\right)\left(\frac{P_{atm}}{\sigma'_{vo}}\right)^n \qquad [3.4]$$

(n = 1 Clay, 0.75 Silt and 0.5 Sand and stress reference level P_{atm} = 100 kPa)

The friction ratio (of sleeve to end resistance), F_r is given by:

$$F_r\ (\%) = f_s/q_t \times 100 \qquad [3.5]$$

Generally coarse grained soils have higher q_c values and lower F_r values than fine grained soils.

Some factors associated with pore pressure measurements are, the excess pore pressure due to cone installation, Δ_u:

$$\Delta_u = u_2 - u_0 \qquad [3.6]$$

where u_0 is the equilibrium pore water pressure.

The excess pore pressure normalised by the net cone resistance gives the pore pressure ratio, B_q:

$$B_q = \Delta_u/q_n \qquad [3.7]$$

3.4 CPT interpretation

The CPT test has gained worldwide application because of the many advantages it has when compared to other in-situ test techniques, these include;

Soil layering

Automatic logging of q_c, f_s and u with depth provide large quantities of high-quality data with which to describe the vertical variability of a soil profile. As an example compare the q_c profile in Figure 3.8 with the more coarser data recorded using Standard Penetration Test in which the number of blows to drive a hammer 300mm is recorded. The additional refinement demonstrated by the CPT is clear with the SPT test having difficulty differentiating the soil layers.

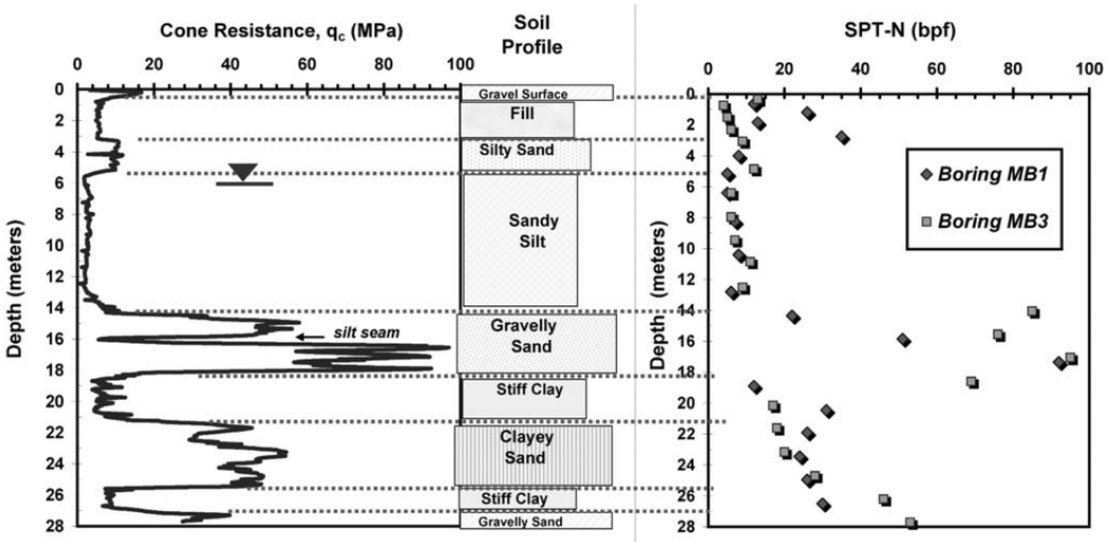


Figure 3.8 Comparison of CPT and Stantdard Penetration Test, SPT N profile measured at the same site, NB. SPT N measured as blows per foot.

The advantage of constant measurements of pore pressures are clear from Figure 3.9 where it can be seen that the pore pressure measured u_2 remains almost equal to the equilibrium pore water pressure, u_0 for the first 11m of penetration. Thus the cone is penetrating in a drained manner and the material is free draining (i.e. a sand or gravel). Between 11m and ≈ 20m below ground level significant excess pore pressure Δ_u develop due to cone installation, indicating undrained conditions and the presence of clays or silt (i.e. soils with low permeability). The cone then passes through a thin layer of free-draining soil before entering a deeper later of low permeability. This gives the designer an indication of when undrained and drained analyses should be performed. An indication of the permeability of the layers can be obtained by pausing penetration at any depth and measuring the rate of pore pressure dissipation with time.

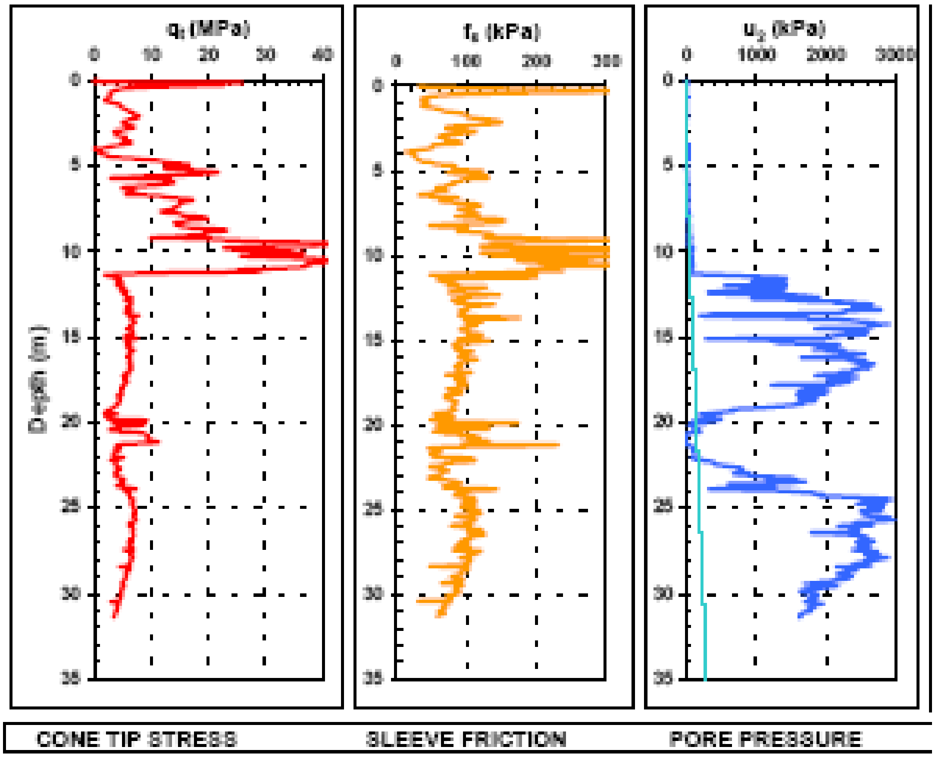


Figure 3.9 Profiles of q_c, f_s and u_2 measured during CPT installation (Data from Paul Mayne 2001)

Excess pore pressure measurements also provide information of the geological history of clay soils. Compare the CPT profiles for two UK Clays in Figure 3.10. The over-consolidated (OC) clay from Brent Cross on the left has:

- Much higher strength as evidenced by the q_c value of 1 to 3 MPa as compared to 0.2 to 0.8 MPa value at Bothkennar.
- In normally consolidated soil q_c generally increases approximately linearly with depth in a given soil layer.
- In OC soils q_c can remain relatively constant or increase gradually with depth in a layer.
- In these profiles we have two pore pressure measurements. The usual u_2 value and another, u_1 value measured using a filter paced on the cone face, See Figure 3.10. For the OC clay (left side of figure) positive pore pressure is developed at the u_1 position,

it is clear that in the NC clay the relative value of this (u_1/q_c) is much higher, with the pore pressure approaching the q_c value in the NC clay.

- The response of the u_2 sensor is even more illustrative, for the OC soil the pore pressure becomes negative as the soil wants to dilate (increase in volume) during shear and the low permeability presents volume change (undrained penetration occurring).

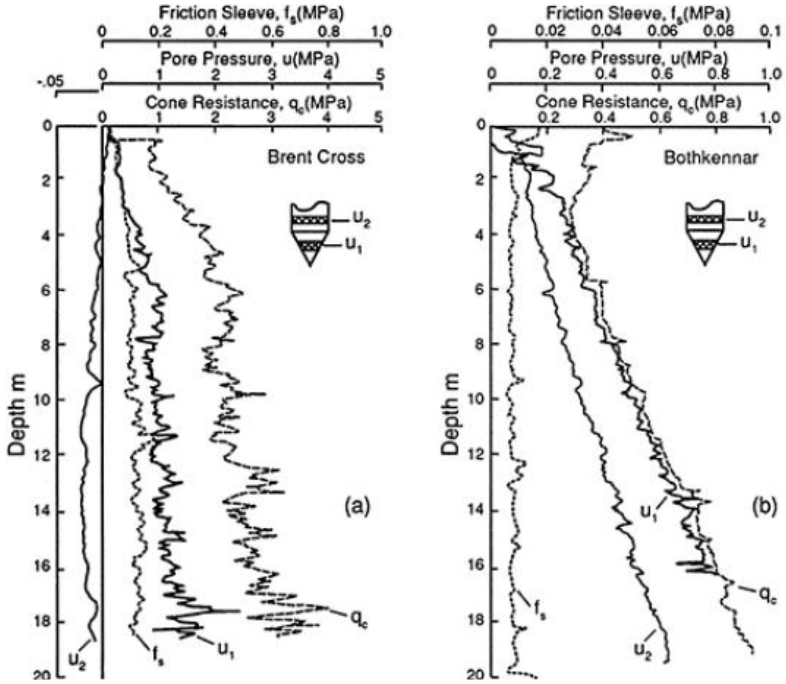


Figure 3.10 Comparison of CPT profiles in an over-consolidated and normally consolidated clay layer (from Lunne et al. 1997)

Soil Classification

The friction ratio, F_r (%) = f_s/q_c See Figure 3.11 is a useful indicator of soil type, with low values indicative of sand and gravels and higher values being indicative of fine grained soils, silts and clays. In typical Dutch soils we expect $F_r \approx 1\%$ in Sand, 3 to 5% in Clay and 8 to 10% in Peat.

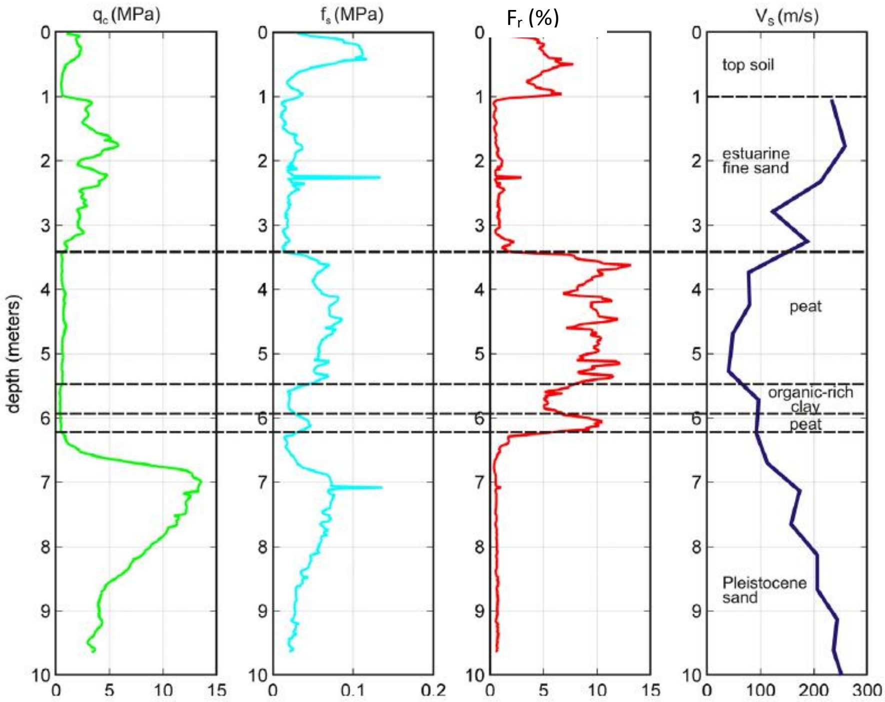


Figure 3.11 Typical CPT profile from the Western Netherlands

This feature has been used to develop classifications charts, see Figure 3.12 as an example. These are based on the following general observations:

- Cone tip resistance in highest in coarse grained soil (gravel and sand) and decrease with fines content)
- Sleeve resistance is relatively lower in sand and increases with fine content
- Pore pressures are lower in sand and increase with fines content

The charts are not applicable in chalk, some glacial soils, unsaturated soils and some fills.

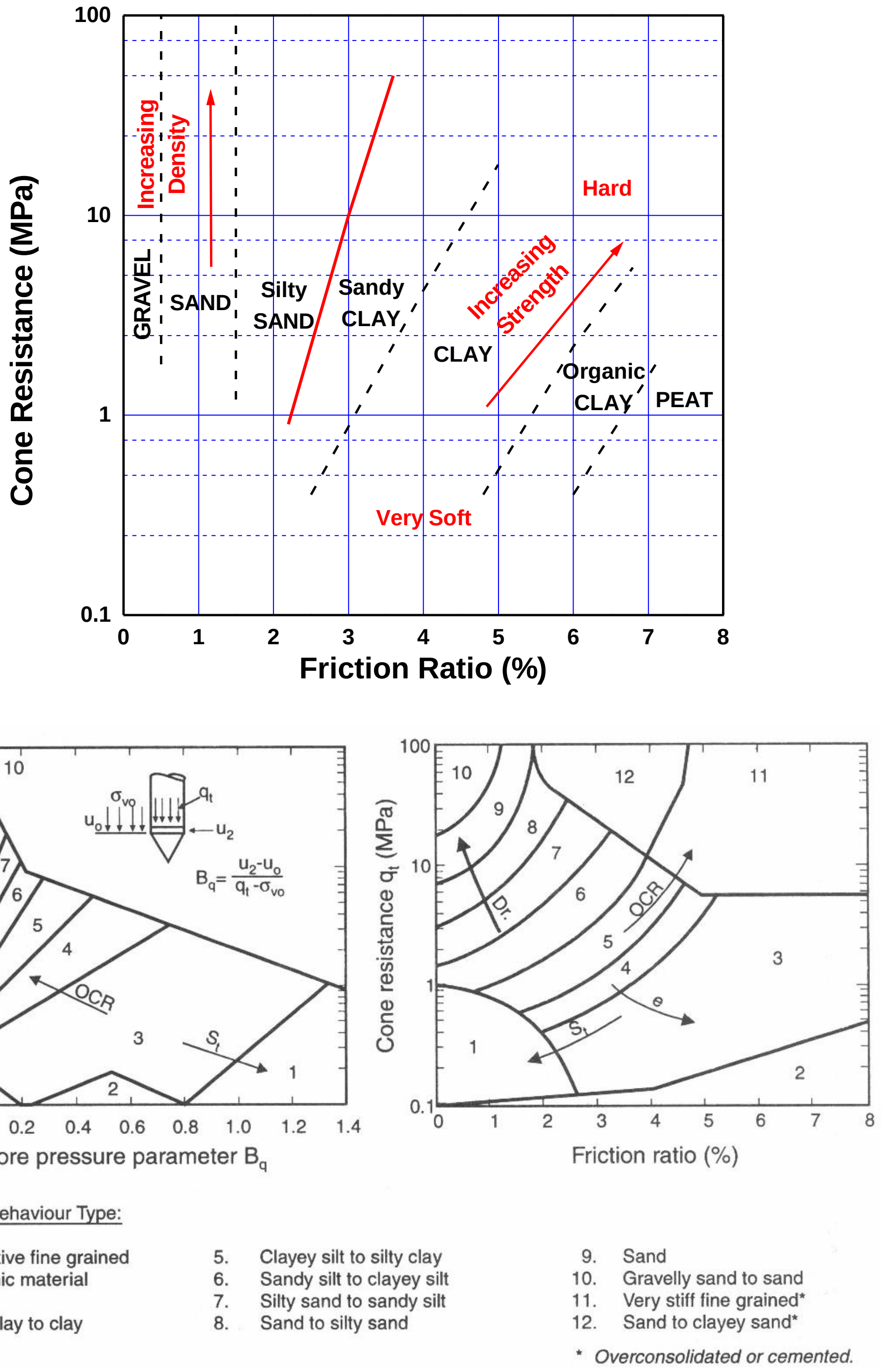


Figure 3.12 (a) Simplified CPT Soil Classification Chart (b) Advanced version using pore pressure data (after Robertson 1990)

Given that the cone resistance is dependent on the mechanical behaviour of soil, e.g. strength, stiffness compressibility and drainage Robertson (1990 and 2009) suggested that the charts were predictive of soil behaviour type, SBT (i.e. they give information on how the soil responds to loading). This led to the development of soil behaviour type index, I_c which defines boundaries between behaviour types, See Figure 3.13.

$$I_c = \sqrt{(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2} \qquad [3.8]$$

In Figure 3.13 Q_{tn} is evaluated using Eqn 3.4 where initially, an exponent n = 1 is used to calculate the starting value of I_c (i.e., $Q_{tn} = Q_t$) and then the exponent is upgraded to:

$$n = 0.381 \cdot I_c + 0.05 \cdot (\sigma'_{vo}/P_{atm}) - 0.15 \qquad [3.9]$$

where $n \leq 1.0$

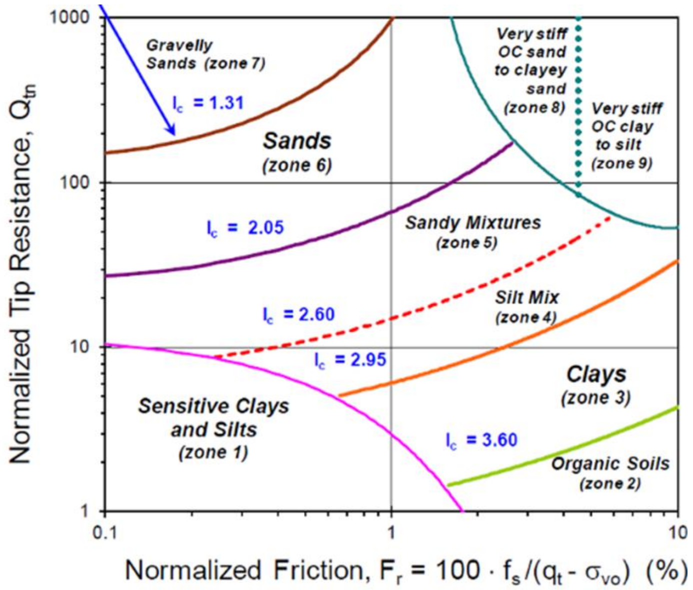


Figure 3.13 Soil behaviour type and soil behaviour type index

Example 3.1 on determination of soil behaviour type index, I_c

Results from CPT tests in Rotterdam are shown in the table. Determine the soil type and behaviour type index, I_c at 2, 5 and 10m depth. The water table is 4m below ground level. Assume γ_{sat} = 20kN/m^3, γ_{dry} = 18kN/m^3, γ_w = 10kN/m^3, P_{atm} = 100 kPa and n = 1 (a = 0.85).

z (m)	q_c (kPa)	u_2 (kPa)	σ_{v0} (kPa)	U_0 (kPa)	$\sigma_{v0'}$ (kPa)	q_t (kPa)	Q_{tn}	F_r (%)	I_c
2	8,500	0		0				1	
5	300	250		10				4	
10	14500	60		60				1	

At 2m depth

σ_{vo} = 18 x 2 = 36 kPa, u_o = 0, σ'_{vo} = 36 - 0 = 36 kPa

Eqn 3.1 $\quad q_t = q_c + u_2\,(1\text{-}a) = 8500 + 0\,(1\text{-}0.85) = 8500$ kPa

Eqn 3.4 $\quad Q_{tn} = \left(\frac{q_t - \sigma_{vo}}{P_{atm}}\right)\left(\frac{P_{atm}}{\sigma'_{vo}}\right)^n \; Q_{tn} = \left(\frac{8500-36}{100}\right)\left(\frac{100}{36}\right)^{0.5} = 141$

Eqn 3.8 $\quad Ic = \sqrt{(3.47\text{-}\log Q_{tn})^2 + (\log F_r + 1.22)^2} = 1.80$

When I_c = 1.80, Material is Sand See Figure 3.13

At 5m depth

σ_{vo} = (18 x 4) + (20 x 1) = 92 kPa, u_o = 10, σ'_{vo} = 92 - 10 = 82 kPa

Eqn 3.1 $q_t = q_c + u_2\,(1\text{-}a)$ = 300 + 250 (1-0.85) = 338 kPa

Eqn 3.4 $Q_{tn} = \left(\frac{q_t - \sigma_{vo}}{P_{atm}}\right)\left(\frac{P_{atm}}{\sigma'_{vo}}\right)^n$ $Q_{tn} = \left(\frac{338-92}{100}\right)\left(\frac{100}{82}\right)^1 = 3.0$

Eqn 3.8 $Ic = \sqrt{(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2} = 3.5$

When I_c = 3.5, Material is Clay See Figure 3.13

At 10m depth

σ_{vo} = (18 x 4) + (20 x 6) = 192 kPa, u_o = 60, σ'_{vo} = 192 - 60 = 132 kPa

Eqn 3.1 $q_t = q_c + u_2\,(1\text{-}a)$ = 14500 + 60 (1-0.85) = 14509 kPa

Eqn 3.4 $Q_{tn} = \left(\frac{q_t - \sigma_{vo}}{P_{atm}}\right)\left(\frac{P_{atm}}{\sigma'_{vo}}\right)^n$ $Q_{tn} = \left(\frac{14509-192}{100}\right)\left(\frac{100}{132}\right)^{0.5} = 125$

Eqn 3.8 $Ic = \sqrt{(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2} = 1.84$

When I_c = 1.84, Material is Sand See Figure 3.13

z (m)	q_c (kPa)	u_2 (kPa)	σ_{v0} (kPa)	u_0 (kPa)	$\sigma_{v0'}$ (kPa)	q_t (kPa)	Q_{tn}	F_r (%)	I_c
2	8,500	0	36	0	36	8,500	141	1	1.80
5	300	250	92	10	82	338	3.0	4	3.50
10	14500	60	192	60	132	14509	125	1	1.84

Determination of Soil Properties from CPT results

Unit Weight

The unit weight, γ of soil is vital for our calculation of in-situ stress conditions and can be estimated from Figure 3.14 or by using Eqn 3.10.

$$\gamma/\gamma_w = 1.22 + 0.15 \cdot \ln(100*f_s/P_{atm}+0.01) \qquad [3.10]$$

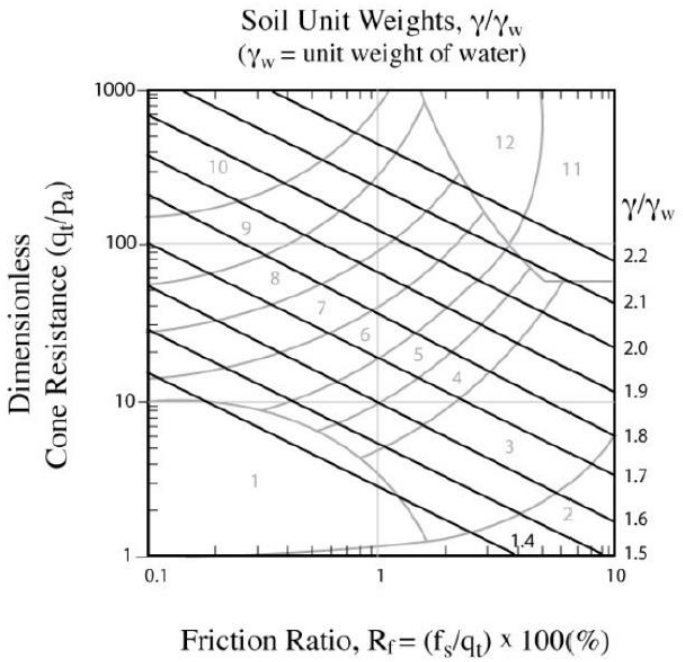


Figure 3.14 Estimate of the unit weight of soil (Robertson and Cabal 2014)

Relative Density

The relative density, D_r of soil is useful in design as it is a measure of the compactness of cohesionless soils such as sand and gravel. D_r can be estimated from CPT using the Eqn. 3.11. Note if the soil is a normally consolidated recent deposit, λ = 1, if an aged, over-consolidated deposit, λ = 2/3.

$$D_r = \frac{1}{2.91} \ln\left(\frac{\lambda\, q_c}{60\ {\sigma'_v}^{0.7}}\right) \qquad [3.11]$$

Friction angle

The peak friction angle can be estimated using Eqn 3.12.

$$\phi'_p\,^\circ = 17.6 + 11 \log Q_{tn} \qquad [3.12]$$

Undrained strength

The undrained shear strength, s_u of fine grained soils (clay and silt) can be estimated from Eqn 3.13.

$$s_u = \frac{q_t - \sigma_v}{N_k} \qquad [3.13]$$

Where: N_k is an empirical cone factor that should be correlated to lab measurements of s_u, normally in the range 10-20. A value of 15 gives a good rough approximation.

Horizontal Stress

Whilst it is relatively straight-forward to calculate the vertical effective stress σ'_v, in the ground, the shaft resistance of piles is in fact controlled by the horizontal stresses acting on the pile σ'_h. These stresses are linked through the earth pressure coefficient at rest, $K_0 = \sigma'_h/\sigma'_v$. For normally consolidated soil we can estimate K_o if we know ϕ'_p:

$$K_0 = (1-\sin\phi'_p) \qquad [3.14]$$

For over-consolidated soils we need to know the maximum past stress experienced by the soil (pre-consolidation pressure, σ'_p), in order to calculate the over-consolidation ratio (OCR= σ'_p/σ'_v):

$$K_0 = (1-\sin\phi'_p)\,OCR^{\sin\phi'p} \qquad [3.15]$$

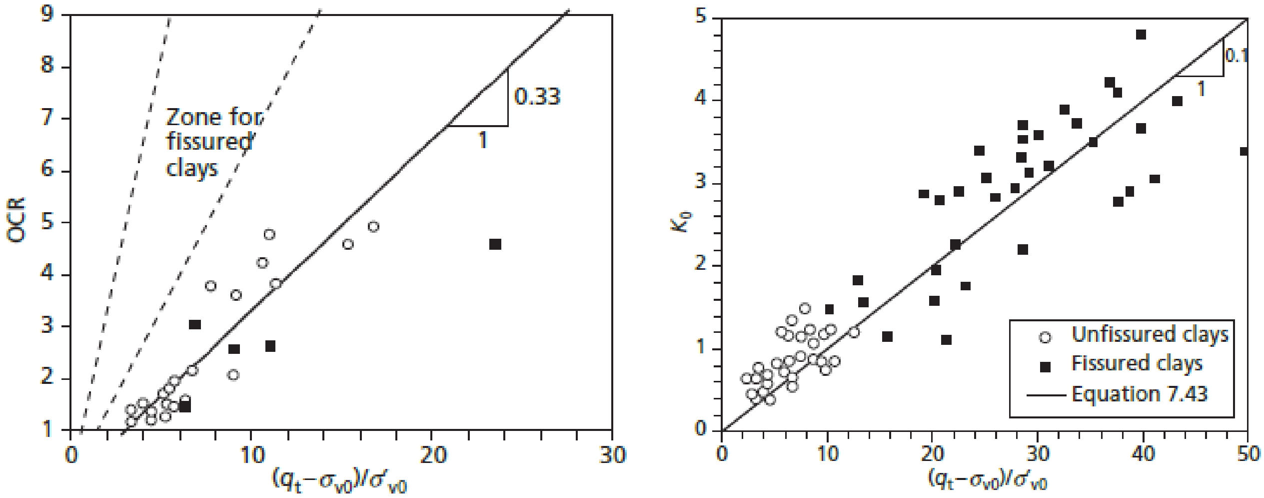


Figure 3.15 Estimate of (a) OCR and (b) K_0 for clay

We can estimate the parameters using the following expressions for clay:

$$\sigma'_p = 0.33\,(q_t - \sigma_{vo}) \qquad [3.16]$$

$$OCR = \sigma'_p / \sigma'_v \qquad [3.17]$$

$$K_0 = 0.1\left(\frac{q_t - \sigma_{vo}}{\sigma'_{vo}}\right) \qquad [3.18]$$

And for sand:

$$\sigma'_p = 0.32\, q_t^{0.7} \qquad [3.19]$$

Permeability

As noted during CPT testing when penetrating through fine grained soils where excess pore pressures are developed, it is possible to pause installation in order to monitor the dissipation of excess pore water pressure, See Figure 3.16. Good correlation between the time, t_{50} to dissipation of 50% of the excess pore pressure and the permeability (hydraulic conductivity) of soil is observed over the range of soil types commonly encountered.

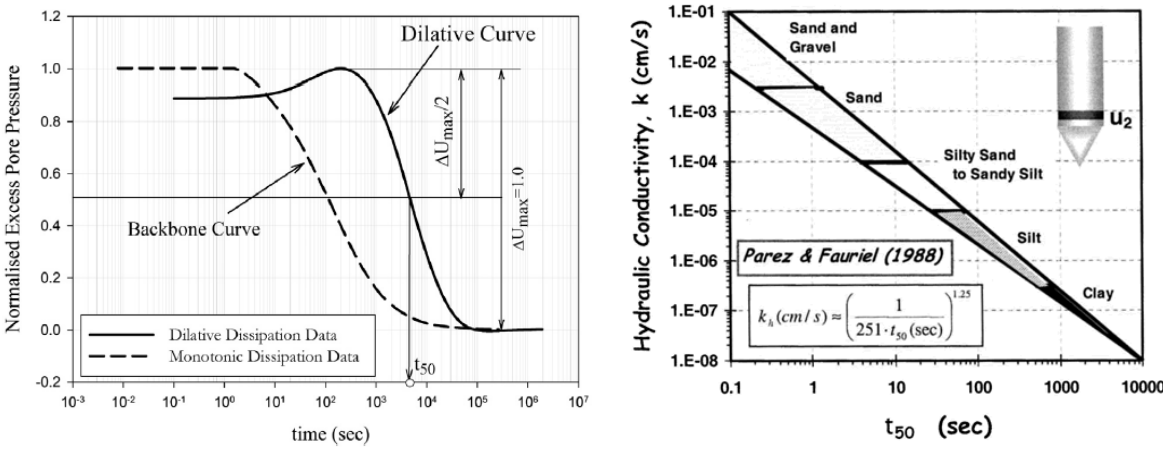


Figure 3.16 (a) Dissipation of excess pore water pressure with time and (b) variation of permeability (hydraulic conductivity) with t_{50}, time for 50% dissipation of the excess porewater pressure

In addition to direct measurement we can correlate the permeability with SBT and I_c data to give an approximation as shown in the table below and using the following equations:

$$When\ 1.0 \le I_c \le 3.27 \quad k = 10^{(0.952-3.04\, I_c)}$$

$$When\ 3.27 \le I_c \le 4.0 \quad k = 10^{(-4.52-1.37\, I_c)} \qquad [3.20]$$

SBT Zone	SBT	Range of k (m/s)	SBT_n I_c
1	Sensitive fine-grained	$3x10^{-10}$ to $3x10^{-8}$	*NA*
2	Organic soils - clay	$1x10^{-10}$ to $1x10^{-8}$	$I_c > 3.60$
3	Clay	$1x10^{-10}$ to $1x10^{-9}$	$2.95 < I_c < 3.60$
4	Silt mixture	$3x10^{-9}$ to $1x10^{-7}$	$2.60 < I_c < 2.95$
5	Sand mixture	$1x10^{-7}$ to $1x10^{-5}$	$2.05 < I_c < 2.60$
6	Sand	$1x10^{-5}$ to $1x10^{-3}$	$1.31 < I_c < 2.05$
7	Dense sand to gravelly sand	$1x10^{-3}$ to 1	$I_c < 1.31$
8	*Very dense/ stiff soil	$1x10^{-8}$ to $1x10^{-3}$	*NA*
9	*Very stiff fine-grained soil	$1x10^{-9}$ to $1x10^{-7}$	*NA*

**Overconsolidated and/or cemented*

Example 3.2 Application of empirical equations to estimate soil properties

Using the data from Exercise 1:

1. Estimate the unit weight, pre-consolidation pressure and horizontal effective stress at depths of 2, 5 and 10m below ground level, bgl.
2. Calculate the relative density and friction angle at depths of 2 and 10m
3. Calculate the s_u value and estimate the permeability at 5m bgl

z (m)	q_t (kPa)	σ_{v0} (kPa)	σ'_{v0} (kPa)	Q_{tn}	F_r (%)	I_c	γ kN/m³	σ'_p (kPa)	σ'_h (kPa)	D_r (%)	ϕ	s_u (kPa)	k m/sec
2	8,500	36	36	141	1	1.80							
5	338	92	82	3.0	4	3.5							
10	14500	192	132	125	1	1.84							

At 2m depth, layer is sand:

Unit weight Eqn. 3.10 $\gamma/\gamma_w = 1.22 + 0.15 \cdot \ln(100^* f_s/P_{atm}+0.01)$

$$\gamma/\gamma_w = 1.22 + 0.15 \cdot \ln(100^*0.01^*8500/100)+0.01)$$

$$\gamma = 1.89 \times 10 = 18.9 \text{ kN/m}^3$$

Pre-consolidation Pressure Eqn. 3.19

$$\sigma'_p = 0.32\, q_t^{0.7}$$

$$= 0.32 \times 8500^{0.7}$$

$$= 180 \text{ kPa}$$

Horizontal effective stress, we know $\sigma'_h = K_0\sigma'_v$ and $K_0 = (1-\sin\phi'_p)\ OCR^{\sin\phi'p}$

we need to know friction angle, from Eqn 3.12

$$\phi'_p{}^\circ = 17.6 + 11 \log 141$$

$$\phi'_p{}^\circ = 41^\circ$$

we need to know K_0 for over-consolidated soil, from Eqn 3.15 and Eqn 3.17

$$K_0 = (1-\sin 41)OCR^{\sin\phi'}$$

$$K_0 = (0.344)(180/36)^{0.656}$$

$$K_0 = 0.99$$

$$\sigma'_h = 0.99 \times 36 = 36 \text{ kPa}$$

Relative Density from Eqn 3.11

$$D_r = \frac{1}{2.91}\ln\left(\frac{\frac{2}{3}x8500}{60\ x\ 36^{0.7}}\right) = 0.70 \text{ or } 70\%$$

At 5m depth, layer is clay:

Unit weight Eqn. 3.10 γ/γ_w = 1.22 + 0.15 · ln (100*f_s/P_{atm}+0.01)

γ/γ_w = 1.22 + 0.15 · ln (100*0.04*338/100)+0.01)

γ = 1.16 x 10 = 16.1 kN/m^3

Pre-consolidation Pressure Eqn. 3.16

$\sigma'_p = 0.33\ (q_t - \sigma_{vo})$

$\sigma'_p = 0.33\ (338 - 92)$

= 69 kPa

Horizontal effective stress, we know $\sigma'_h = K_0\sigma'_v$, for Clay K_0 from Eqn 3.18

$$K_0 = 0.1\left(\frac{q_t - \sigma_{vo}}{\sigma'_{vo}}\right)$$

$$K_0 = 0.1\left(\frac{338 - 92}{82}\right)$$

K_0 = 0.25

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

σ'_h = 0.25 x 82 = 21 kPa

Undrained shear strength s_u from Eqn 3.13

$$s_u = \frac{q_t - \sigma_v}{N_k}$$

$$s_u = \frac{338 - 92}{15} \qquad = 16 \text{ kPa}$$

Permeability from Eqn 3.20

$k = 10^{(-4.52 - 1.37\, I_c)}$ $\quad I_c = 3.5$

$k = 10^{(-4.52 - (1.37\, x\, 3.5))}$

K = 1 x 10^{-9} m/sec

At 10m depth, layer is sand:

Unit weight Eqn. 3.10 γ/γ_w = 1.22 + 0.15 · ln (100*f_s/P_{atm}+0.01)

γ/γ_w = 1.22 + 0.15 · ln (100*0.01*14500/100)+0.01)

γ = 1.97 x 10 = 19.7 kN/m^3

Pre-consolidation Pressure Eqn. 3.19

$\sigma'_p = 0.32\ q_t^{0.7}$

$= 0.32 \times 14500^{0.7}$

= 262 kPa

Horizontal effective stress, we know $\sigma'_h = K_0\sigma'_v$ and $K_0 = (1-\sin\phi'_p)OCR^{\sin\phi'p}$

we need to know friction angle, from Eqn 3.12

$$\phi'_p{}^\circ = 17.6 + 11 \log 125$$

$$\phi'_p{}^\circ = 41^\circ$$

we need to know K_0 for over-consolidated soil, from Eqn 3.15

$$K_0 = (1\text{-}\sin 41)OCR^{\sin\phi'p}$$

$$K_0 = (0.344)(262/132)^{0.656}$$

$$K_0 = 0.54$$

$$\sigma'_h = 0.54 \times 132 = 72 \text{ kPa}$$

Relative Density from Eqn 3.11

$$D_r = \frac{1}{2.91} \ln\left(\frac{\frac{2}{3}x14500}{60\ x\ 132^{0.7}}\right) = 0.57 \text{ or } 57\%$$

z (m)	q_t (kPa)	σ_{v0} (kPa)	σ'_{v0} (kPa)	Q_{tn}	F_r (%)	I_c	γ kN/m^3	σ'_p (kPa)	σ'_h (kPa)	D_r (%)	ϕ	s_u (kPa)	k m/sec
2	8,500	36	36	141	1	1.80	18.9	180	36	70	41		
5	338	92	82	3.00	4	3.5	16.1	69	21			16	$1x10^{-9}$
10	14500	192	132	125	1	1.84	19.7	262	72	57	41		

Practice Questions Section 3

Question 3.1

For the CPT (where a = 0.85) profile deposit shown:

(i) Estimate the unit weight at 10m and 30m below ground level, bgl.
(ii) Given the undrained strength measured in a triaxial test at 10m below ground level was 70 kPa, estimate the N_k value for the clay.
(iii) Estimate the strength and permeability at 30m bgl.
(iv) Estimate the over-consolidation ratio at 30 m bgl.

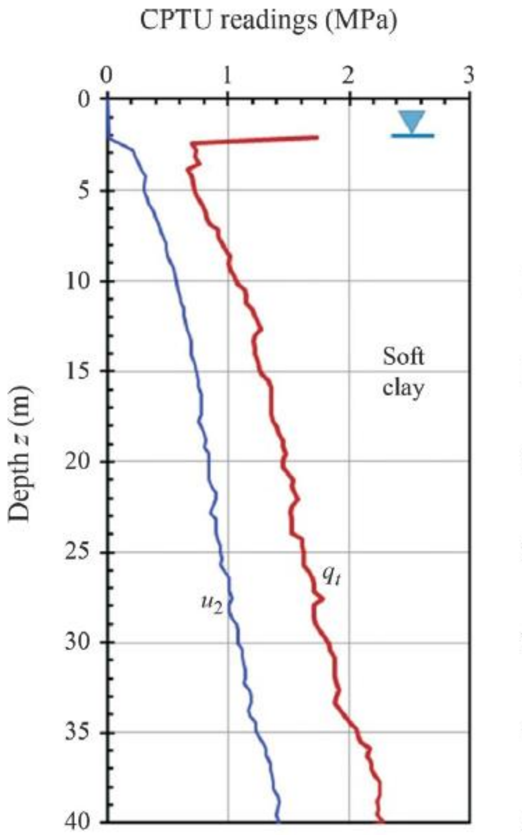


Figure 3.17 CPT Profile in Clay for Question 3.1

Question 3.2

For the soil deposit shown:

(i) Based on visual assessment only, estimate the likely soil type present from ground level to 11m below ground level, bgl and the second layer from 11m to 26m bgl.

(ii) Determine the SBT and I_c value at a depth of 5m and 15m bgl, given (given q_c and fs are 3500 and 20 kPa at 5m and 1000 and 10 kPa at 15m bgl respectively).

(iii) Estimate the relative density, friction angle and the horizontal stress at 5m bgl. (assume for this CPT a = 0.85)

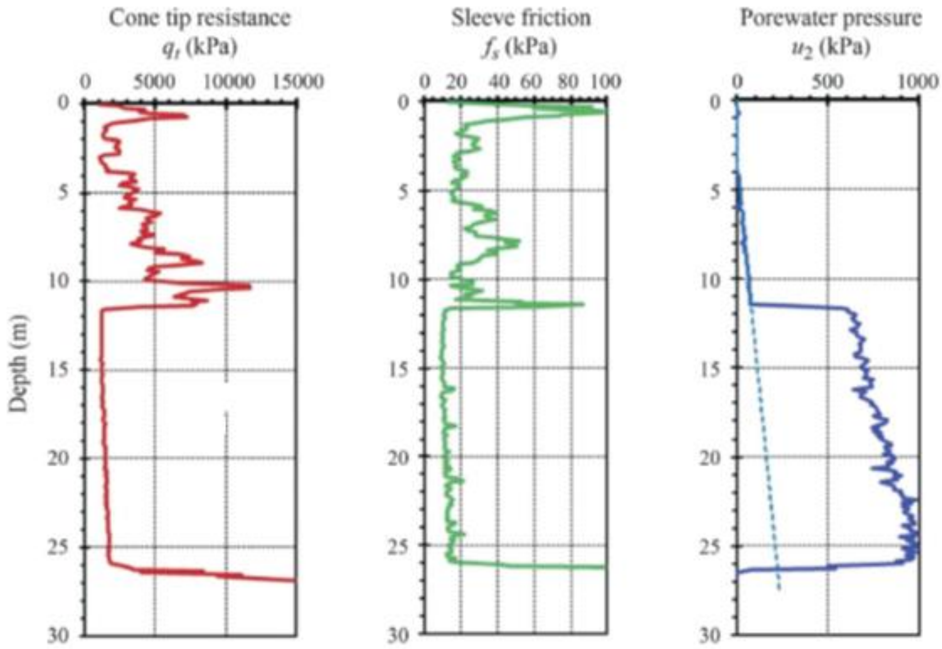


Figure 3.18 CPT Profile in Stratified Soil for Question 3.2

Question 3.3

A number of CPT profiles were made in a deposit of fine sand, see Figure below. Given the water table is at great depth, at depths of 5 and 10m below ground level estimate:

(i) The unit weight,
(ii) friction angle,
(iii) pre-consolidation stress and
(iv) K_o.

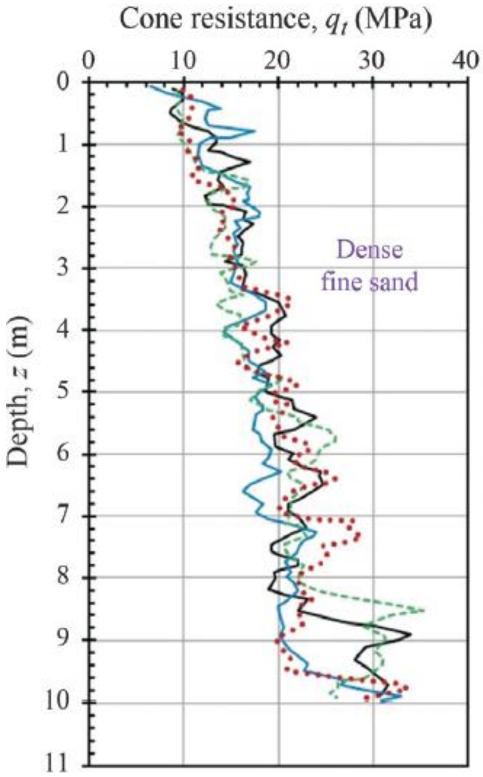


Figure 3.19 CPT profile in Stratified Soil for Question 3.

4.0 Axial Pile Design

4.1 Introduction

In this section we will consider the design of single piles to resist axial loads. Given that most piles installed in the Netherlands are founded in sand and are designed using CPT based approaches we will focus predominantly on this approach. We will also consider total stress and effective stress methods of design that are popular in other regions and applied widely in offshore practice.

In Figure 4.1 a pile with diameter, D is embedded length L in soil. When a load, Q is applied the pile develop side shaft shear resistance, τ_f and end bearing resistance, q_b.

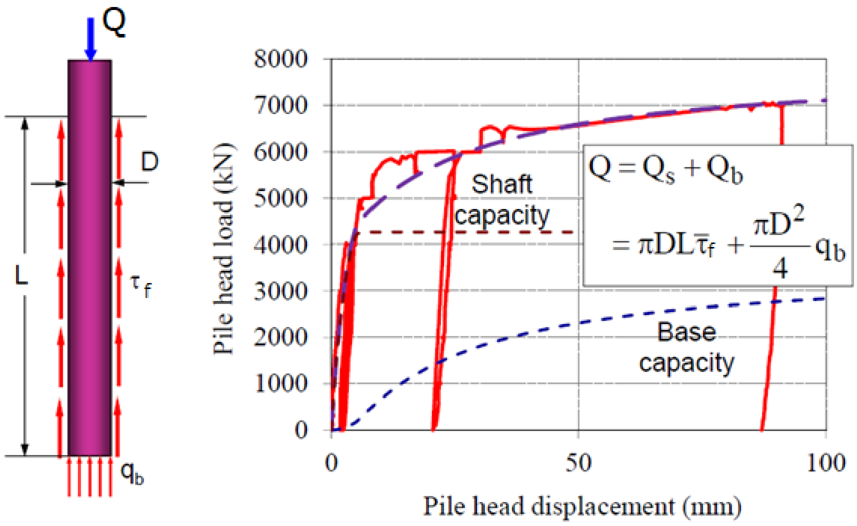


Figure 4.1 Resistance components for a 20m long, 800m diameter pile (image courtesy of Mark Randolph)

In this section we will consider how to estimate the shaft and base resistance and how the choice of pile type influences these values.

4.2 Axial capacity of displacement piles in sand using CPT

In the current Dutch code, a CPT based design method links the shaft and base resistance components directly to the cone end resistance, q_c measured during the CPT test using constant reduction factors α_s and α_p for the unit shaft, τ_f and base, q_b resistance respectively:

$$\tau_f = \alpha_s\, q_c \qquad [4.1]$$

$$q_b = \alpha_p\, q_c \qquad [4.2]$$

A range of constant α_s and α_p values for common pile types are given in Table 4.1. Note that α_p is designated as the correlation factor between q_b and q_c when the pile base settlement, s normalised by the equivalent pile diameter, D_{eq} is at a specified level, usually s/D_{eq} of 10% and 20% for displacement and replacement piles respectively. D_{eq} is the diameter of a circular pile with equal cross-section therefore:

For square piles $$D_{eq} = 1.13\, a_s \qquad [4.3]$$

For rectangular sections $D_{eq} = 1.13\ a_s \sqrt{a_L/a_s}$ [4.4]

where: a_s is the side length of a pile with square cross-section or the length of the smaller side of a rectangular pile and a_L is the length of the longer side or a rectangular pile.

Pile type			Pile class factor [a]			Load-settlement graphs
Type	**Typical specification**	**Installation method**	α_p	α_s	α_t	
Concrete-pile	Precast; with constant cross section	Driven	0.7	0.010	0.007	1
	Cast in situ with constant casing diameter and lost bottom plate, concrete in direct contact with surrounding soil.	Driven; the casing is removed by reverse driving in combination with static withdrawal from the ground; the bottom plate remains in the ground.	0.7	0.014	0.012[c]	1
	Cast in situ with constant casing diameter and lost bottom plate, concrete in direct contact with surrounding soil.	Driven; the casing is removed by vibrating the casing in combination with static withdrawl from the ground; the bottom plate remains in the ground.	0.7	0.012	0.010[c]	1
	Cast in situ with constant casing diameter and drilling tip, in which concrete is in direct contact with surrounding soil.	Screwed; during redrawal of the temporary casing the drilling tip remains in the soil.	0.63	0.009	0.009	1
	Cast in situ with Continuous-Flight-Auger.	Screwed	0.56	0.006	0.0045	2
	Cast in situ fluid stabilized excavation.	Excavated or bored	0.35	0.006	0.0045	3
Steel pile	Constant cross section; pipe pile closed ended [b]	Driven	0.7	0.010	0.007	1
	Constant cross section; steel profile or open ended pipe pile	Driven	0.7	0.006	0.004	1
	With cast in situ grout around the profile with foot plate	Driven; with grout injection	0.7	0.014	0.012	1
	Constant cross section above screw tip	Screwed	0.56	0.006	0.0045	1
	Cast in situ grout around the pipe pile with screw tip(shaft diameter ≥ 300 mm)	Screwed without alternating up - and down movements during installation; permanent in-situ soil mixing with grout	0.63	0.009	0.009	1
	Constant cross section	Cable tool drilled	0.35	0.005	-	3

[b] The foot plate of the close ended pipe pile cannot extend more than 10 mm from the pipe.
[c] For tension piles, the footplate cannot extend more than 25 mm from the casing .
[d] Values valid for micropiles with diameter < 200 mm for drilled systems and vibrated piles and <400 mm for self-boring and screwed systems.
[e] High values may be used if pile load tests are performed after pile installation, see [f] and [g].
[f] With pile load tests the value is 0.012.
[g] With pile load tests the value is 0.017.
[h] If the casing is not moved up and down over the deepest part of the pile (from 8x pile diameter above the tip to tip level) AND the grout is pressed in with high pressure at the tip AND the pile is fixed in place by screwing, the value is 0.9.

Table 4.1 NEN 9997 factors for shaft and base resistance for a range of pile types

Example 4.1 Calculate the total resistance of a precast concrete pile using CPT α factors

A 285mm square precast concrete pile is driven into 1a sand deposit with the q_c profile shown in Figure 4.2. Assume the average q_c value along the pile shaft is 5,000 kPa; the design q_c at base is 5,250 kPa.

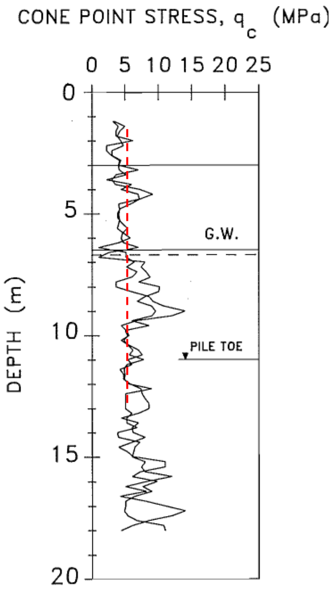


Figure 4.2 CPT Profile for precast concrete pile in Ex. 4.1

$$\tau_f = \alpha_s \, q_c$$

$$\tau_f \text{ (kPa)} = 0.01 \times 5{,}000 = 50 \text{ kPa}$$

$$Q_s \text{ (kN)} = 50 \times (4 \times 0.285 \times 11) = 627 \text{ kN}$$

$$q_b = \alpha_p \, q_c$$

$$q_b \text{ (kPa)} = 0.7 \times 5{,}250 = 3675 \text{ kPa}$$

$$Q_b \text{ (kN)} = 3675 \times (0.285 \times 0.285) = 298 \text{ kN}$$

$$Q = Q_s + Q_b$$

$$= 627+298$$

$$= 925 \text{ kN}$$

The measured load-displacement response from a compression load test performed on the pile is shown in Figure 4.3 where it appears that the predicted load of 927 kN was a reasonable estimate of the pile resistance.

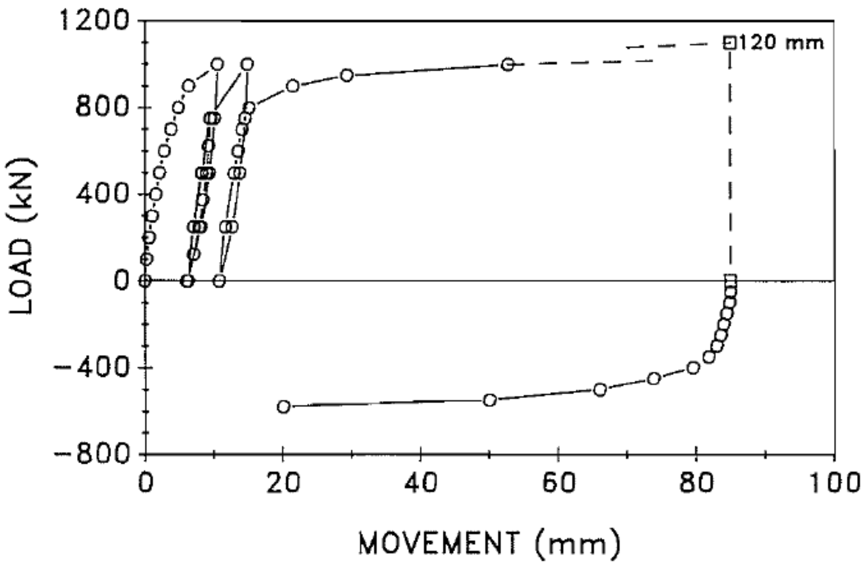


Figure 4.3 Measured load-displacement response of pile from Example 4.2

Limiting and average q_c design values in NEN 9997-1

Shaft resistance in Sand

In the Netherlands when calculating the shaft resistance of piles in sand layers where the cone resistance is above 12 MPa, the cone resistance used in Equation 4.1 is limited to an upper-bound value, See Figure 4.4.

(i) If the soil layer is less than 1m thick, the upper-bound value is a maximum of 12 MPa

(ii) if the soil layer is greater than 1m in thickness then the maximum value is taken as the lowest value in that layer. This value cannot be higher than 15 MPa.

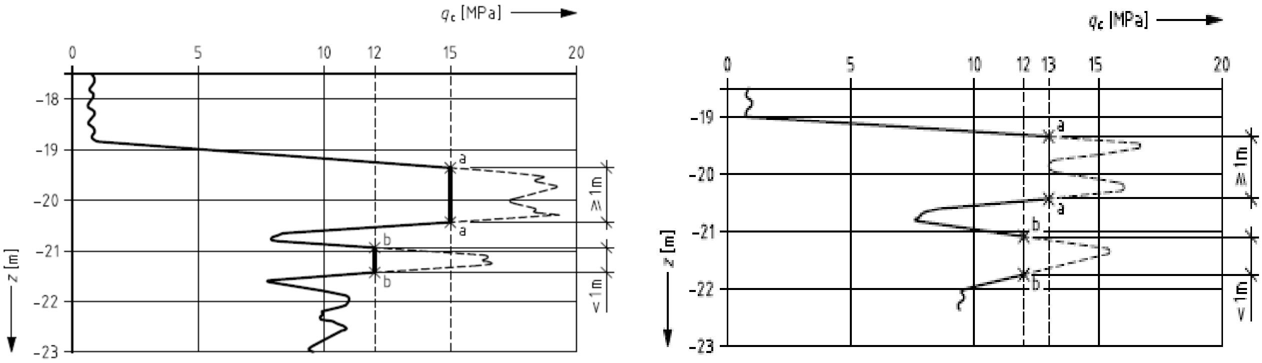


Figure 4.4 Concept of limiting qc values for shaft resistance from NEN 9997-1

Base Resistance in Sand

For the calculation of base resistance, the design q_c value is evaluated using the Koppejan averaging technique, See Figure 5.5 wherein q_c is evaluated over a zone of 0.7 to $4D_{eq}$ below the pile tip and $8D_{eq}$ above the pile tip, where D_{eq} is the equivalent pile diameter.

$$q_{c\,ave} = 0.5\ x\ \left(\frac{q_{c\,I}+q_{c\,II}}{2} + q_{c\,III}\right) \qquad [4.3]$$

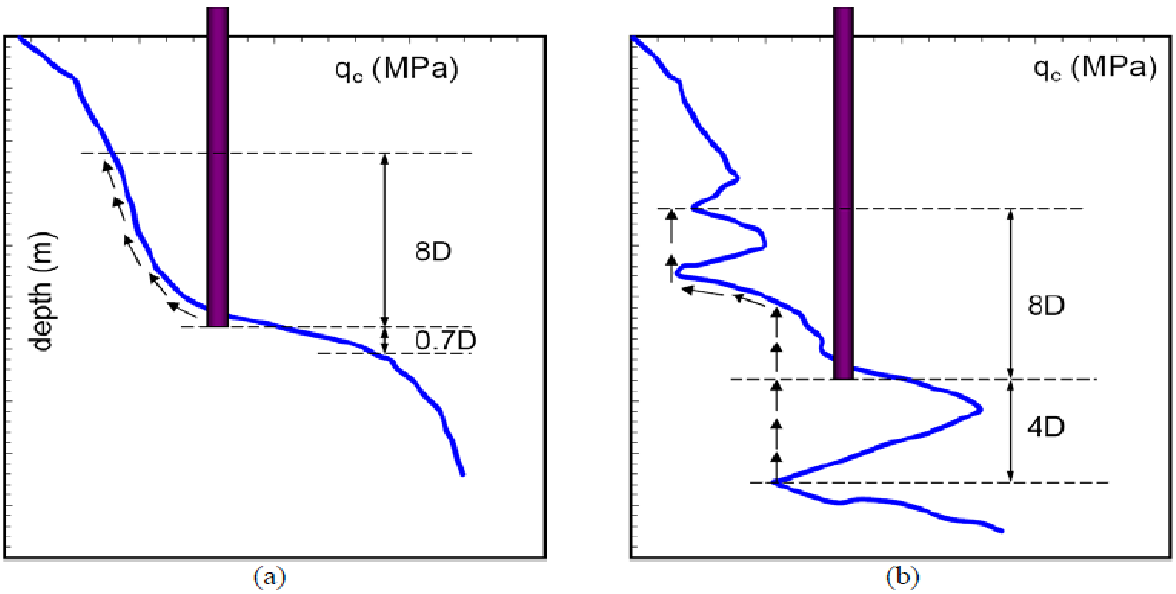
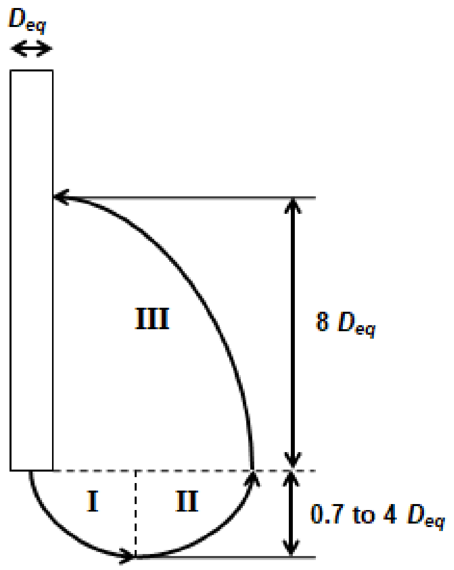


Figure 4.5 Averaging techniques for CPT q_c profile around a pile base (a minimum depth below the base, b maximum depth below the base to consider)



Where: q_{cI} is the average of the q_c values over a depth which varies between 0.7D to 4D below the pile tip depending on which length gives you the minimum average.
q_{cII} is the average of the q_c values following a minimum path rule over the same depth used to evaluate q_{cI} (the minimum path rule states as you move in the direction of measurement never use a value higher than the previous step, See red line in Figure 4.6)
q_{cIII} is the average of the q_c values following a minimum path rule from the pile tip to 8D above the tip and starting from the lowest value of q_c used to determine the q_{cII} value.

The application of the Koppejan averaging technique on a CPT profile from the Netherlands where 15.8m of soft clay overlies the sand bearing layer is shown in Figure 4.6.

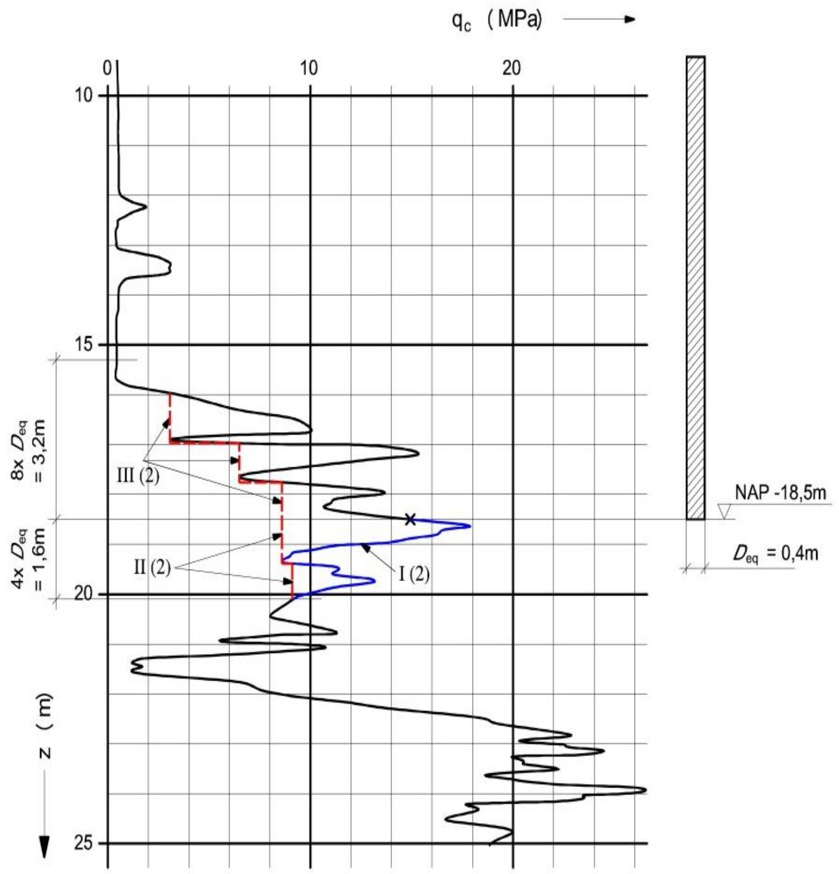


Figure 4.6 Koppejan averaging technique demonstrated on a typical Dutch CPT profile (Reinders et al 2016)

The maximum base resistance is then given by:

$$q_{bmax} = \alpha_p \times \beta \times s \times q_{c\,ave} \qquad [4.4]$$

Where: α_p is the empirical base factor from Table 4.1

β is a factor for enlarged pile bases determined from Figure 4.7

s is a cross-sectional shape reduction factor for non-square or circular piles, See Figure 4.8

q_{bmax} is limited to a maximum of 15 MPa

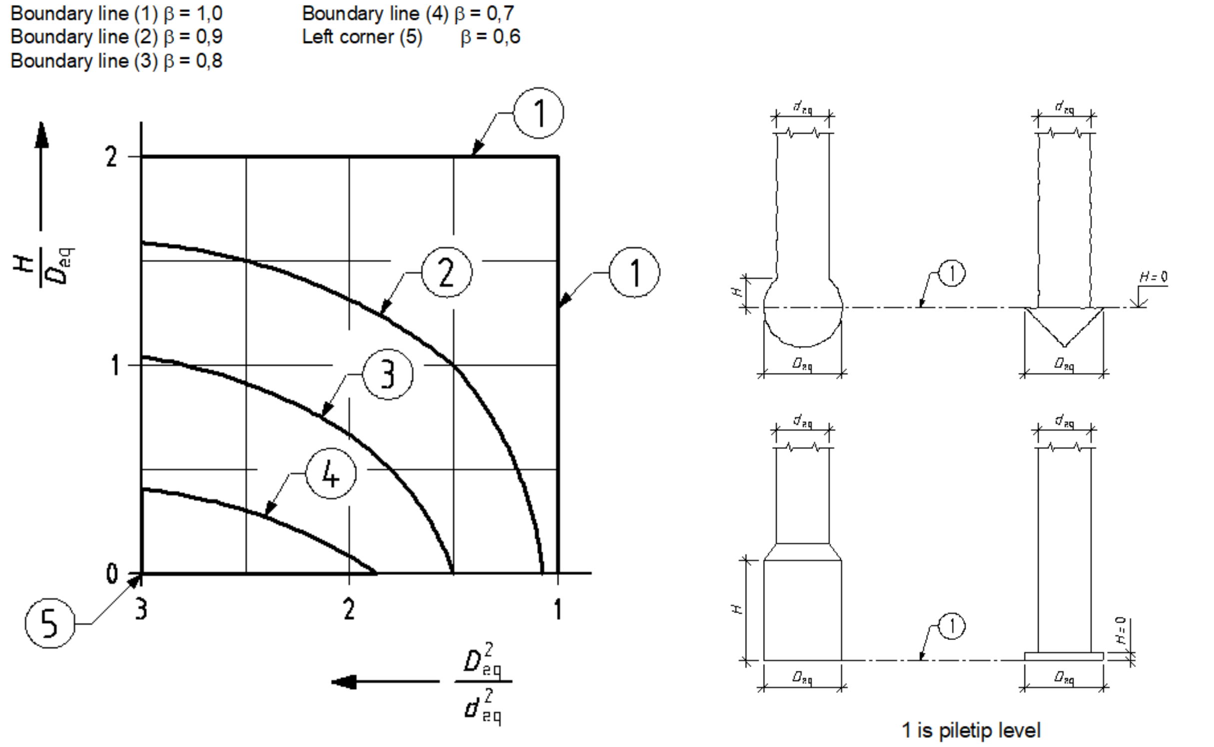


Figure 4.7 Determination of β value for Equation 4.4 (NEN 9997)

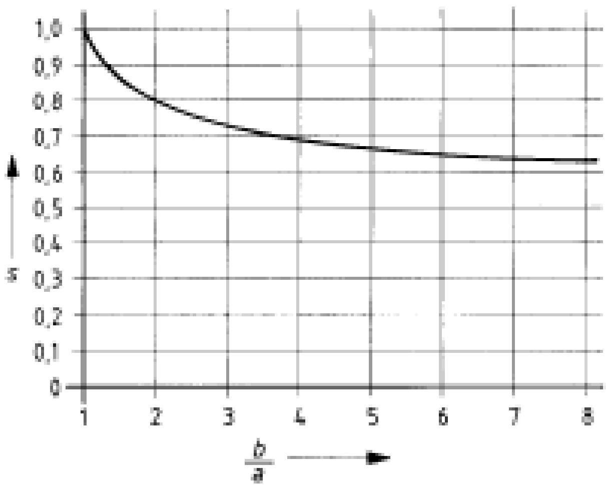


Figure 4.8 Reduction factor for rectangular piles (NEN9997)

Example 4.2 Calculate the total resistance of a closed-end pile using the NEN approach

A 356mm diameter, circular steel-closed ended pile was driven 6.75m into soil consisting of a loose over dense sand deposit, See Figure 4.9. The water table is at 3m bgl and the unit weight of the sand is 18 kN/m^3. The steel plate at the end of the pile is flush with the tube and therefore β and s are 1.0.

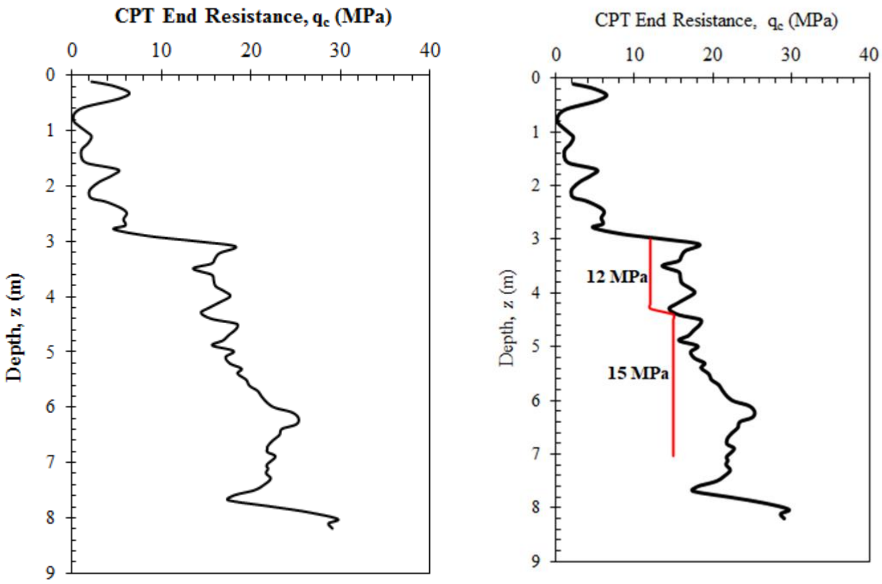


Figure 4.9 (a) CPT profile for example 4.2 (b) with NEN shaft resistance limits

Calculate the shaft resistance

$\tau_f = \alpha_s\, q_c$

Over the pile shaft length, See Figure 4.9b,

$q_{c\ ave}$ = 9518 kPa

$\tau_f = \alpha_s\, q_c$

= 0.01 x 9518 = 95.2 kPa

Q_s = 95.2 x 3.142 x 0.356 x 6.75= 719 kN

Calculate the base resistance

$q_{bmax} = \alpha_p \times \beta \times s \times q_{c\ ave}$

where: $q_{c\ ave} = 0.5\ x \left(\frac{q_{c\,I}+q_{c\,II}}{2} + q_{c\,III}\right)$

From Figure 4.10,

q_{cI} = 21.05 MPa, q_{cII} = 17.48 MPa and q_{cIII} = 16.75 MPa

$q_{bmax} = 0.7x1x1x0.5x\left(\frac{21.05+17.48}{2} + 16.74\right)$

= 12.60 MPa

$Q_b = A_b\, q_b = (0.356^2 x\ \pi/4)$ x 12,60 = 1.252 MN or 1254 kN

Calculate the total resistance

Total Resistance Q = 719 + 1254

= 1972 kN

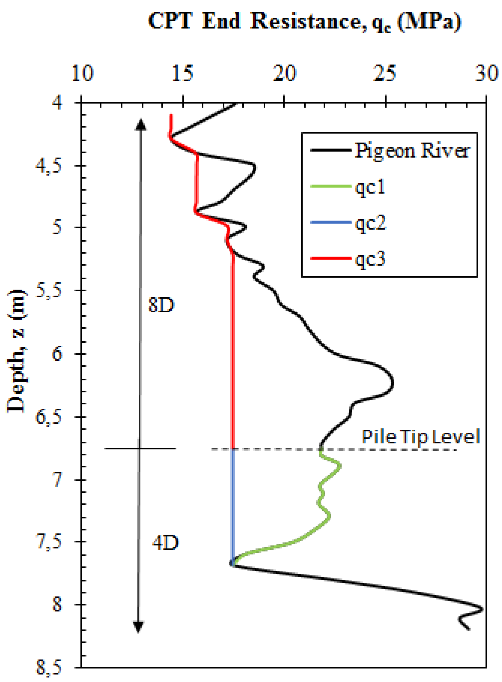


Figure 4.10 Determination of average q_c values using the Koppejan approach Ex. 4.2

Negative skin friction

For soft soils where an external load is applied (e.g. due to the construction of a highway embankment), in delta regions which experience regional settlement, the downward movement of soil can induce negative skin friction on an installed pile, See Figure 4.11. To develop positive skin friction as used above for shaft friction, the pile needs to settle more than the soil. In layers where the soil settles more than the pile, the shaft friction is called negative skin friction.

In the Netherlands, negative skin friction is in the design code calculated based on effective stresses, for single piles with large soil displacements:

$$\tau_{sneg} = K_0 \, \sigma'_v \tan \delta \qquad [4.5]$$

Where:

K_0 = coefficient of earth pressure at rest and $K_0 = (1 - \sin\phi')$

σ'_v = vertical effective stress in the middle if the settling layer

δ = interface friction angle between the soil and pile. $\delta = 0.75\,\phi'$ for prefabricated piles and $\delta = \phi'$ for piles cast in place.

The value $K_0 \times \tan\delta$ should be at least 0.25.

For pile groups, the negative skin friction is limited to the soil weight of the soil between the piles. For piles in soils with smaller soil displacements (less than 10 cm) an interaction calculation can be made, leading to less conservative results.

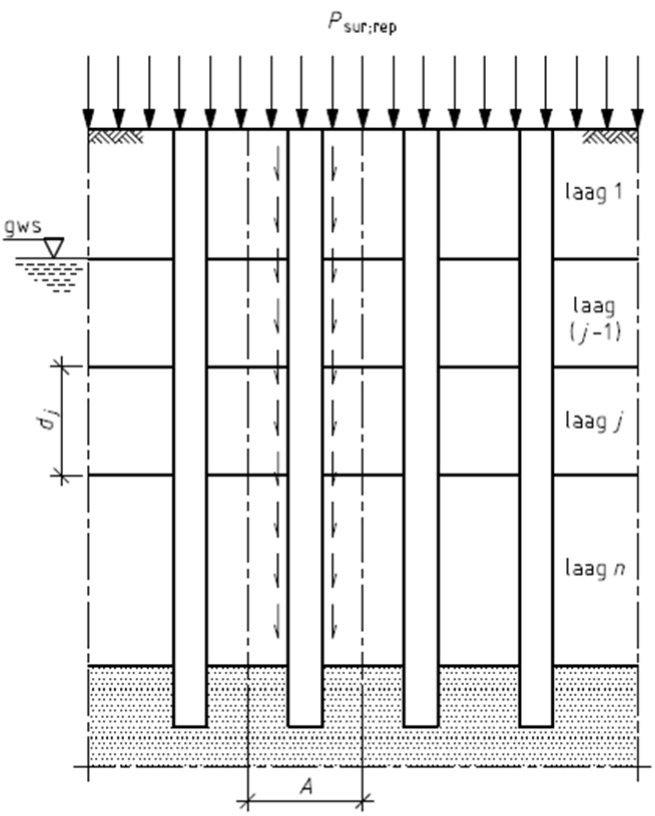


Figure 4.11 Development of negative skin friction on piles

Example 4.3 Axial compression and extension resistance of a precast concrete pile with negative skin friction

A 250 mm square precast concrete pile was driven to -16m NAP in the soil profile shown in Figure 4.12. Calculate the resistance to both compression and tension axial loading.

Solution:

(i) Calculate negative skin friction:

Negative friction zone is from -1.5m NAP to -12.5 m NAP. For soft clay $\phi' = 25°$ and $\gamma_{SAT} = 16$ kN/m^3.

$q_{sneg} = K_0 \, \sigma'_v \tan \delta$

$K_0 = (1 - \sin 25) = 0.58$

$\sigma'_v = ((16 \times 5.5)-(10 \times 5.5) = 33$ kPa

$\delta = \tan (0.75 \times 25) = 0.34$

Note $K_0 \times \tan \delta = 0.20$ so use min value of 0.25

$q_{sneg} = 0.25 \times 33 = 8.25$ kPa and $Q_{sneg} = 8.25 \times (4 \times 0.25 \times 11) = 91$ kN.

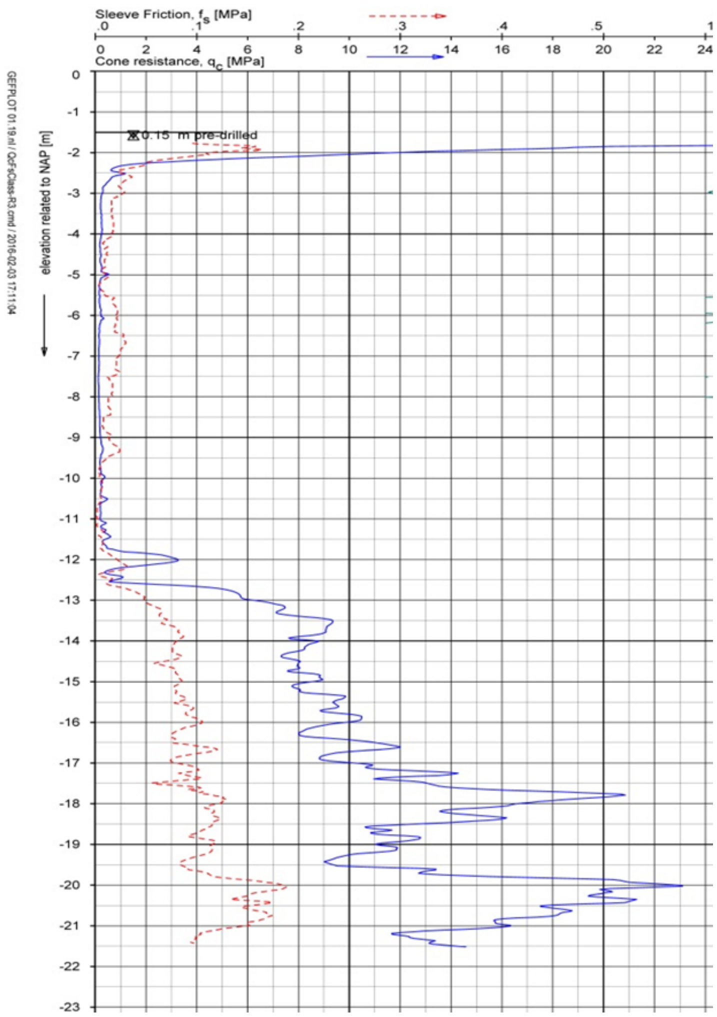


Figure 4.12 CPT Profile for Example 4.3

(ii) Calculate base resistance:

$$q_{b;max} = 0{,}5 \times \alpha_p \times \beta \times s \times \left(\frac{q_{c;I;mean} + q_{c;II;mean}}{2} + q_{c;III;mean}\right)$$

From, Figure 4.12, q_{c1} = 8.5 MPa, q_{c2} = 8.03 MPa and q_{c3} = 7.69 MPa

β and s are 1.0 for precast concrete

q_b = 0.5 x 0.7 x ((8.5+8.03)/2)+7.69)
= 5.58 MPa

Q_b = 5.58 x 0.25^2 = 349 kN

(iii) Calculate positive shaft resistance:

Positive shaft friction acts over the depth -12.5 NAP to – 16.0 NAP, Length = 3.5m.

$q_{c12\text{-}16.5m}$ = 7.89 MPa

$q_{sav} = \alpha_s\, q_c$
= 0.01 x 7.89
= 0.789 MPa or 78.9 kPa

Q_s = 78.9 x 4 x 0.25 x 3.5
q_{sav} x perimeter x length
= 276 kN

(iv) Total Pile Resistance Q

In compression

$Q = Q_b + Q_s - Q_{sneg}$
= 349 + 276 – 91
= 534 kN.

In tension

$q_{savt} = \alpha_{st}\, q_c$
= 0.007 x 7.89
= 54.5 kPa

Tension resistance Q_{st}

Q_{st} = q_{savt} x (4 x 0.25 x 3.5) - Q_{sneg}
= 54.5 x 3.5 - 91
Q_{st} = 99.75 kN

Open-ended tubular piles

While the majority of piles driven onshore are closed-ended, for some port developments, large bridges and offshore projects it is often preferable to drive open-ended steel tubular piles as they offer superior moment resistance, high axial capacity and can be driven to greater depths to provide larger tension resistance. Open-ended steel piles up to 10m in diameter have been driven for offshore wind turbines while piles up to 2.5m are driven routinely to depths over 100m to support large fixed oil and gas platforms.

The degree of soil displacement imposed by open-ended piles during installation can be much lower than that of equal diameter closed-ended piles. Xu et al., 2005, states that the behaviour of an open-ended or 'pipe' pile is expected to lie between that of a 'full-displacement' (Closed-ended driven/jacked) and 'non-displacement' (Bored / CFA etc.) piles. During driving or jacking of an open-ended pile, a soil plug is known to advance up the inside of the pipe. The pile can be fully coring, partially plugged or fully plugged during installation, and the mode of plugging can dramatically affect the degree of soil displacement and hence the pile installation resistance and post-installation load test capacity. The degree of soil displacement will not only affect the end-bearing capacity of the pile but will also affect the radial stresses developed along the pile outer shaft, with fully coring piles generally developing lower radial stresses than equivalent plugged or closed ended piles (as seen in Figure 4.12). The degree of soil plugging can be best be described by the Incremental Filling Ratio (IFR) which is defined as the incremental change in plug length, L_p, relative to change in pile penetration, L (IFR = $\Delta L_p/\Delta L$).

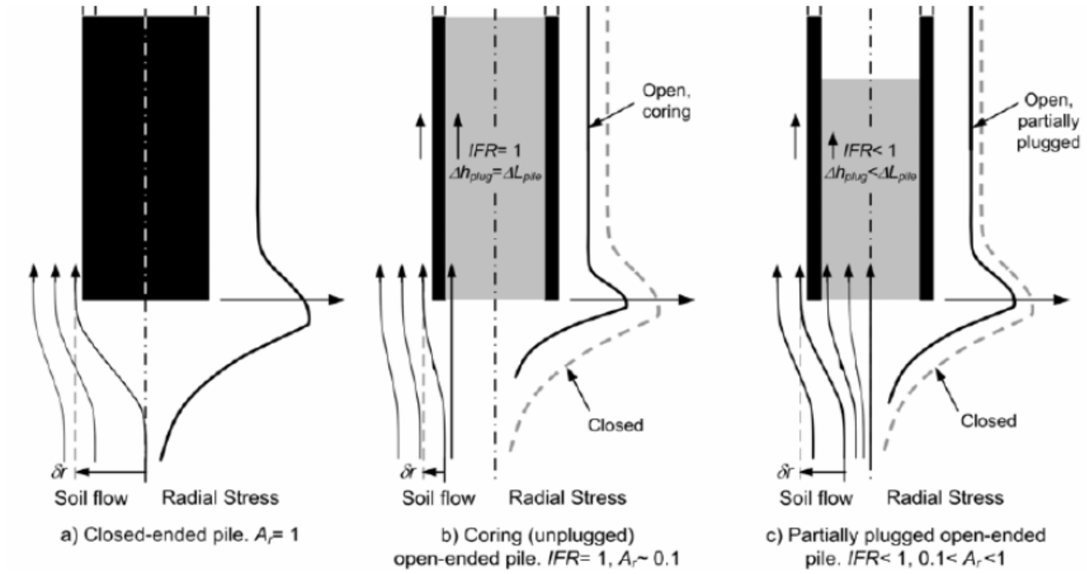


Figure 4.12 Schematic Streamlines of Soil Flow and Profiles of Radial Stress (after White et al., 2005)

During static loading (e.g. during a jacked installation or a maintained load test where pile acceleration, a_p, ≈ 0), plug slippage will occur if the end bearing of the soil beneath the plug, Q_{plug}, exceeds the friction generated between the internal soil column and the inner pile wall, $\Sigma\tau_{si}$. In a dynamic loading situation such as pile driving, plug slippage will occur when the internal friction force is less than the plug inertia force, F_i, plus the end bearing against the pile plug, Q_{plug}. Rausche and Webster, 2007, state that the plug inertia force increases with the square of the internal pile diameter, D_i, and proportionally with the pile acceleration at or above the pile toe when the plug moves with the pile. Therefore, although a pile may exhibit plugged behaviour during load testing, it is the behaviour during installation that controls the resistance developed during subsequent loading, if IFR values are high the soil has experienced less pre-stress and exhibits lower stiffness than a closed-end pile, this is accounted for directly in offshore design codes, e.g. See Lehane et al. (2020) where α_p varies from 0.15 for an unplugged to 0.5 for a fully plugged pile.

Example 4.3 Axial Capacity of Open-Ended Pile in Sand

Taking the soil conditions in Example 4.2 and considering a 356mm outer diameter open-ended pile, with a wall thickness of 32mm was driven 6.75m into the dense sand layer considered in Example 4.2. At the end of installation the final plug length inside the pile, L_p was 5m.

Shaft Capacity

In NEN 9997-1 the impact of plugging on shaft resistance is accounted for by adopting a 40% lower α_s value for open-ended steel piles compared to closed-end piles, See Table 4.1.

$\tau_f = \alpha_s\, q_c$

Over the pile shaft length, See Figure 4.9b, $q_{c\ ave}$ = 9518 kPa (See Page 39)

$\tau_f = \alpha_s\, q_c$

= 0.006 x 9518 = 57 kPa

Q_s = 57 x 3.142 x 0.356 x 6.75= 430 kN

Base Capacity

Whilst the base resistance factor α_p for closed and open-ended piles are identical, a check on pile plugging should be performed as follows:

The base capacity of an open-ended pipe pile is the lower of

- Case A - the capacity of the steel surface area together with the inner friction
- Case B - the base capacity of the full surface area of the pipe

In the code there no rules for how to assess the inside shaft capacity. Thus this is often taken similar as the outside shaft capacity.

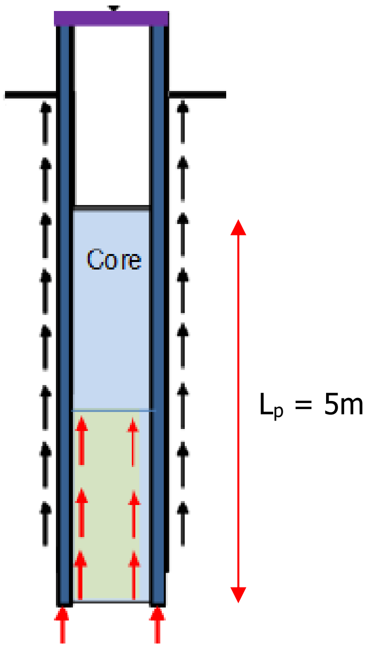


Figure 4.13 Components of base resistance for unplugged open-end pile

Case A: The base resistance is given from a combination of the load on the steel annular area, Q_{ann} and the inner friction of the soil plug, Q_{plug}. Wall thickness is 32mm and therefore internal diameter of pile in 292mm.

Area of pile annulus = $\pi(356/2)^2 - \pi(292/2)^2$ = $0.0326m^2$

Q_{ann} = q_b x Area of annulus
q_b is the same as closed-ended pile (Ex.4.2)

Q_{ann} = q_b x Area of annulus
= 12600 x 0.0326
= 411 kN

Inner friction, Q_{si} = assume friction developed on the inside equal to outer friction, τ_f = 0.06 x 9518 = 57 kPa

Therefore Q_{si} = 57 x 3.142 x 0.292 x 5
= 261 kN

Q_b Case A = 411 + 252 = 672 kN.

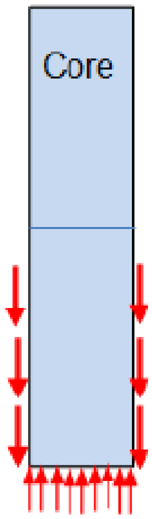


Figure 4.14 Components of base resistance for plugged open-end pile

Case B:

Full base area of pile = $\pi(356/2)^2$ /4
= $0.0996m^2$

Q_b = q_b x Full base area

q_b is the same as closed-ended pile (Ex.4.2)

Q_b = q_b x Area of base
= 12600 x 0.0996
= 1254 kN

Case B > Case A use lower value, Q_b = 672 kN.

Total Capacity of Example 4.3

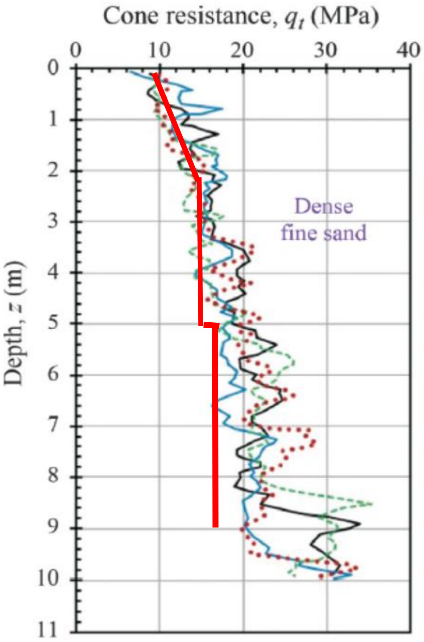
Q = Q_s + Q_b = 430 + 672 = **1102 kN**

4.3 Axial capacity of replacement piles in sand using CPT

In general the design of replacement piles follows all the same procedures as displacement piles, with the appropriate α factors from Table 4.1.

Example 4.4

Calculate the capacity of an 8m long, 800mm diameter CFA pile installed in the soil profile shown in Figure 4.15. The sand has a unit weight of 20kN/m^3 and the water table level is at great depth.



Step 1 Calculate the shaft and base resistance

$$\tau_f = \alpha_s \, q_{cav}$$

$$\tau_f \text{ (kPa)} = 0.006 \times 12875 = 77 \text{ kPa}$$

$$Q_s \text{ (kN)} = 77 \times (3.142 \times 0.8 \times 8) = 1548 \text{ kN}$$

Step 2 Calculate the shaft and base resistance

$$q_b = \alpha_p \, q_c$$

$$q_b \text{ (kPa)} = 0.56 \times 20{,}000 = 11{,}200 \text{ kPa}$$

$$Q_b \text{ (kN)} = 11{,}200 \times (3.142 \times 0.8^2/4) = 5630 \text{ kN}$$

Step 3 Calculate the total resistance

$$Q_t = 1548 + 5630 = 7178 \text{ kN}$$

4.4 Effective Stress Design

In many regions of the world including in the offshore sector pile design has traditionally been performed using effective stress methods.

Shaft Resistance

The peak unit shaft resistance (τ_f) mobilised by a pile in sand can be estimated using earth pressure theory as:

$$\tau_f = K\,\sigma'_v \tan\delta_f \qquad [4.7]$$

Where: K is the operational earth pressure coefficient, σ'_v is the in-situ vertical effective stress and δ_f is the soil-pile interface friction angle. This approach is similar to the approach used for negative skin friction in the Netherlands. For concrete piles with a rough interface the failure surface occurs not at the pile-soil interface but in the soil mass and therefore δ_f ϕ'. For steep piles $\delta_f = 0.75\phi'$ for normal roughness. If the piles are painted or smoothed the interface friction angle should be measured.

A common difficulty with the application of Equation 4.7 is the choice of an appropriate K value for design. From Equation 3.15 we know that the in-situ horizontal effective stress prior to pile installation is affected by the stress history (OCR). This value is then affected by pile installation. A simple assumption is that replacement piles have no effect on piles resistance and closed-end displacement piles have a large effect, See Table 4.2.

Pile Type	K see Eqn 4.7
Bored or Jetted	K_0
Low displacement driven (i.e. open-ended)	1.2 K_0
Large Displacement	1.6 K_0

Table 4.2 Influence of pile type on K_0

Base Resistance

The peak unit base resistance (q_b) mobilised by a pile in sand can be estimated using earth pressure theory as:

$$q_b = N_q\,\sigma'_v \qquad [4.8]$$

Where: N_q is a bearing capacity factor which depends on ϕ' and σ'_v = vertical effective stress at the pile base.

For large displacement piles the bearing capacity factors derived by Berezantsev (1961) are commonly used in design, See Figure 4.16 where the value is seen to depend on the penetration depth, L_p into the founding soil deposit. For other pile types use Eqn. 4.9:

$$N_q = \frac{(1+sin\phi\prime)}{(1-sin\phi\prime)}e^{\pi tan\phi\prime} \qquad [4.9]$$

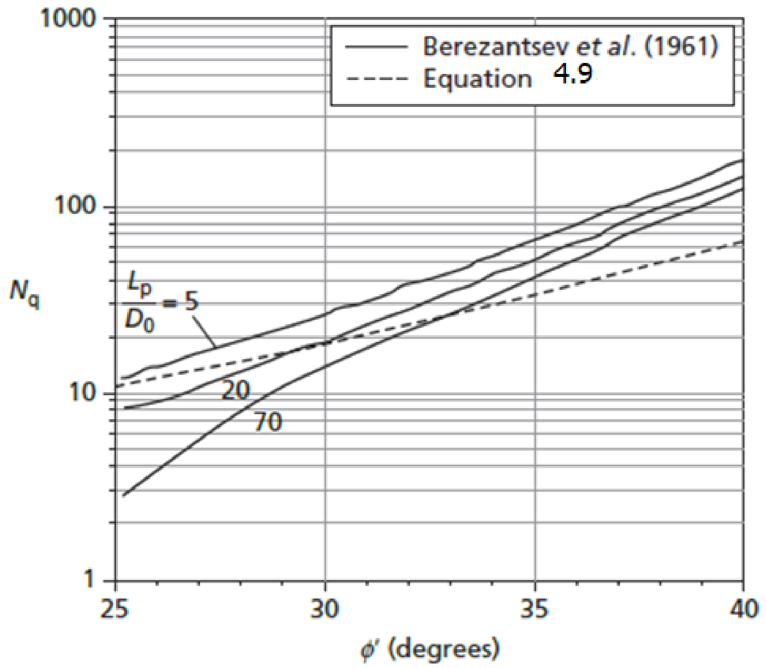


Figure 4.17 End-Bearing capacity factors for driven piles after Berezantsev (1961)

Example 4.5

Redo example 4.1 using effective stress design methods and compare the predicted resistance. The sand has a dry unit weight of 18kN/m^3 saturated unit weight of 20kN/m^3 and the water table level is at 5.5 m bgl.

Step 1 Estimate of ϕ' and K_0

$\phi'_p{}^\circ = 17.6 + 11 \log Q_{tn}$

$K_0 = (1 - \sin\phi'_p) OCR^{\sin\phi'p}$

$\sigma'_p = 0.32\, q_t^{0.7}$

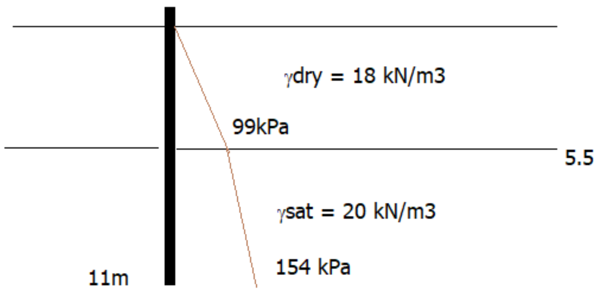


Figure 4.18 Soil layers in Example 4.5

Given the water table is present at 5.5m, see Figure 4.18 we consider the deposit as a two-layer soil. For shaft resistance we consider the stress conditions at mid-point of each layer, i.e. at 2.75m and 8.25m bgl. The $q_c \approx q_t$ values are from Figure 4.2.

z (m)	q_c (kPa)	u_2 (kPa)	σ_{v0} (kPa)	U_0 (kPa)	$\sigma_{v0'}$ (kPa)	q_t (kPa)	Q_{tn}	ϕ'_p °	OCR
2.75	5000	0	49.5	0	49.5	5000	70.36	38	2.51
8.25	5000	27.5	154	27.5	126.5	5000	43.09	36	1
11	5250	55	209	55	155	5250	40.48	35	1

at 2.75m bgl:
σ_{vo} = 18 x 2.75 = 49.5 kPa, U_0 = 0 and σ'_{v0}= 49.5 kPa
From Eqn 3.4 Q_{tn} = (5000-49.5)/100 x $(100/49.5)^{0.5}$ = 70.36
ϕ'_p° = 17.6 + 11 log 70.36 = 38°
σ'_p = 0.32 x $5000^{0.7}$ = 124.3
OCR = 124.3/49.5 = 2.51

at 8.25m bgl:
σ_{vo} = (18 x 5.5)+ (20 x 2.75) = 154 kPa, U_0 = 27.5 and σ'_{v0}= 126.5 kPa
Q_{tn} = (5000-154)/100 x $(100/126.5)^{0.5}$ = 43.09
ϕ'_p° = 17.6 + 11 log 43.09 = 36°
σ'_p = 0.32 x $5000^{0.7}$ = 124.3
OCR = 124.3/126.5, since current stress is highest stress OCR = 1.0

at 11m bgl:
σ_{vo} = (18 x 5.5)+ (20 x 5.5) = 209 kPa, U_0 = 55 and σ'_{v0}= 155 kPa
Q_{tn} = (5250-209)/100 x $(100/155)^{0.5}$ = 40.48
ϕ'_p° = 17.6 + 11 log 40.48 = 35°
σ'_p = 0.32 x $5250^{0.7}$ = 128.6
OCR = 128.6/155, since current stress is highest stress OCR = 1.0

Step 2 Calculate the shaft resistance

at 2.75m bgl:

$$\tau_f = K\,\sigma'_v \tan\delta_f$$

$$K = 1.6\ K0 \text{ (See Table 4.2)}$$
$$= 1.6 \times (1\text{-}\sin 38)\ 2.51^{\sin 38}$$
$$= 1.08$$
$$\tau_f = 1.08 \times 49.5 \times \tan 38 = 42 \text{ kPa}$$

at 8.25m bgl:

$$K = (1\text{-}\sin 36)$$
$$\tau_f = 1.6 \times (1\text{-}\sin 36) \times 126.5 \times \tan 36 = 61 \text{ kPa}$$

$$Q_s = 42 \times (4 \times 0.285 \times 5.5) + 61\ (4 \times 0.285 \times 5.5)$$
$$= 642 \text{ kN}$$

Step 3 Calculate the base resistance

at 11m bgl:

$$q_b = N_q\ \sigma'_v$$

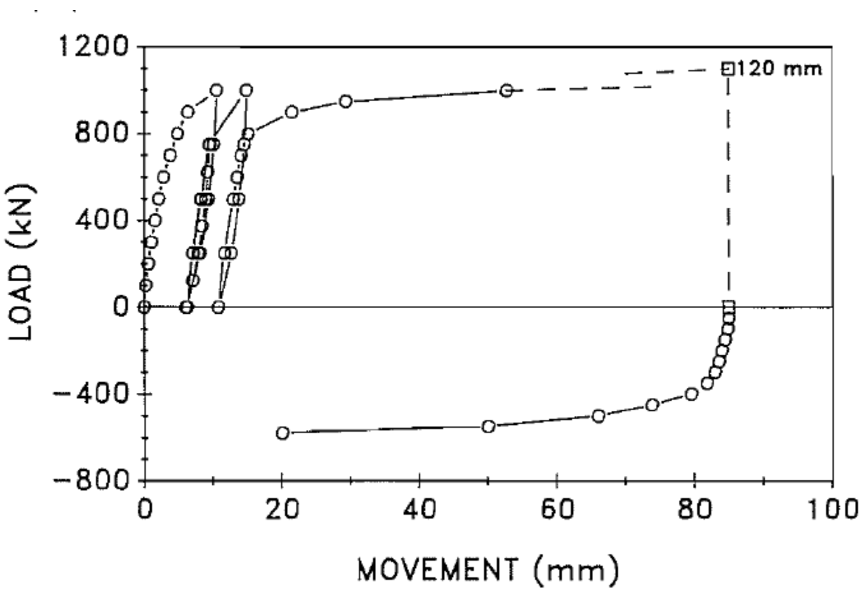
For $\phi' = 35^\circ$ and $L_p/D_{eq} = 5.5/(1.13 \times 0.285)=17$, $N_q = 40$ from Figure 4.17.

$$q_b = 40 \times 155 = 6200 \text{ kPa}$$
$$Q_b = q_b\ A_b$$
$$= 6200 \times 0.285^2$$
$$= 504 \text{ kN}$$

Step 4 Total resistance

$$Q_t = 643 + 504 = 1146 \text{ kN}$$

Very similar to the value of 927kN calculated using NEN and close to measured pile capacity. And note that this prediction (effective stress) is not determined at 10% of pile diameter, rather at maximum displacement.



1200
800
400
0
-400
-800
LOAD (kN)
0
20
40
60
80
100
MOVEMENT (mm)
120 mm

Practice Questions Section 4

Question 1

A 400 mm diameter *circular* precast concrete pile is driven through a 10m thick layer of soft clay (q_c = 150 kPa, f_s/q_c = 3%, ϕ' =20°, γ_{SAT} = 17 kN/m^3) into a layer of sand with the q_c values shown in Table 1 and f_s/q_c = 1%. The final tip depth of the pile is 15m bgl and water table is at ground level. Report all answers to the nearest whole number. Calculate;

(i) The negative skin friction stress q_{sneg} that develops at 5m below ground level.

(ii) Using the answer from (i) as the average stress for the entire soft clay layer calculate the negative skin friction force Q_{sneg}.

(iii) Calculate the average shaft resistance (in kN/m^2) acting in the sand layer between 10m and 15m below ground level using the q_c values in Table Q1.1

(iv) Estimate q_{c1mean}, q_{c2mean} and q_{c3mean} according to the Koppejean method of averaging using the value given in Table Q2.2 and estimate q_{bmax}

(v) Calculate the total resistance of the pile, Q_t.

z (m)	q_c (kPa)
10	5700
11	5600
12	5500
13	6000
14	5500
15	5700

Table Q1.1 q_c values for calculation of shaft resistance calculation

z (m)	q_c (kPa)
11.8	5600
12.5	5600
13	6000
13.5	5750
14	5500
14.5	5600
15	5650
15.28	5750
15.5	5800
16	5900
16.3	6000
16.6	5900

Table Q1.2 Additional q_c values for base resistance calculation

Question 2

A single 20m long, 600 mm diameter circular reinforced concrete pile is required to carry a maximum compression load of 700 kN. Soil conditions at the site consist of dense sand to great depth (water table is at ground level). The saturated unit weight of the sand is 20 kN/m^3, the unit weight of water is 10 kN/m^3 and the soil friction angle $\phi' = 36^o$ can be assumed to be equal to the interface friction angle, δ.
N_q values for typical friction angles are given in the Table:

ϕ'	N_q
28	8
30	11
32	15
34	21
36	48

Calculate:

(i) The effective vertical stress at the mid-depth of the pile
(ii) The effective vertical stress at the pile base.
(iii) Calculate the average unit shaft friction q_{sav} acting on the pile (answer to the nearest whole number).
(iv) Calculate the unit base resistance q_b acting on the pile.
(v) From part iii calculate the pile shaft resistance, Q_s
(vi) From part iv calculate the pile base resistance, Q_b
(vii) In your opinion does the pile sufficient resistance to carry the applied load?

Question 3

An existing four storey building is supported by 8 m long, 350 mm diameter bored pile foundations. The working load on each pile is 400 kN. Ground conditions at the site consist of normally consolidated soils, with 2 m of loose sand ($c'=0$, $\phi'_{cv} = 30°$ and $\gamma = 18$kN/m^3), overlying gravel ($c'=0$, $\phi'_{cv} = 36°$ and $\gamma = 18$ kN/m^3). The water table is at the top of the Gravel layer. The building owner wishes to add two storeys to the building, thereby increasing the pile loads by 50%. Determine the factor safety of the piles. Do you think the factor of safety is adequate?

Question 4

The CPT profile at a site in Delft with ground level at 1.12m NAP is shown in Figure Q4. 1 (the data are given in excel format in Brightspace CPTforQ4.xls). Considering a 250mm square precast concrete pile is driven to -13m NAP, using the Dutch design code and Koppejan CPT averaging technique calculate:

(i) Calculate the unit base resistance (in MPa)
(ii) Calculate the average shaft resistance (in kPa) developed between a depth of -11.5m and -13m NAP.

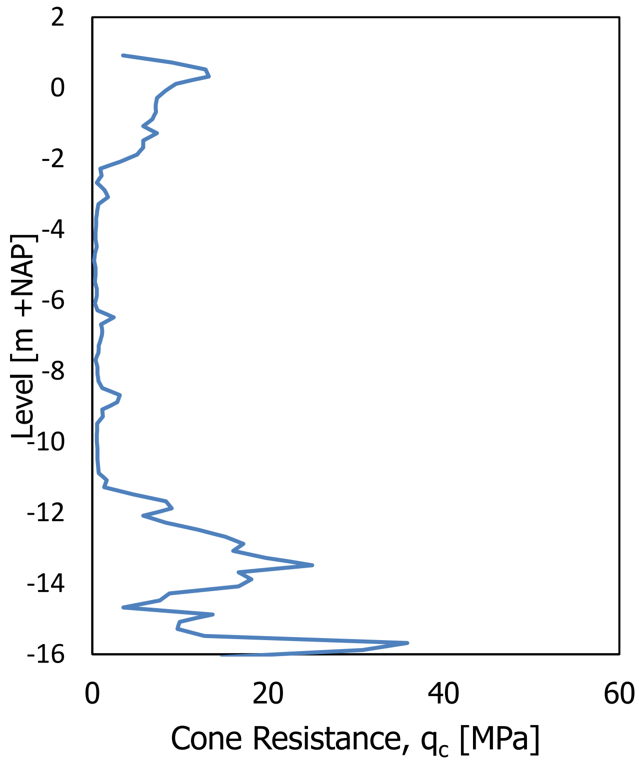


Figure Q4.1 CPT Profile

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APPENDIX A

Introduction to Effective Stress

Principle of effective stress

Soil can be considered as a two or three phase material as a skeleton of solid particles enclosing continuous voids which contain water and/or air. For the range of stresses usually encountered in practice the individual solid particles and water can be considered incompressible; air, on the other hand, is highly compressible. Upon the application of loading or unloading (e.g. due to construction a building, an excavation or a change in water table level) the volume of the soil skeleton as a whole can change due to rearrangement of the soil particles into new positions, mainly by rolling and sliding, with a corresponding change in the forces acting between particles. The compressibility of the soil skeleton will depend on the structural arrangement of the solid particles. In a fully saturated soil, since water is considered to be incompressible, a reduction in volume is possible only if some of the water can escape from the voids. In a dry or a partially saturated soil a reduction in volume is always possible due to compression of the air in the voids, provided there is scope for particle rearrangement.

Shear stress can be resisted only by the skeleton of solid particles, by means of forces developed at the interparticle contacts. Normal stress may be resisted by the soil skeleton through an increase in the interparticle forces. If the soil is fully saturated, the water filling the voids can also withstand normal stress by an increase in pressure. The importance of the forces transmitted through the soil skeleton from particle to particle was recognized in 1923 when Terzaghi presented the principle of effective stress, an intuitive relationship based on experimental data. The principle applies only to fully saturated soils and relates the following three stresses:

1. the total normal stress (σ) on a plane within the soil mass, being the force per unit area transmitted in a normal direction across the plane, imagining the soil to be a solid (single-phase) material;
2. the pore water pressure (u), being the pressure of the water filling the void space between the solid particles;
3. the effective normal stress (σ') on the plane, representing the stress transmitted through the soil skeleton only.

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

Watch the video https://www.youtube.com/watch?v=GvK0D-wBp88 for a quick tutorial

Example 1 from lecture slides

A soil profile consists of 5m of sand overlying 4m of gravel resting on bedrock. The water table level is 2m below ground level.

(a) Determine the distributions of total vertical stress, porewater pressure and effective vertical stress with depth down to bedrock, given the bulk density, ρ of the sand above the water table is 1.70 Mg/m^3, and the saturated density of sand and gravel are 2.05 and 2.15 Mg/m^3 respectively.

(b) How do these change if the water level is lowered to the sand/gravel interface.

Solution Part a:

At the surface of the sand all the stress are zero.

At a depth of 2m below ground level (bgl):
The vertical total stress σ_v = (1.7 x 9.81 x 2) = 33 kPa or kN/m^2
The porewater pressure u = 0
The effective vertical stress σ'_v = 33 – 0 = 33 kPa

At a depth of 5 m bgl.
The vertical total stress σ_v = 33 + (2.05 x 9.81 x 3) = 94 kPa
The porewater pressure u = 1 x 10 x 3 = 30 kPa
The effective vertical stress σ'_v = 94 – 30 = 64 kPa

At a depth of 9 m bgl.
The vertical total stress σ_v = 94 + (2.15 x 9.81 x 4) = 178 kPa
The porewater pressure u = 1 x 10 x 7 = 70 kPa
The effective vertical stress σ'_v = 178 – 70 = 108 kPa

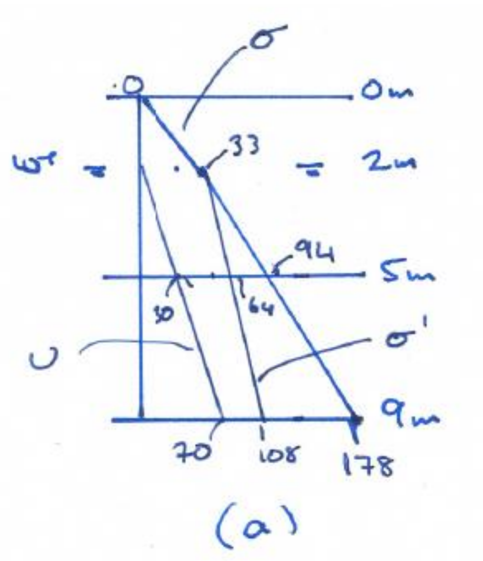


Figure Plot of Solution Part a

Solution Part b:

Calculate stress at points where unit weight/density changes and at material boundaries:

At a depth of 5 m bgl.
The vertical total stress σ_v = 1.7 x 9.81 x 5 = 83 kPa
The porewater pressure u = 0 kPa
The effective vertical stress σ'_v = 83 – 0 = 83 kPa

At a depth of 9 m bgl.
The vertical total stress σ_v = 83 + (2.15 x 9.81 x 4) = 168 kPa
The porewater pressure u = 1 x 10 x 4 = 40 kPa
The effective vertical stress σ'_v = 168 – 40 = 128 kPa

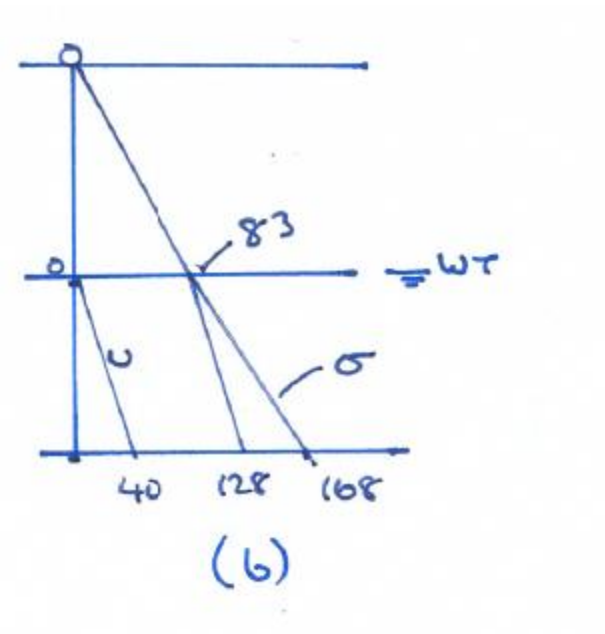


Figure Plot of Solution Part b

Example 2 from lecture slides

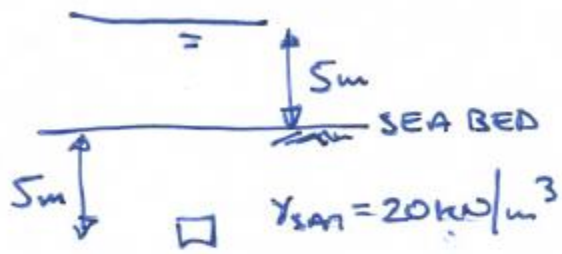
Calculate the effective vertical stress in a layer of sand with a saturated density of 20kN/m^3, at 5m below the bottom of a body of water (assume γ_w = 10 kN/m^3):

(a) 5 m deep

(b) 1000m deep

Solution (a)

At sea bed level:
Total stress (σ) = 5 x 10 = 50 kPa
Porewater pressure (u) = 5 x 10 = 50kPa
Effective stress (σ') = 50 – 50 = 0 kPa

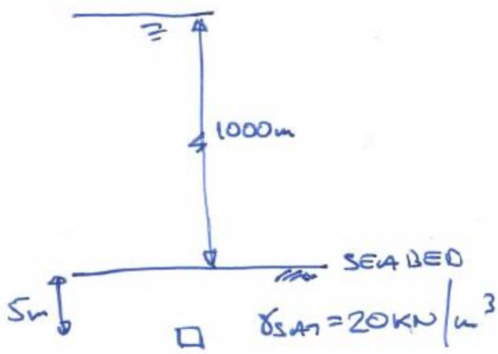


At 5m below sea bed level

Total stress (σ) = 50 + (5 x 20) = 150kPa
Porewater pressure (u) = 10 x 10 = 100kPa
Effective stress (σ') = 150 – 100 = **50kPa**

Solution (b)

At sea bed level
Total stress (σ) = 1000 x 10 = 10,000 kPa
PWP (u) = 1000 x 10 = 10,000 kPa
Effective stress (σ') = 10,000 – 10,000 = 0 kPa



At 5m below sea bed level

Total stress (σ) = 10,000 + (5 x 20) = 10,100 kPa
Porewater pressure (u) = 1005 x 10 = 10,050 kPa
Effective stress (σ') = 10,100 – 10,050 = **50 kPa**

Example 3 from lecture slides

A 4m thick layer of soft clay overlies a deep deposit of sand. Water table is at ground level. Given the saturated unit weight of sand is 18 kN/m^3, calculate the total stress, porewater pressure and effective vertical stress at 2m below original ground level:

(a) Given the conditions outlined above.

(b) Immediately after a very wide, 4m high embankment of Clay with a bulk unit weight of 18kN/m^3 was built above it.

(c) Many years after construction – when consolidation of the clay layer is complete.

(d) Describe the physical response to the applied loading and explain how the soil characteristics would change in response to the applied load

Solution (a)

Stress at ground level = 0

At 2m bgl.
Total stress (σ) = 2 x 18 = 36 kPa
Porewater pressure (u) = 2 x 10 = 20 kPa
Effective stress (σ') = 36 – 20 = 16 kPa

Solution (b)

Stress at ground level = (4 x 18) = 72 kPa

At 2m below ground level
Total stress (σ) = 36 + (4 x 18) = 108 kPa
Porewater pressure (u) = 20 + 72 = 92 kPa
Effective stress (σ') = 108 – 92 = 16 kPa

Solution (c)

Stress at ground level = (4 x 18) = 72 kPa

At 2m below ground level
Total stress (σ) = 36 + (4 x 18) = 108 kPa
Porewater pressure (u) = 2 x 10 = 20 kPa
Effective stress (σ') = 108 – 20 = 88 kPa

Solution (d)

After porewater pressure dissipates the vertical effective stress has risen from 16 kPa to 88 kPa, i.e. all additional stress is transferred to the soil. – This is accompanied by settlement of the clay layer.

Test Question 1

Calculate the total stress, pore pressure and effective stress at 5 m depth in a uniform deposit of soil with a water table level of 2 m below ground level. The dry unit weight γ_d is 15 kN/m^3 and the saturated unit weight is γ_{SAT} = 18 kN/m^3. γ_w = 10 kN/m^3

Answer = 84 kN/m^2, 30 kN/m^2 and 54 kN/m^2

Test Question 2

A site investigation revealed a deep deposit of sand with the water table at 2 m below ground level. A designer proposes to drive a 10 m long concrete pile into the sand layer. Calculate the original effective stress acting at the 10 m below ground level prior to pile installation as this is an input parameter to calculate the pile resistance using effective stress methods. The density of the sand above the water table level is 1.8 Mg/m^3 and below the water table is 2.0 Mg/m^3. γ_w = 10 kN/m^3

Answer at 10m below ground level the total stress is 192 kPa, the pore pressure is 80 kPa and therefore the **effective stress = 112 kPa.**

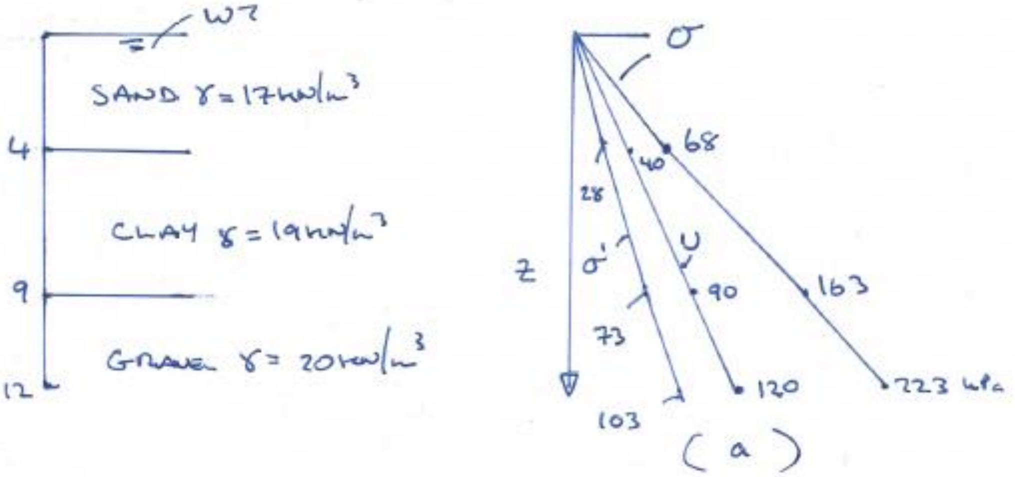
Homework Question from slides

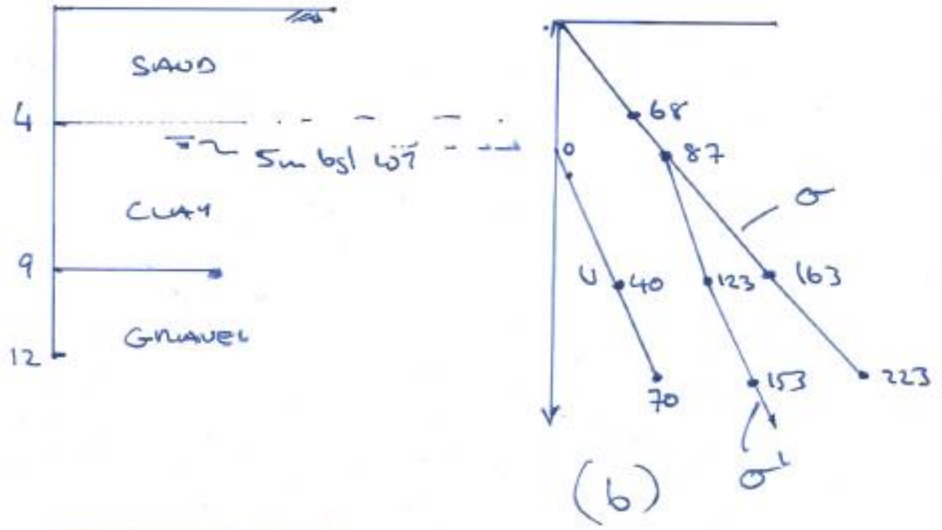
A soil profile consists of 4m of sand (total unit weight=17kN/m^3), over 5m of Clay (unit weight = 19kN/m^3) over 3m of Gravel (unit weight =20kN/m^3). Calculate the total stress, effective stress and pore water pressure at the base of each layer (i.e. at 4m, 9m and 12m below ground level), if:

(a) Water table is at ground level
(b) Water table is 5m below ground level
(c) Water table is 12m below ground level

Plot your answer as stress against depth

Answer below:


WT
SAND γ = 17 kN/m³
4
CLAY γ = 19 kN/m³
9
GRAVEL γ = 20 kN/m³
12
σ
68
40
28
z
σ'
U
90
163
73
103
120
223 kPa
(a)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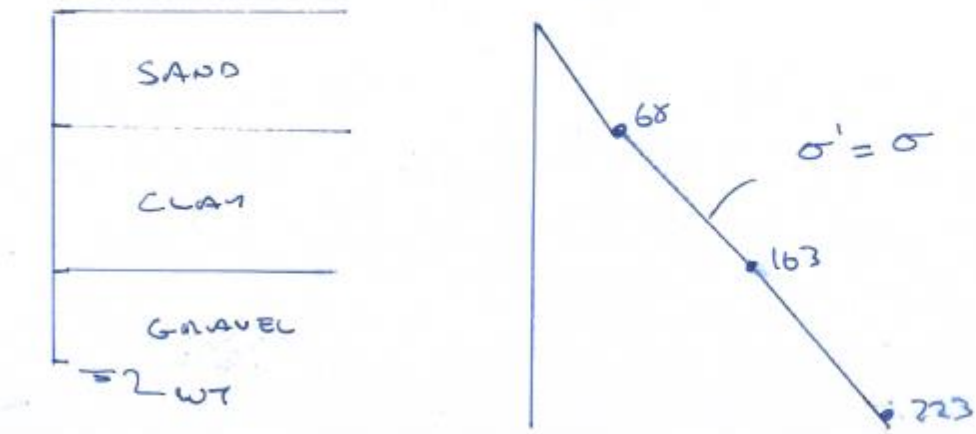
5m bgl WT
0
68
87
σ
CLAY
9
U
40
123
163
GRAVEL
12
70
153
223
σ'
(b)


SAND
68
σ' = σ
CLAY
163
GRAVEL
WT
223