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Construction on peat and organic soils

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TC 15: "Peat and Organic Soils"
technical committee of the ISSMFE

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Summary of report:

This technical document describes the behaviour of peat and gives an overview of the design methodes for peat. Subject headings include general characteristics and classification issues, prototypes of peat problems, design and calculation methods, special construction techniques, parameter determination safety aspects and monitoring.

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Delft Geotechnics

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1 Introduction

1.1 Background

Civil engineering activities on peat and organic soils pose special challenges due to the large compressibility and low strength. There has been a strong tendency in the past to avoid this challenge by either choosing other building sites or by simply removing the soil and replacing it by e.g. sand. Even in prehistoric times, it was sometimes inescapable to deal with these soils. Increasingly in modern times areas covered by peats and organic soils, hitherto often considered as marginal land, are being developed for habitation, industry and exploitation of peat resources. This calls for the application of geotechnical science to these soils.

In some countries, peat areas were developed early for farming and habitation. The moors of in the central Netherlands e.g. were successfully reclaimed from about 900 A.D. onwards, leading to early familiarity with the idiosyncrasies of peat soils, not only as for farming and fuel, but also as foundation for building activities. Buildings were affected by these soils, while dykes were often built out of these soils. In the latter case, decomposition of the vegetable remains in the peat sometimes led to breaching. The trial and error process of interacting with these soils has led to considerable empirical understanding of their behaviour, which has been the starting point for further development of the geotechnical science with regard to peats and organic soils.

Gradually, peat geotechnics is developing to maturity. Milestones have been publications such as the Muskeg Engineering Handbook [ref. 1.1], the Peat Engineering Handbook [ref. 1.2], Amaryan [ref. 1.3], Hobbs [ref. 1.4]. An important impetus was given by the instigation in 1985 of Technical Committee TC15 for Peats of the ISSMFE. A major activity during the first years of TC15, when it was chaired by prof. P.A. Konovalov of the former USSR, was the 2nd Baltic Conference in Tallinn in 1988 [ref. 1.5], where 48 papers where entered in two sessions dealing with peats and organic soils. In june 1993, an international workshop on peat geotechnics was held in Delft [ref. 1.6], with 29 papers on 1-d and 2-d modelling and construction issues. TC15 was reformed in 1994 with the Netherlands as host-nation, and its title was extended to 'Peats and Organic Soils'. Its brief is to:

- promote cooperation and exchange of information on the geotechnical behaviour of peat and organic soils,
- organize a working meeting at the 1997 ICSMFE in Hamburg,
- propose reference tests and calculation procedures for evaluation of long term settlements on peat and highly organic soils,
- explore the feasibility of a Specialty Conference on the engineering behaviour of peats and highly organic clays.

Within TC15 the need was felt to condense the accumulated experience of participating individuals and countries, into guidelines for construction on peat and organic soils. These would be designed to be of assistance to geotechnical engineers who encounter these soils in their projects. The Dutch government authority for public civil engineering works, Rijkswaterstaat (section Dienst Weg- en Waterbouwkunde) responded to this need by initiating and financing the Dutch contribution to the guidelines.

1.2 Contents of this report

These guidelines have been drawn up by assembling and harmonizing material from the Netherlands, U.S.A., France, Poland, U.K., Ireland, Sweden, Indonesia, Japan and Malaysia. It first addresses the formation, classification and terminology of peats and organic soils (Chapter 2). Chapter 3 deals with typical engineering problems related to peat and organic soils. Chapter 4 digs deeper into the determination of design parameters and safety considerations. Chapter 5 then treats calculation methods and the design process. Chapter 6 describes special construction techniques. Chapter 7 deals with the site investigations and project management and chapter 8 is dedicated to field instrumentation and monitoring techniques. Finally, chapter 9 discusses mitigation of ecological impact of civil engineering.
A wide diversity of methods between the participating nations was found, and it has proven difficult to find a common denominator. Therefore, in some cases, various methods specific to different countries have been included.

1.3 Status of the report

This report is not written as a stand alone handbook. It should be used beside the general available engineering handbooks on 'constructions on soft soils' as for example the CUR -162 book [ref. 1.7]. This report aims to give specific information on items where peat behaviour or peat characteristics deviate from the behaviour or characteristics of other soft soils.

1.4 References

[1.1] MacFarlane I.C. (1969), "Muskeg Engineering Handbook, the Muskeg Subcommittee of NRC", pp. 31-52 and pp. 127-149.

[1.2] Noto, S. (1991), "Peat Engineering Handbook", Section 7.8: Deep Mixing Method of Soil Stabilization, Hokkaido Development Bureau, 102-110.

[1.3] Amaryan, L.S. (1993), "Soft soils properties and testing methods", A.A. Balkema, Rotterdam, 180pp.

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British and foreign peats", Quarterly Journal of Engineering Geolology, London, 19, pp. 7-80.

[1.5] Proceedings II Baltic Conference of the ISSMFE, Tallinn, 1988, (2 volumes).

[1.6] Den Haan, E.J., Termaat, R.J. & Edil, T.B. (ed.), Proceedings Int. Workshop "Advances in Understanding and Modelling the Mechanical Behaviour of Peat", Delft, 1994, Balkema, Rotterdam, 428 pp.

[1.7] P. Lubking and M.A.Van (ed.), "Building on soft soils, CUR report C162, ISBN 90 54101 46 6, 1992.

2 Genesis and general properties

How to distinguish peat from organic soil is a matter of much debate, see figure 2.1. A multitude of definitions circulate in the literature, most of which differentiate between such categories as peat, muck, gyttja and organic soil on the basis of organic content.
If more pure peat occurs within a country, there is a marked tendency to put the borderline between peat and organic soil at higher organic contents. Another reason for the existing large variety of classification systems, is the very diverse group of professionals who use peat. Among those are people working in horticulture, agriculture, forestry, combustion industry and geotechnics. When a fuel is sold as a peat, according to most classification systems [ref. 2.1] it should have a mineral content less than 25%. In other, more geotechnically oriented systems, soil with more than 25 % organic contents is classified as peat.

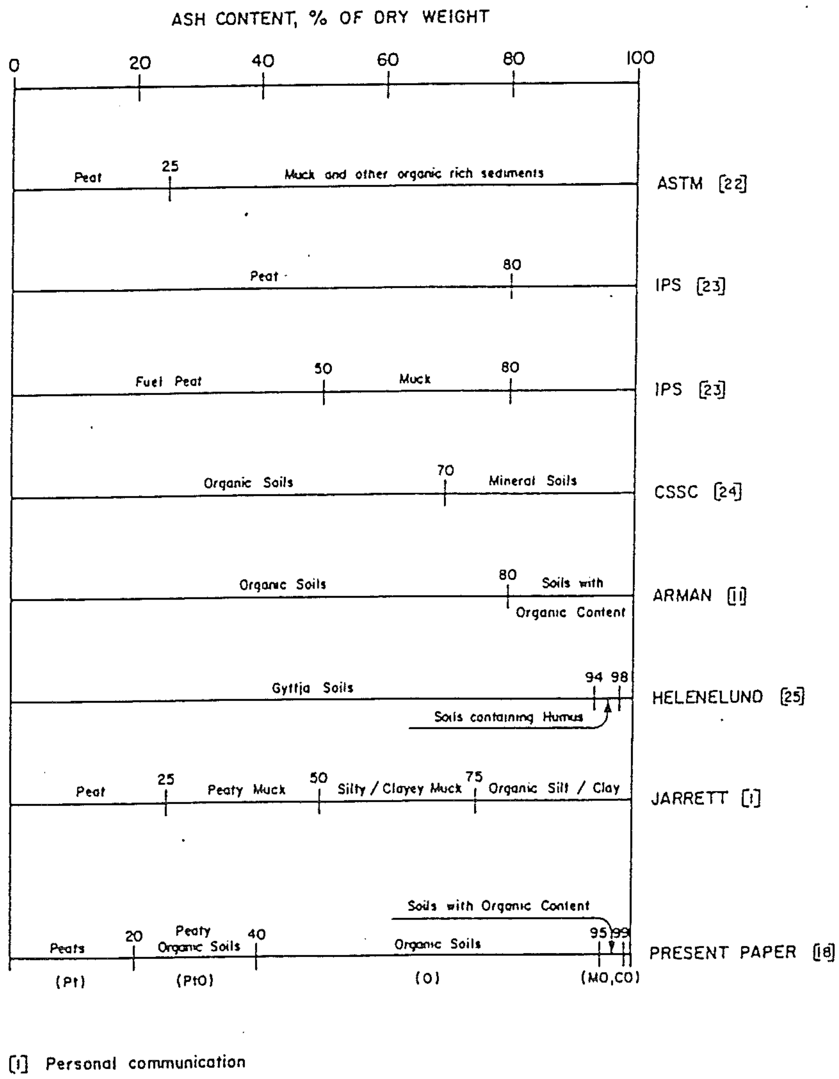

Figure 2.1 Comparison of classification systems (Landva 1983).

Hobbs [ref. 2.2] argues it is more important to recognize the morphological stage, i.e. fen, transition or bog peat. He further shows, see figure 2.2, that even at only 27.5% organic matter content, the volume of the voids in the peat is a factor 2.5 higher than the volume of the mineral component, even if this is taken as bulked clay with a water content of 100%. I.e., the vegetable matter dictates behaviour. Such a material, if it is sedentary, should be given 'pride of place' and called a (very clayey) peat. There is an approximate relationship between morphological category and organic content, as shown in figure 2.2.

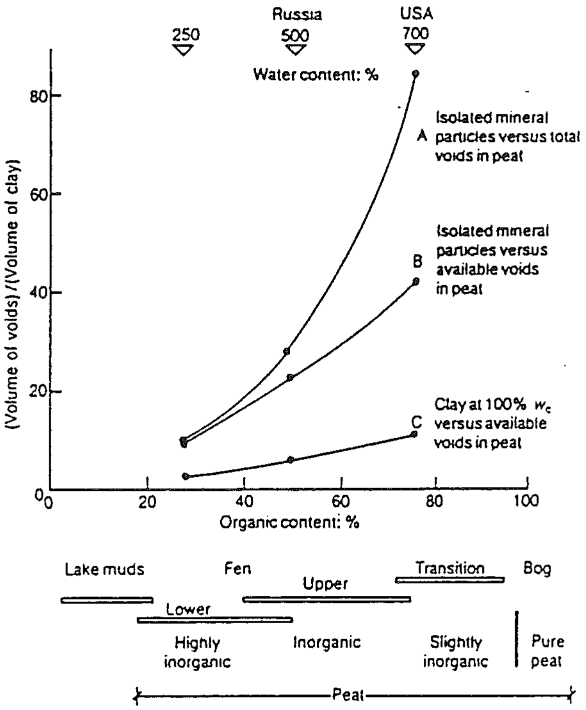

Figure 2.2 Relative volume of voids vs. volume of bulked clay.

2.1 Development of peat and its related properties

2.1.1 Genesis of peat

The following summary is derived largely from Hobbs [ref. 2.3] who gives a thorough treatment of the genesis and morphology of mires.

Very broadly speaking, peats are distinguished according to their genesis in fen, transition and bog peats. They generally form in successive stages of the wetland succession, which is illustrated in figure 2.3. In the first rheotropic stage, a supply of nutrients derived from mineral soils is brought into depressions in the landscape, lakes or basins, by flowing water, allowing the development of eutrophic vegetation such as reeds, rushes and sedges. The remains of this vegetation are conserved under water as fen peat. The early stages of lake filling often involve

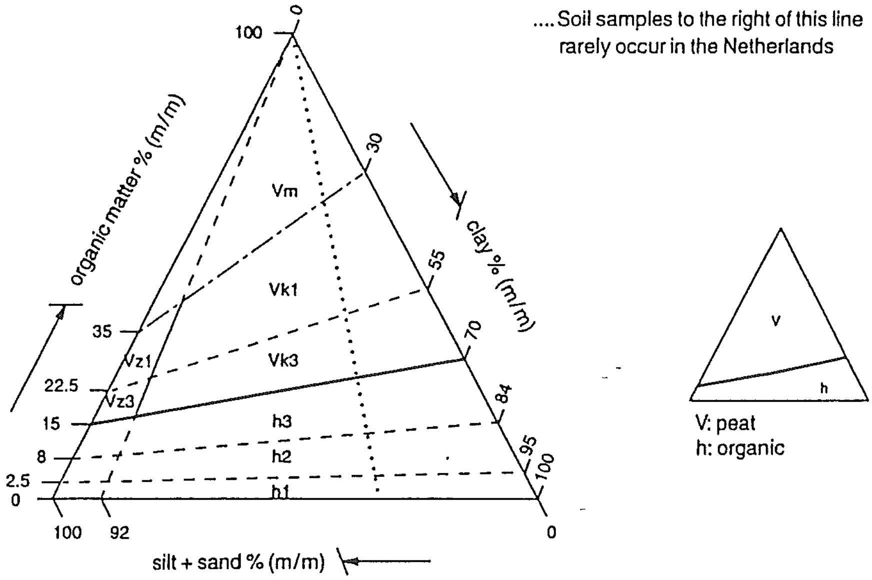

Figure 2.6 Organic matter - clay - silt+sand triangle according to NEN 5104 [ref. 2.10].

Notwithstanding the disagreement regarding classification, published relationships between various index properties commonly show a quite satisfactory degree of agreement. A recent survey of Dutch organic soils yielded the relationships shown in figures 2.7 and 2.8. One part of these results was obtained from the interfluvial district in central Holland, the other from a borehole in a peat polder near Woerden. Both sets show slight differences, which result from differences in the composition of the organic and mineral components, degree of saturation, degree of humification, conditions during formation (erosion, desiccation) etc. Although complete correlation is not to be expected, similar relationships for Hungarian soils [ref. 2.11 and 2.12] and San Joaquin peats [ref. 2.13] are quite near the Dutch results.

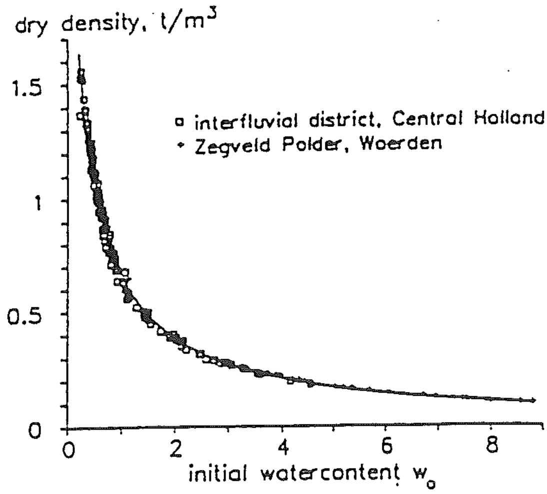

Figure 2.7 Saturated density vs. watercontent (Dutch organic soils).

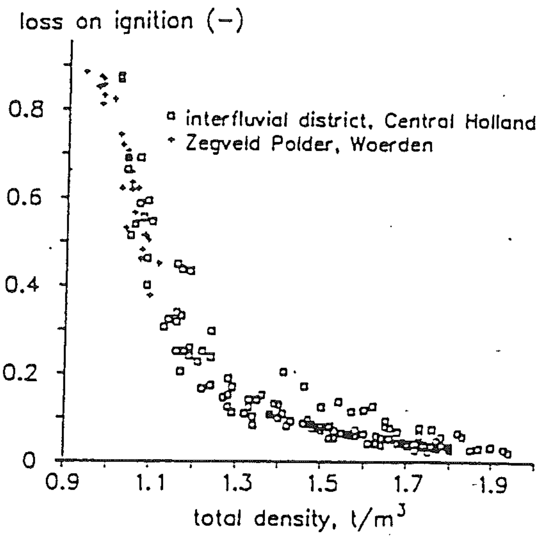

Figure 2.8 Saturated density vs. Loss on ignition (Dutch organic soils).

The results in figure 2.7 are well fitted by

$$\rho_d\ (10^3 kg/m^3) = 0.872\,(w + 0.317)^{-0.982} \tag{2.1}$$

Water content (w %) is determined at various temperatures between 60°C and 105°C. Skempton and Petley [ref. 2.14] prove that at 105°C during 24 h, loss of organic matter is limited, while at lower temperatures, small amounts of free water are retained.

Organic content can be determined in several ways, but for high organic contents, it is sufficiently accurate to simply equate organic content to loss on ignition. Temperature and duration of firing in the loss on ignition test differ, but Delft Geotechnics has good experience with 550°C during 5 h. Betelev [ref. 2.15] states that the loss on ignition method is acceptable in soils containing up to 15% organic matter, and advocates 550°C ± 50°C during 3 h. Correlations are most simply cast directly in terms of loss on ignition. Ash content is the complement of loss on ignition, and is preferred by some. Betelev [ref. 2.15] presents a method for determining the organic content of soils and rocks based on dry combustion at 500°C in an oxygen or air stream. The quantity of CO_2 production is measured by a gas analyzer. The method is economical and can also be applied to eg. oil-polluted soils.

A close relationship exists between the specific gravity of the solids in organic soils and peats, and loss on ignition N. The relationship can generally be modelled as a mixture of mineral material with a specific gravity $G_s \approx 2.7$ and organic material with $G_s \approx 1.4$

$$\frac{1}{G_s} = \frac{N}{1.4} + \frac{1-N}{2.7} \tag{2.2}$$

This equation is due to Skempton and Petley [ref. 2.14] and applies to a wide spectrum of British organic soils and peats. A similar relationship has been found for Dutch soils:

$$\frac{1}{G_s} = \frac{N}{1.365} + \frac{1-N}{2.695} \qquad (2.3)$$

It is interesting to note that from equation 2.2 it follows that volumewise, the organic constituents dominate the minerals above approx. 34% organic content by weight.

2.4 References

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[2.14] Skempton A.W. and Petley D.J. (1970), "Ignition loss and other properties of peats and clays from Avonmouth", King's Lynn and Cranberry Moss, Géotechnique 4, pp. 343-356.

[2.15] Betelev N.P. (1995), "Determining the organic matter content in soils and rocks", XIth ECSMFE Congress, Copenhagen, Vol 3, pp. 13-18.

problems related to civil constructions on peat or organic soil. Arranged by country, a number of *proto-type* constructions in some typical circumstances are described.

aspects
Organisation building industry, responsibilities, requirements
main topics
coming years construction time and dike control

3.1.1 Suburban development in peat areas

In many parts of the Netherlands, the natural soil is too soft and soggy, with a high ground water table. In case of suburban development in such areas, sand/clay fill is placed on top of the soft layers to improve bearing capacity and drainage. This provides a good foundation for roads and utilities. Nevertheless, buildings still need to be founded on (timber) piles down to the dense underlying pleistocene sand.

Pre-compression of the soft layers typically lasts 1-4 years. Vertical drains are seldom installed. Post-construction settlements can be problematic if the precompression is too short. Then, extra maintenance of roads and utilities, negative pile friction, breakage of connections between utilities and buildings are the result.

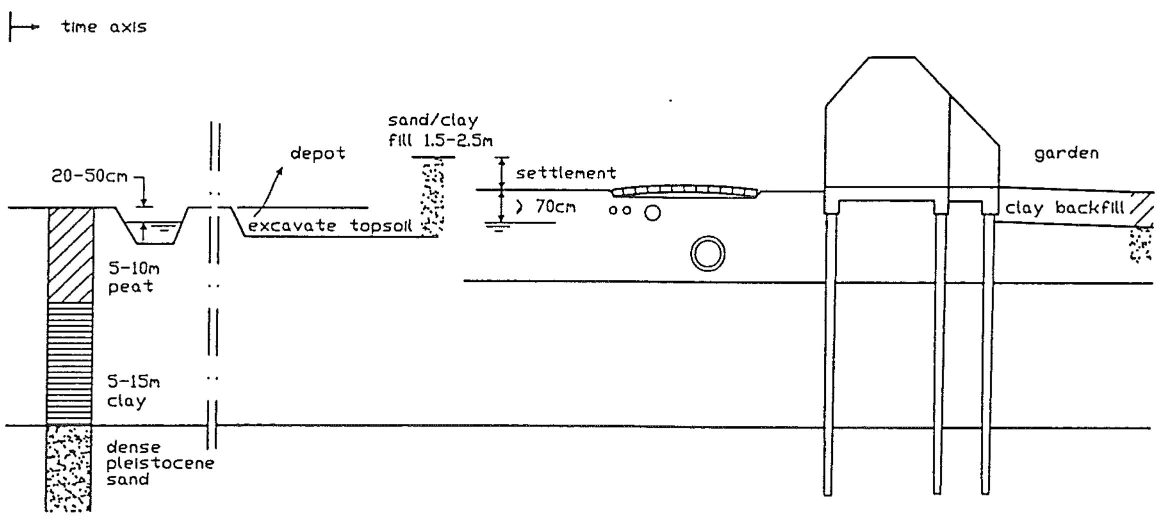

Figure 3.3 Fill placement on peat required for suburban development.

The high unit weight of sand fill leads to significant settlement. Therefore sometimes light-weight materials have been applied: flugsand (porous volcanic sand from Germany and Iceland), foamed concrete, argex (expanded clay), E.P.S. (Expanded Poly Styrene) blocks (see section 6.1.1).

3 Constructions on peat and organic soils

In chapter 2 the specific behaviour of peat and organic soils has been described. The high compressibility, the long term behaviour and the low shear strength cause specific problems in designing and realisation of constructions on such type of soil. The most common problems are embankments for roads, railways, dams and dikes. Less frequent problems are swallow foundations and excavations, whether or not with a retaining construction.
The engineering problems are not only characterized by the properties of the peat and organic soil layers and the type of construction, but also by the design requirements, which are effected by the economical conditions and the organisation of the building industry.
As examples can be mentioned:
- construction time: important in areas with high industrial and economic activities.
- control and maintenance: roads, dikes and dams; can traffic jumps be acceptable
- construction space: environment and inner cities.
- availability of materials.
- environmental problems
- responsibilities: design, construction, maintenance, research

To illustrate the engineering problems, a selection of countries is made in which we met different conditions: Netherlands, Indonesia, Japan, Canada, Ireland, France, Sweden and Russia.

3.1 The Netherlands

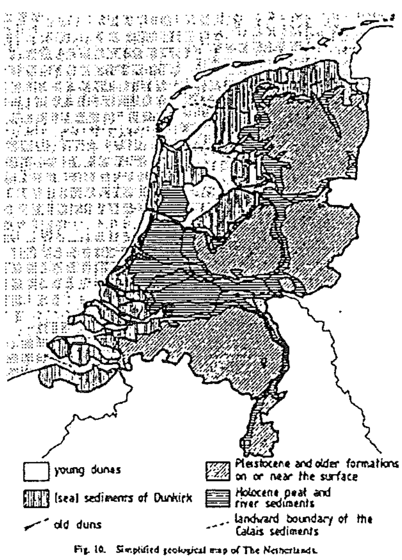

Figure 3.1 Map with occurence of peats in the Netherlands.

Peat characterisation
The occurence of peat in the Netherlands is shown in figure 3.1.

Building activities
In the Netherlands the main activities are for roads, railways and dikes and next to that retaining constructions.

In the Netherlands, stability of river dykes is affected by peat, especially due to uplift of peat layers adjacent to the inner toe by high river levels, figure 3.2. Levees in the San Joaquin delta founded on peat have been known to slide en masse during failure due to extreme high water loading: Duncan and Houston [ref. 3.1].
Embankment widening or dyke heightening causes distress to the existing road, services buried near the toe and adjacent buildings.

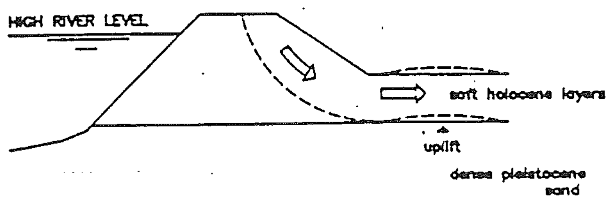

Figure 3.2 Uplift of toe reduces stability and increases lateral deformation of river dykes.

The transfer of loads from building foundations to peat is preferably avoided by replacement or use of piles. Otherwise, the effects on settlements and bearing capacity need evaluation during design [ref. 3.2].

Suburban development in peaty areas requires the application of a sand cover to increase ground level sufficiently above ground water level, and to provide a clean environment for services and road bases. However, placing sand fill on peat will often induce large settlement.

Canal dykes in Holland frequently contain peat. Flow of aerated and nutrient-rich water through the dykes may gradually lead to decomposition of the peat. Some occurrences of sudden dyke settlements have been attributed to this cause [ref. 3.3]. Vonk [ref. 3.4] describes how excessive desiccation of the slope of a dyke along the Rotte containing organic soil, lead to increased seepage, erosion and near-failure. Vonk also provides evidence for the occurrence of methane gas pockets under dykes. While these probably will not endanger stability significantly, they may be a hazard during field penetration testing.

The following sections deal with some special construction techniques, applied to cope with

Prediction of settlements is necessary to quantify in advance the amount of sand, the duration of precompression, and the post-construction settlements. A method developed by Fokkens (see section 5.1.1) is often used to predict final settlement in peat. Settlements are monitored by settlement plates. Some piezometers are placed to gauge pore pressure development.

3.1.2 Embankment widening

Many expressways and railways are being widened, to cope with increasing traffic. Embankment widening on peat soils causes distress to the existing road/rail.

Existing roads have often been constructed on a sand body created by dredging a canal and infilling it with sand (cunette method). It is not possible to widen the cunette without inducing collapse, so soil improvement rather then soil replacement must be resorted to.

Surcharge is traditionally applied in layers. Vertical drains are used to accelerate settlements (see section 6.1.2). However, it has been shown by Delft University of Technology that only 1% rotation of the surcharge is sufficient to damage the existing road. Settlements can amount to 2.5 m below the new embankment. Lateral deformations are up to 1 m, and affect utilities buried near the toe, and adjacent structures.

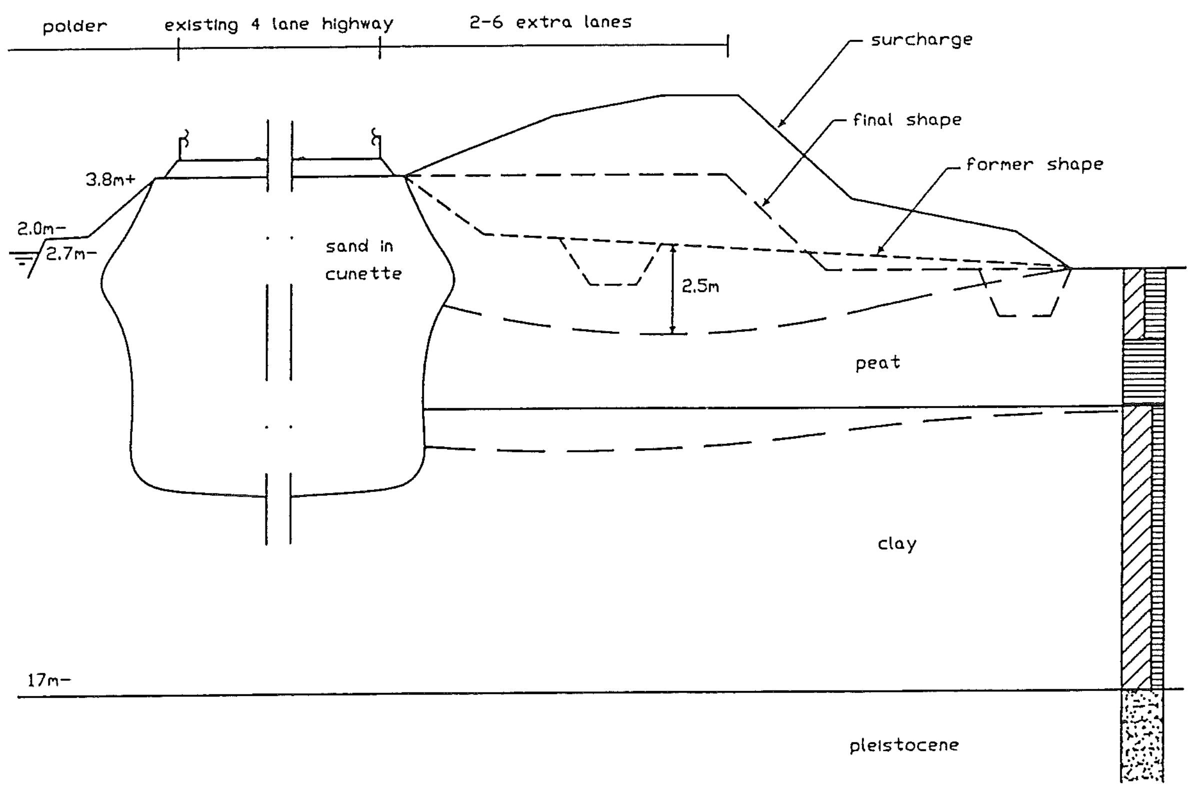

Figure 3.4 Embankment widening.

Solutions include:

- new embankment at a distance away from the existing embankment (A12 The Hague - Utrecht)
- stone columns (trials along A2 Abcoude. See section 6.1.2 and 6.1.4)
- vacuum preloading (A16 Rotterdam, dyke strengthening project Streefkerk. See section 6.1.3)
- gap method: build surcharge from outside - in, thus increasing arching and reducing lateral deformation (trials along A16, Rotterdam and A2, Haarrijn).

Application of lime cement columns (see section 6.1.4) is presently being investigated. If these can be designed to give a sufficiently large increase of strength despite the acidity and variability of the peat and organic soils, they may possibly effect a revolution in the Dutch practice of soil treatment. Greatly reduced construction periods, much less damage to existing adjacent structures, greatly reduced settlements and horizontal displacements, and greatly increased stability would result.

3.1.3 Stability of river dykes

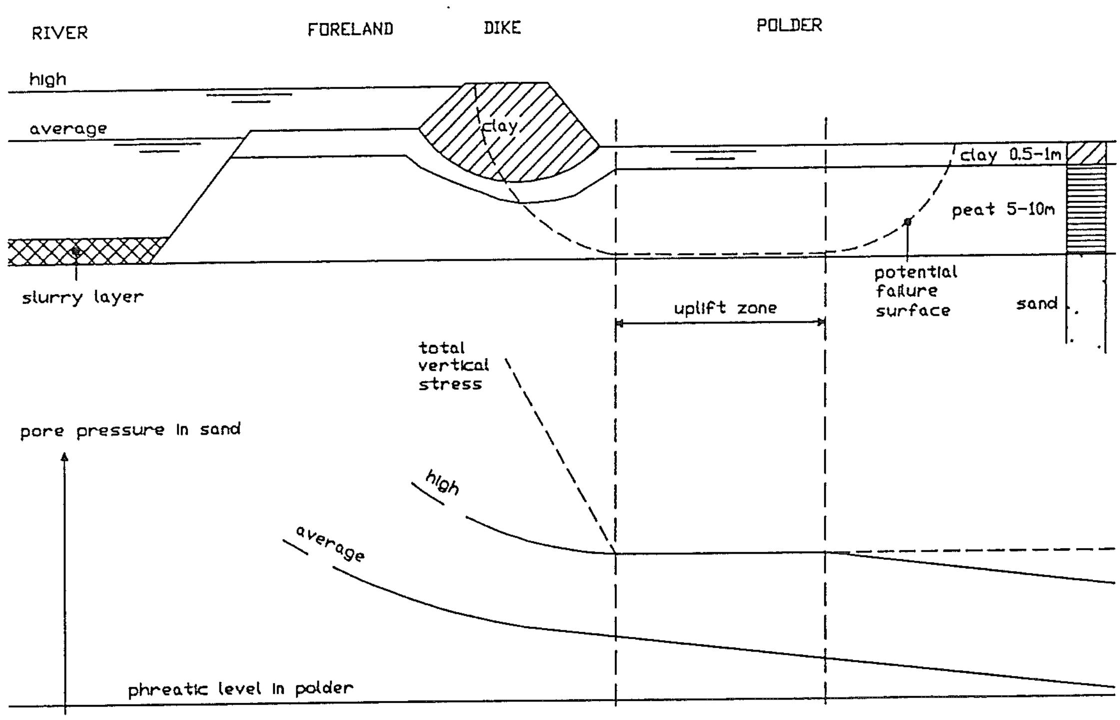

Figure 3.5 Stability of river dykes.

The stability analysis of river embankments is performed by two types of calculations:

- conventional Bishop stability analysis (see section 5.2.1)
- if appropriate, analysis of stability during uplift (see section 5.2.2)

In peat areas, stability during uplift is usually most critical. Problem, design methods and counter measures are described below.

Peat layers of significant thickness are present in the subsoil of many polders in the lower river area of the Netherlands. The soft layers are underlain by a sand stratum, which is in good hydrological contact with the river. Pore pressures in the sand are directly related to the river level. Because of the low vertical total stress at the peat-sand interface, pore pressures in the sand during high river discharges and storm surges cause uplift of the soft layers and a loss of shear strength at the interface. The load of the dyke causes horizontal compression of the uplift zone and possibly failure of the soft layers. Failure surfaces are typically non-circular with a horizontal section along the peat-sand interface. Even without failure, the high compressibility of the peat results in large deformations in the uplift zone and damage to the dyke body.

A complete stability analysis comprises the following steps:

- determine critical pore pressures during high river discharges and storm surges by extrapolation of measurements during less critical circumstances,
- determine geometry, soil weights, effective strength parameters, deformation parameters and initial horizontal stress,
- calculate the stability of circular failure surfaces using the Bishop method,
- if uplift or near-uplift occurs, perform analytical or numerical stability analysis based on non-circular failure surfaces,
- if the stability is sufficient, check for excessive deformations of the uplift zone and the dyke body.

Counter measures for insufficient stability include:

- application of a sand berm behind the dyke, restoring shear strength along the peat-sand interface,
- widening the existing dyke towards the river and partial excavation of the existing dyke body, reducing the load on the soft layers in the uplift zone,
- partial excavation of the existing dyke and construction of a L- or U-shaped retaining wall founded on piles to the sand stratum, reducing the load on the soft layers in the uplift zone
- construction of coffer dams or diaphragm walls in the existing dyke, reducing the load on the soft layers in the uplift zone.

3.1.4 Polder subsidence

Subsidence of peat areas due to decomposition (oxidation) is often extreme. Some estimates: some areas in Northern part of Holland up to 1.5 m in the last 450 years; in the western parts of the Netherlands, up to 2 m in the last 1000 years. Rate of subsidence increases drastically when ditch levels which control the water table are lowered eg. for agricultural purposes: Schothorst [ref. 3.5] found a threefold increase, from 2 to 6 mm per year, when the traditional shallow water table at 20-40 cm depth was lowered by 50 cm in experimental fields in Holland.

Legal acts control lowering of water tables in Dutch polders. Levels are adapted once every 10 years, correcting downwards for the subsidence in the preceding period. To guarantee minimum depths of drainage, it is being contemplated to index these levels, i.e. to lower ditch levels such that the guaranteed level is not exceeded in the coming period. This requires predictions of subsidence, including such effects as rate of oxidation, shrinkage etc. Such predictions would profit from application of modern compression theories for soils based on unique stress - strain strain rate relationships, accounting for loss of material due to oxidation at the surface.

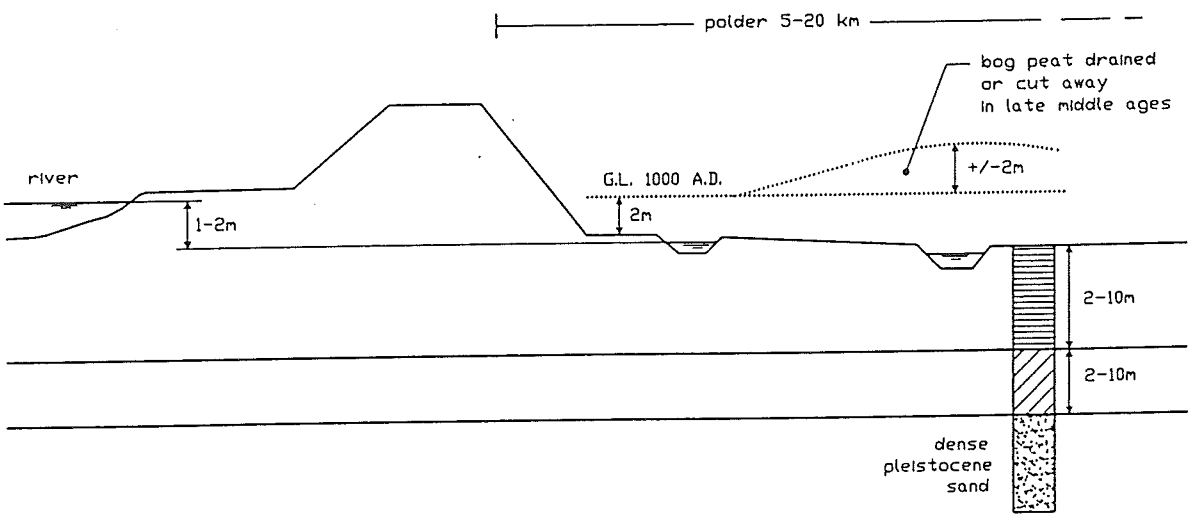

Figure 3.6 Typical section of a polder.

Dutch polders in peat areas subside at rates of 3 - 20 mm/year. This figure is related to the present day freeboards of 60 - 80 cm. This was much lower historically: about 20 cm, resulting in some 2 m of subsidence from about 1000 A.D., when the marsh reclamations in Central Holland started. Most of this historical subsidence is due to bio-degradation: only a small part is due to shrinkage of the topsoil, and a negligible amount is due to consolidation of the peat below the ground water table.

Dykes and dams came into existence when the ground water levels had dropped to river level. Up to then, drainage was entirely by gravity, now it was by gravity at low tide through trapdoor sluices in the dams which blocked off small rivers at the confluence with tidal rivers. Around 1400 A.D., subsidence had reached a level requiring mechanical drainage, and this greatly stimulated the development of windmill technology in Holland. These polders were therefore not *wrested* from the sea, as popular myth has it, (that is true of reclamation work in other parts of the Netherlands) but almost *given* to the sea!

Subsiding polders create a relative increase in ground water table. Because of the diminishing area of farming land in Holland, some polders are being returned to a wetland state. In the remaining polders, ground water tables
are regularly lowered to correct for the subsidence. Legal acts regulate this. The present method is to correct the subsidence of the past 10 years in one go. It is being investigated whether or not the corrections could take place more regularly, say once or twice a year, thus guaranteeing a certain freeboard to land users. An indexation system is to be developed for this. One question to be answered is whether or not the overall rate of subsidence will increase in the new system. A consolidation theory incorporating material loss due to bio-degradation will be developed to assist in defining the indexation system.

3.1.5 Monitoring of staged construction

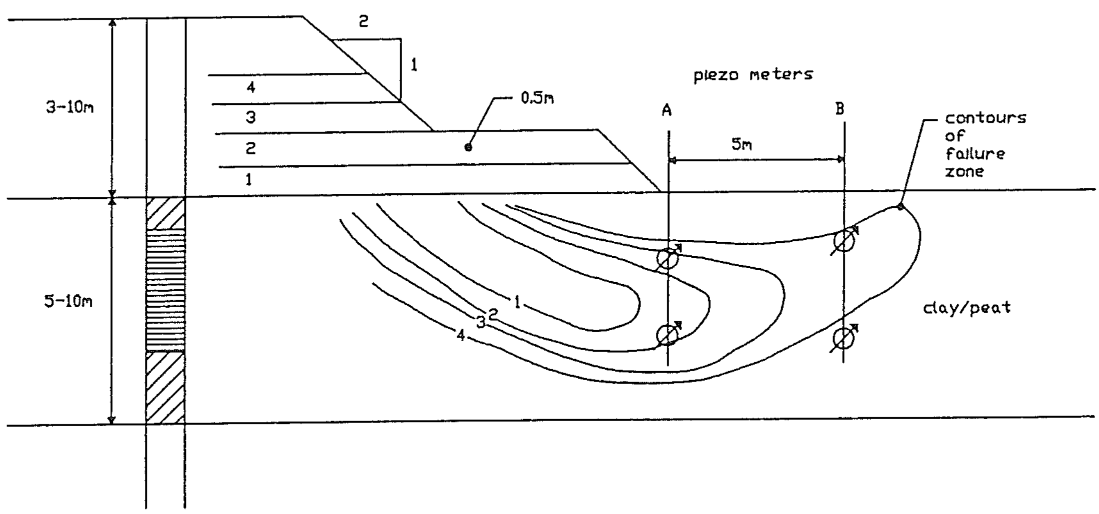

Figure 3.7 Stability monitoring using piezometers.

Placing road and river dyke embankments on soft soil generally requires staged construction because of low strength and poor consolidation characteristics of the subsoil. Predictions of the rate of filling suffer from uncertainties in subsoil shear strength, pore pressure distribution and pore pressure dissipation rate. Monitoring the construction operation with piezometers has been shown to be a cost-effective method for minimizing the risk of embankment failure.

Monitoring of staged construction requires the following actions:

- prediction of expected behaviour in terms of parameters to be monitored with an initial set of soil and embankment properties
- adjustment of the initial set of properties to match observed behaviour in early stages of filling
- assessment of the possibilities for applying the next stage.

Monitoring of staged construction is best performed on the basis of pore pressure measurements because of the fast response of pore pressures to stress changes and the direct relation with effective stress and therefore shear strength. Preferably measurements of settlement (surface settlement plates) or horizontal deformations (inclinometer tubes) and visual field inspection should be used to support conclusions derived from pore pressure observations (see chapter 7).

Analytical and numerical methods are available to predict pore pressure generation by embankment loading. Preferably two rows of piezometers (A and B) are applied; the filling rate is chosen as to confine the front of the failure zone (zone of full mobilization of available shear strength) between the rows A and B.

Monitoring can be used successfully only if modifications to the filling scheme are possible within the contract applying to the operation. In the Netherlands contracts tend to become more and more rigid in this respect.

3.1.6 Peat and vibrations

Mainly two aspects concerning peat and vibrations are typical research items in the Netherlands. Train passage on railways, trucks on highways and roads as well as machines cause vibrations which travel trough the soil medium and enter a nearby building. The tolerated vibration level in buildings is limited. Especially in peat layers low frequencies (with higher energy capacities) can be transmitted due to low wave velocities.

Secondly the stability of track gives problems if high speed trains reach or pass the critical wave velocities in the soil. The low wave velocities are typical for peat.

Both items are in a state of research. No general design methods have been developed, but for typical cases finite element methods are used. The parameters for the models are found from laboratory tests such as free vibration torsion tests and bender elements in triaxial tests as well as from measurements of wave velocities in field tests. Also an approximate correlation for the dynamic shear modulus of peat with CPT q_c-value exists (i.e. $G_{dyn} = 3\ q_c$).

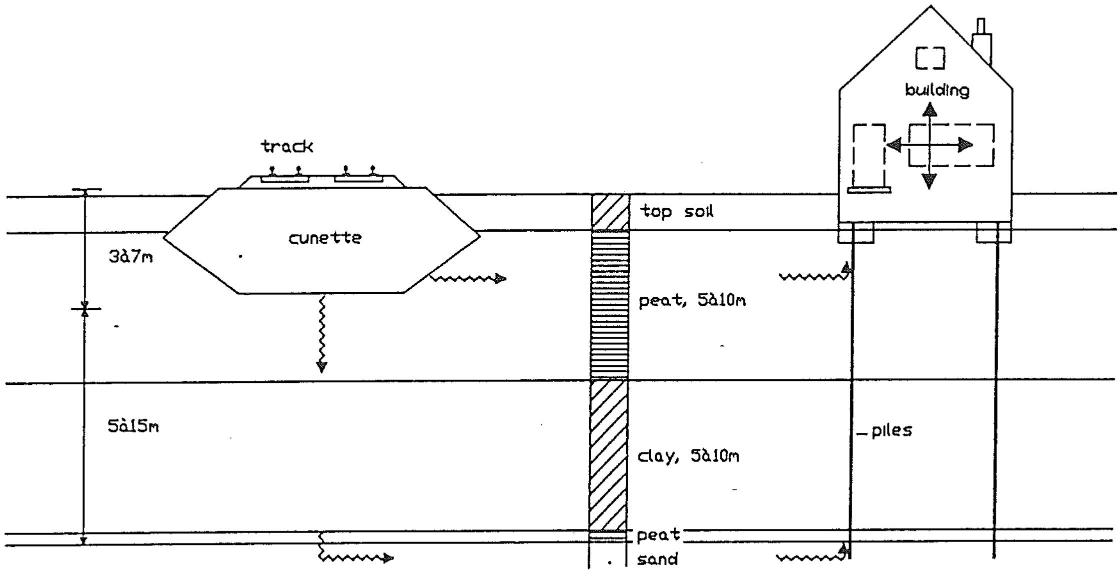

Figure 3.8 Transmission of shear waves.

The shear wave velocity V_s in peat can be calculated from

$$V_s = \sqrt{(\frac{G_{dyn}}{\rho})} \tag{3.1}$$

Therefore the shear wave velocity is mainly dependent on the dynamic shear modulus G_{dyn}. Measured values in laboratory for G_{dyn} for surface peat layers are in the order of 0.5 to 1 Mpa, what results in shear wave velocities of around 20 m/s. Field measurements of shear wave velocities confirm these values. In the peat layers below the track values of 2 or 3 Mpa can be measured.

Another item is the settlement, for example of small dykes, due to traffic vibrations. Also an item that recently is mentioned as a possible problem is the influence of vibrations due to ship motors on the stability of saturated dykes.

3.1.7 Small polder roads

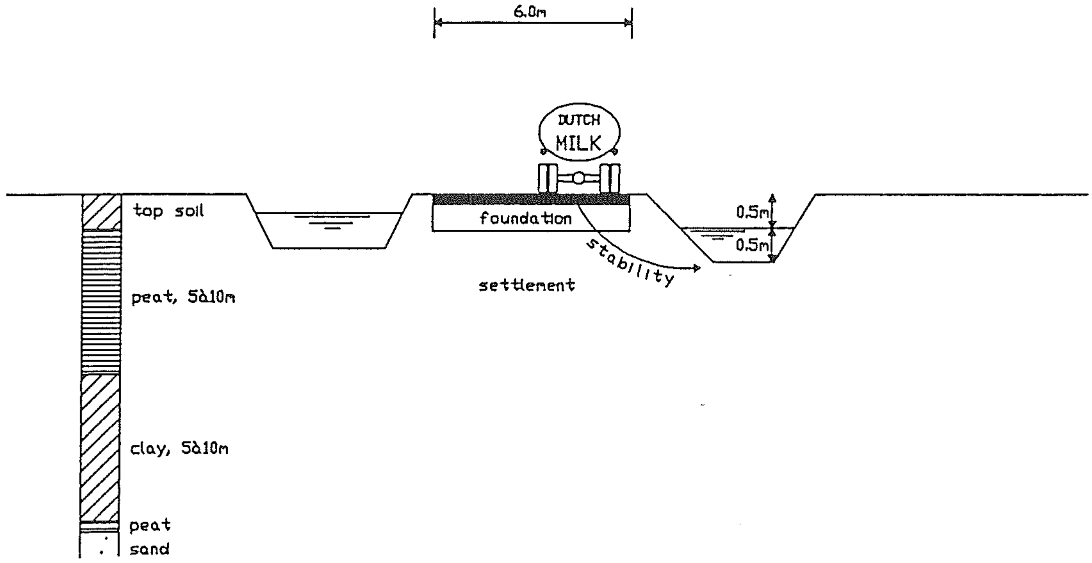

Figure 3.9 Section of a typical polder road.

Maintenance of small polder roads with thick (5 à 10 m) peat layers consumes a lot of money each year. Due to polder subsidence (see specific prototype) and the weight of the road, the road's foundation regularly sinks below the ground water table. Water enters the foundation layer of the road and causes damage during frost periods.
Another cause for damage is the heavy loading due to for example milk trucks. Normally on both sides of the road ditches are present. The slopes of these ditches are insufficiently stable under the load of heavy trucks. This leads to excessive deformations of the road.

Reconstruction of the road is done by a so called cold recycling method. The old foundation layer is excavated, broken, mixed with some cement and than reused as new foundation. The experience with this method is very positive and research is being conducted to optimize the method.

Some reconstruction works are carried out without excavating the old foundation layer. Then the thickness of the road foundation accumulates (upto 1.5 m is found) and the problem with settlements can aggravate.

Design methods for settlement of the road and slope stability of the ditches are similar to the methods used for large structures as dykes and highways. Special soil improvement techniques are not yet used for these small roads.

Light weight fill materials (see section 6.1.1) may be applied to raise the foundation above the ground water level and to stop the settlement. Reinforcement of the pavement can be effective to prevent deterioration due to insufficient stability of the ditch slopes.

3.1.8 Horizontal displacements

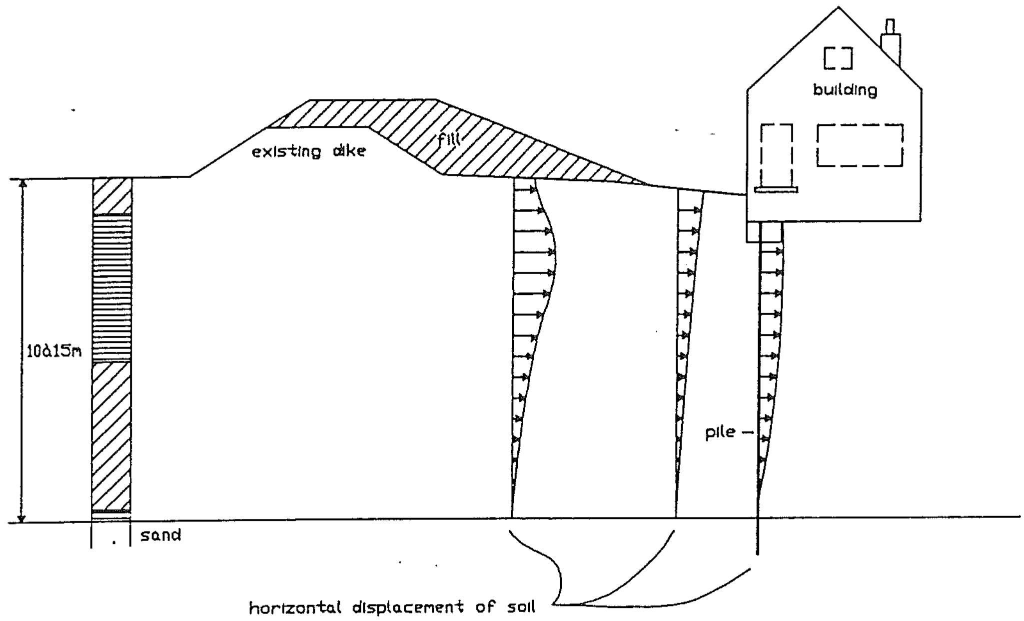

Figure 3.10 Widening of the dike induces lateral forces on piles.

Due to widening projects of roads and dykes horizontal deformations in the soil will result. Nearby the buildings or pipelines are submitted to these deformations. In peat layers the horizontal deformations (as the vertical) are very large.
For the calculation of the effect of horizontal deformations due to soil fills on foundation piles a so called method 'De Leeuw' (see section 5.4.1) is very often used. The horizontal displacement is calculated in one deformable layer on top of a solid base. The horizontal deformation of the upper boundary of the layer can be taken as either free or fixed; for the pile support conditions a arbitrary degree of fixity at the upper and lower boundaries can be assumed. The deformation in the layer is combined with the possible deformation of the pile.

More recently finite element programs with for instance a Mohr-Coulomb model are also used. The main problem with these models is that if the horizontal deformations are fitted to measurements the vertical deformations are too large (i.e. factor 2). More sophisticated models, for example cam-clay models may give better results.
Parameters for the finite element method are found from inclinometer tubes or pressure meter tests, from laboratory triaxial tests or from correlations with unit weight, CPT's or oedometer tests.

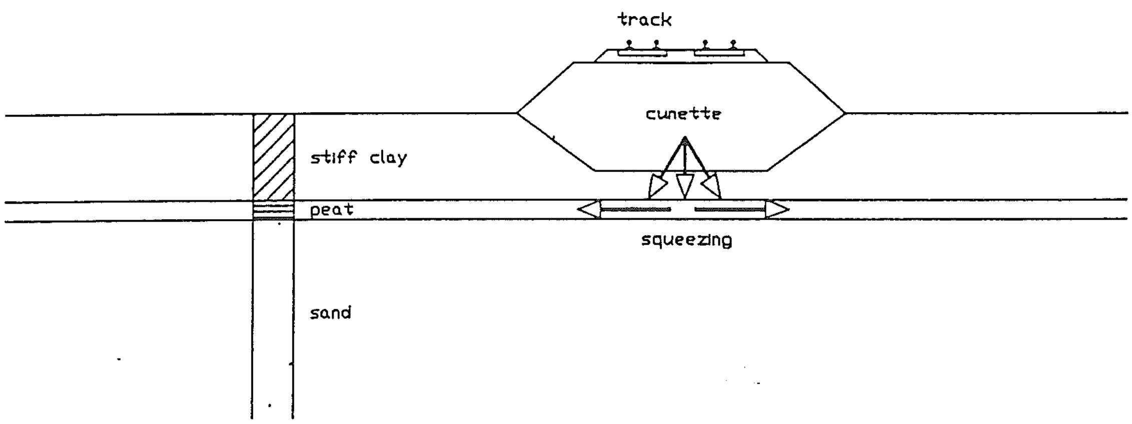

Figure 3.11 Squeezing of a peat layer.

If the load on a relatively thin peat layer is too large, squeezing of this layer can occur. In the past this effect has been used as a soil replacement method, but is not recommendable.

The so called method 'IJsseldijk' (see section 5.2.3) is mostly used to check if squeezing of small peat layers (around 1 meter thickness) can occur. This method is based on the vertical equilibrium in the peat layer besides the embankment. Other methods as i.e. 'Mater-Salençon' are based on horizontal equilibrium of the peat layer.

3.1.9 Underwater placed fill on peat

constructions (fill) on peat below the watertable.
(to be inserted)

3.2 Indonesia

Indonesia is a tropical country in which consists of thousand islands has a great varieties of soil deposits, and one of them is peat soils, concerning to the wetlands in Indonesia. As we know some parts of Indonesia are covered by wetlands, such as: North and East Sumatra, most of Kalimantan, Sulawesi and Irian, see figure 3.12.

Large areas of Sumatra and Borneo in particular contain significant quantities of peat soils, sometimes to depth of several metres, in turn underlain by soft to firm cohesive strata. These peats are young and fibrous and very highly compressible. In many cases there is evidence of little organic breakdown with leaves and twigs of trees and bushes clearly shown. Natural moisture contents can be in the order of 800-1000% (3)

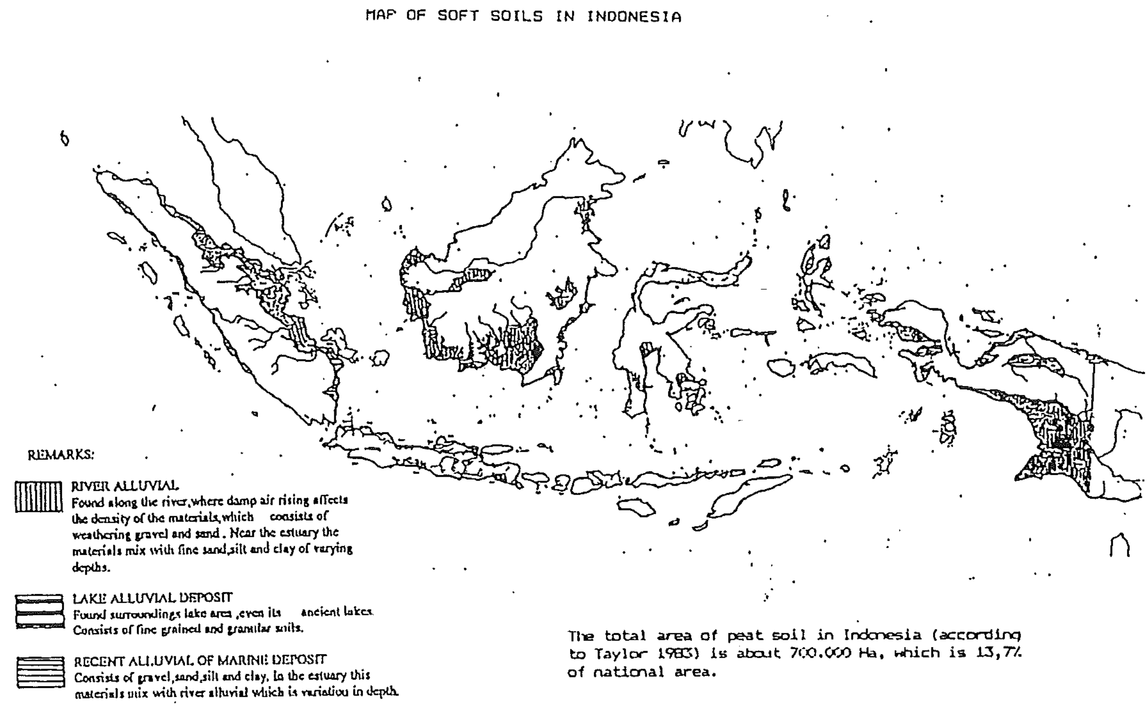

Figure 3.12 Map with occurence of peats in Indonesia.

The example of peat characteristic and classification in Indonesia (Semarang - Central Java) is almost the same with peat soil from another areas, as follows:

I) Von Post classification:

1. Degree of humification - (H6)
2. Water content > 500% - (B3)
3. Much of fine fibre content - (F3)
4. Less of fresh fibre content - (R1)
5. Wood content (medium) - (W2)
6. Organic content (high) - (N4)
7. Vertical strength (low) - (Tv1)
8. Horizontal strength (zero) - (ThO)
9. Low plasticity - (P1)
10. Moderately acidic (5) - (pH1)
11. Botanical composition - (imperata sylindrica - otela - woody peat)
    (ilalang) (eceng gondok) (kayu)

II) Physical properties:

1. Shear strength = 0.17 kg/cm2 (very low)
2. Compressibility index Cc = 3.5 - 6.3 (high compressibility)
3. Coefficient of Consolidation Cv = 1-6 x 10 cm/sec (high)
4. Thickness = 1-5.20 m.

Because of the development in some parts of Indonesia where peat deposits are extensive, highway, agriculture and residential areas must be found for the economy

A particular situation exists in the large scale lowland development projects in Central Kalimantan. In these areas large peat deposits are present. The construction of roads, bridges and culverts will have to accommodate to these conditions.

- Applied research of local materials suitable to reduce the compressibility of the subsoil and to increase the shear strength.
  Possible options are reinforcement of the subsoil by piles (cerucuk, bamboo, gelam) or by horizontal elements, like mattresses in local materials or geotextiles, see figure 3.13.
- The use of local soil as fill material for the embankments.
- land drainage
- Environmental problems

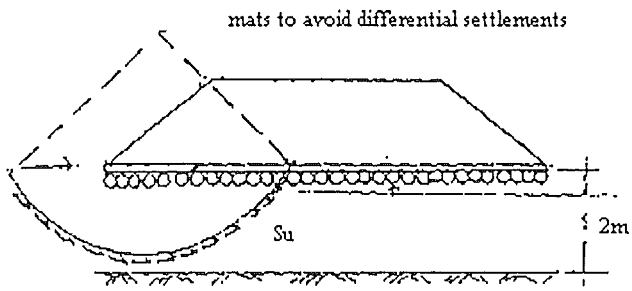

Figure 3.13 Mats to avoid differential settlements.

3.3 Other countries

> > > > > > please submit prototypes < < < < < <
for example: earthquakes, tropical conditions etc.

3.4 References

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[3.3] Delft Geotechnics (1981): LGM-mededelingen XXII-1, 56p, Visser R. Ch.: On the weathering of peat.

[3.4] Vonk B.F. (1994): Some aspects of the engineering practice regarding peat in small polder dykes, Delft Peat Workshop.

[3.5] Schothorst C.J. (1982): Drainage and behaviour of peat soils, Proc. Symp. Peatlands below sealevel,. ILRI publication nr. 30, Wageningen.

4 Geotechnical characterization

The large compressibility of peats induces large settlements, which in turn lead to an increase of the thickness of the fill necessary to reach a given final level of the fill platform. In some cases, the final height of the fill cannot be obtained with reasonable costs and bridges are then designed instead of embankments.
In the last fifteen years, designers tended to avoid construction of fills on top peat layers and to replace the surface peat layers by granular fill material. The replacement of peat is usually executed by dredging or by means of an excavator. Care must be given to the lateral extent of the excavation. Fibrous peats are more easily extracted by means of an excavator. The lateral slope of this type of excavation may be very steep in fibrous peats, provided the excavation is made under water.

Classical methods of improving the soft soils are used in the case of peats, too. These include preloading for controlling the long term settlements, stage-construction for the construction of fills which would be unstable if they were constructed in one phase, installation of stone columns and total or partial replacement of the peat. Vacuum preloading has been used too in recent years. Nevertheless there are several problems that have to be solved for the design of peat improvement. Creep and the large variations of the compressibility and permeability of peat must be carefully taken into account in the design.

All these items together underline the importance of high quality soils investigations, in the design, construction and maintenance phase of a project. Furthermore, for design safety factors have to be defined.

4.1 Stages in the design process

From a practical point of view, the questions that must be answered when construction works are planned in a peaty area, are related to the horizontal extent and thickness of the peat layer(s) and their short and long term geotechnical behaviour. The short term behaviour of the peat may dictate the planning during construction phase, whereas the long time behaviour of peat may influence the performance of the construction during its designed life time.
Therefore, in all stages of the construction process, knowledge about the extent and characteristics of the peat are required.

In the preliminary design phase, filtered aerial photographs or remote sensing images for detecting humid zones under the ground surface give a possibility to assess quickly the extent of the peaty zones at the site. Also geological maps might be helpful in gathering information about the occurrence of peat and organic soils in an early project stage.

When the preliminary design phase turns to the detailed design phase more information about the sub surface is required and to detect buried layers of peat, soundings and borings are required. More detailed information comes from in-situ and laboratory tests which will provide information about the water contents and mechanical properties of the peats (see section 4.2.2 and 4.3 respectively).

The fibrous or amorphous state of organic matter in the peat layer will influence the quality of the samples taken from the boreholes (see section 4.2.1). Amorphous peats are easier to extract in a relatively intact state. Fibrous peats need to be sampled by means of larger core samplers. In both cases, special care must be taken of the conservation of the initial state of the peats, which can loose much of their natural pore water if they are not properly packed. Special equipment will sometimes be needed for working on peat bogs with low bearing capacity of the ground surface.
Laboratory tests will include a detailed estimation of all the relevant compressibility, shear strength and permeability parameters of the in-situ peat, and of their variations with the densification of the peat under the loads applied to it.

During the actual construction phase and even in the maintenance phase, monitoring is often required to determine the stability of applied loads and to further predict settlements (see chapter 7).

Related to the various design phases, the amount and detail of site investigations and laboratory tests required will increase during the development of a project.

4.2 Site investigations

< < < < < paragraph about remote sensing and geological maps to be included. > > > > >

Site investigations are required to gain knowledge about the soil properties, to enable design of civil constructions. Often the following soil parameters are required:

- unit weight and watercontent
- strength characteristics
- deformation characteristics
- percentage organic matter (loss on ignition).

These parameters can be obtained from in-situ tests or from laboratory tests executed on field samples. Beside the mentioned laboratory tests, peat samples should be examined visually. A visual description may give valuable additional information on the mass characteristics of peat and may aid the interpretation of test results.
During the evolution of the project, the soil properties have to be known with increasing accuracy. While in the preliminary design phase, often determination of unit weight, water content and loss on ignition may be sufficient, in the detailed design phase, detailed knowledge of the strength and deformation characteristics of the peat may become important.

Unit weight and water content

Unit weight of dry soil and water content are important for calculating stress state at various depths. Also, data are required about groundwater level and degree of saturation. Soil, beneath the water table, can generally be assumed to be fully saturated. Above the water table, especially in the case of cohesive soils, allowance needs to be made for completely or partially saturated soil.

Loss on ignition

The loss on ignition or ignition loss, is a measure for the amount of organic constituents. There are numereous correlations between loss on ignition and engineering parameters. As the determination of the loss on ignition is relatively fast and cheap, it is a very useful test which helps in the determination engineering parameters.

Strength characteristics

Soil shear strength is quantified by Coulomb's law via cohesion and the angle of internal friction which are particularly dependent on soil type, degree of saturation and degree of deformation. Owing to the deformation-dependence of shear strength, this needs to be determined by in-situ and laboratory investigation at several representative deformation percentages.
The greater the deformations, the greater the proportion of shear strength mobilised. In the case of firm clays and sand, this mobilisation takes place at smaller deformations than for less firm clays or peat.
Primarily for calculations in which deformations are important, not only the maximum shear strength plus the corresponding percentage deformation and residual strength, but also change of shear strength with increasing deformation are important.

The strength of natural peat does hardly increase in the depth direction, and it does not show the changes in density and water content often found in usual alluvial clay. Further, the engineering characteristics may be affected by the organic and water contents.

Deformation characteristics

On the whole, in deformation behaviour and particularly during soil consolidation, vertical deformations are the largest because the vertical pressures are greatest. Sometimes the horizonal deformations can likewise be important, for instance, when pipes or adjacent structures are present. Furthermore, in highly compressible strata, the horizontal deformations will affect the magnitude of the vertical deformations, i.e. settlements. When determining settlement behaviour, a distinction must be made between:

- the preconsolidation pressure
- the compressibility above and below preconsolidation pressure
- the primary and secondary settlement.

The preconsolidation pressure can deviate from the overburden pressure owing, for instance, to earlier pre-loading and/or lower groundwater levels or because the site has recently been excavated.

The modulus of elasticity is deduced from the non-linear stress/strain curve and is hence dependent on strain magnitude.

4.2.1 Drilling and sampling

Whilst executing various borings, at the same time samples can be taken for laboratory analysis. In order to obtain undisturbed samples, resort should be made to techniques in which the sample is not mixed with water. During flush drilling, the quality is inadequate, from the soil mechanics standpoint, whereas the quality during bailer boring, depending upon the type of soil and the sample test envisaged, may be acceptable. The taking of undisturbed samples calls for a careful approach in which not only the sample tube penetration method but also the sample tube size mode of transport and storage need particular attention. The optimum sampling method for peat depends on the peat texture and water content.

As far as mode of sampling in an open bore hole is concerned, push sampling is preferred to drive sampling, particularly when taking samples from low load bearing and highly compressible strata. With drive sampling, there is always a strong chance of the sample being disturbed as the result of the impact energy deployed. The firmness of the soil largely determines the sampling depth obtained. In the process, both the maximum cone resistance of the stratum from which a sample must be taken and the maximum push capacity available (usually 200 kN) are determinants.

Sampling methods

Fixed piston samplers
Thin-walled sampling is generally used for taking undisturbed samples. In the Netherlands, sampling tubes are usually driven into the ground with the help of the Ackerman unit. Generally, this is done by installing a fixed piston type thin walled steel sampling tube beneath the cutter

unit. Once installed at the bottom of the bore hole, the tube is driven into the ground with the aid of an internal drop weight. Then, the entire arrangement is pulled out of the ground under reduced pressure (by closing off the top of the tube) to prevent loss from the tube. A sample taken by thin-walled sampler is sealed at both ends. Samples for laboratory tests are obtained without disturbing the peat by cutting the thin-walled tube in round slices.

Split spoon samplers

Similarly, with the aid of the SPT unit, a sampling tube ('split spoon') is driven into the soil from the bottom of the bore hole. The tube is thick walled so that the operation presents a disturbed sample, if a sample is obtained at all. Therefore this method should not be used.

Peat samplers

Disturbed peat samples, without changing the layer order, can be obtained using the peat sampler. Today samplers with cover, as shown in figure 4.1, are commonly available. Peat samplers with closed covers are forced vertically into the substratum to the required depth, and rotated clockwise to open the cover, then the edge of the cover scrapes the surrounding peat into the sampler.

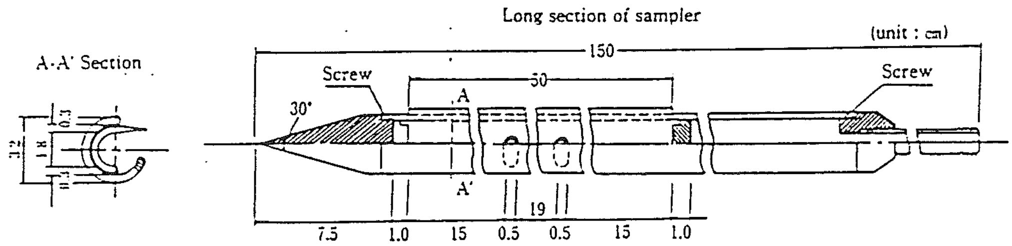


Figure 4.1 Peat sampler with cover.

Scraping the peat into the sampler, scratches the peat horizontally, but not vertically, and this method is very reliable to observe stratification. The peat sampler is effective for tests that can accommodate disturbed samples, for example tests for water content and loss on ignition, or the evaluation of the composition of soil layers. However, care is required when withdrawing the sampler from ground with the very high water contents, because water may sometimes drain from the sampler. Since the peat sampler is simply portable and can be carried and operated by one man, it can efficiently evaluate the stratification of peat to a depth of about 5 m. The depth limit for the peat sampler is about 10 m.

Continuous samplers

In the Netherlands, a 66 mm *Begemann boring unit* is employed for continuous taking of undisturbed samples. The Begemann boring unit is a well proven method to obtain high quality undisturbed (continuous) samples. To obtain long undisturbed soil samples, the friction between sample and liners must be negligible. To achieve this, the sample is enclosed in a stockinette. The stockinette is designed to envelop the sample tightly, it is flexible and watertight. The very small annulus between stockinette and liner is filled with a lubrication liquid. For lateral support, the fluid's density is adapted to fit the local soil conditions. A clamping device is activated automatically on pulling the extraction tubes, preventing soil to drop out of the sampler. The sample tubes consists of parts of 1 m long. The maximum length of samples is 25 m. Achievable sample lengths may be restricted by local soil conditions. These conditions include horizontally layered or fibrous peat (a stiff layer may be pushed forward, squeezing underlying soft layers). In this case, disturbance of the compressibility characteristics may occur before the sample has entered the sample tube.
The Begemann sampler is used in conjunction with a cone penetrometer testing unit.

Frozen-core Sampling

To obtain undisturbed peat samples with high water content, the frozen-core sampling in figure 4.2 is sometimes considered effective. Though somewhat cumbersome, this sampling method provides high quality samples for research tests [ref. 4.1]. Nevertheless, the texture of the peat may damage by the expansion of pore water and the stresses in the sample may change.

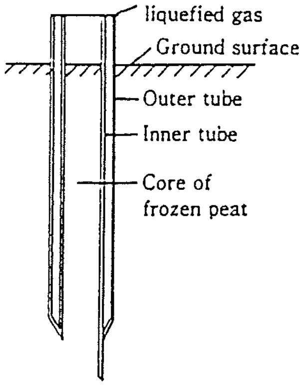


Figure 4.2 Frozen core Sampling.

Block Sampling

With thin-walled sampling it is sometimes hard to obtain samples from the surface of peat deposits containing roots of living vegetation, un-decomposed vegetable matter, and dried fibrous matter. In such cases, block sampling is recommended. There are several such methods: excavating the surroundings of a sampling site to be able to remove samples from the outside, or by excavating a pit to be able to remove samples from the pit wall. Tools for cutting samples include shovel-type plows, large knives, fine-teethed saws, and razors.

Dimensions of sample tubes

Apart from differentiation in terms of sampling, a distinction can also be made concerning the type of sample tube. Sample tubes are used in a variety of diameters and lengths.

The sampling tube most commonly used in the Netherlands consists of thin walled steel and contains no liner. The diameter of the tube is 68-70 mm externally and 66 mm internally. The length is 400 mm. A type with 100 mm or 150 mm diameter is also available. Usually, the tube is driven into the ground with the aid of the Ackerman unit or pushed into the ground with the cone penetrometer unit. The samples have to be extruded from the sampling tube again later in the laboratory. In highly compressible strata, a liner is used to prevent the quality of the sample from being affected when extruding the soil.
In the case of a Begemann boring, the material passes through the cutter shoe into a stocking on the inside of a liner. In the laboratory, liner and a stocking are then cut open.

Generally speaking, thin-walled sampling tubes in which the wall of the tube is comparatively thin relative to the diameter, can be said to cause less disruption than thick walled tubes. However, these latter can be fitted with liners.
As a further sample disruption constraint, in some cases, the lower edge of the sampling tube is equipped with a special cutting shoe. Generally, the shoe is made as sharp as possible, but the wall thickens progressively. Furthermore, internal diameter of the shoe must not be greater than internal diameter of the sleeve.

Sample size is likewise an important factor. Where diameters are relatively large, the sample volume to diameter ratio is greater than for comparatively small diameters.
Consequently, the disturbance is not so extensive as with small samples. In addition, laboratory tests on small samples of a non-uniform soil, often will produce very variable results. In view of the fact that Dutch subsoil can vary widely, this needs to be taken into account. Result reliability increases very markedly with inaccuracy increasing volume of soil examined. Block samples are largest but are difficult to handle so they are only worth considering for shallow strata.

Transport and storage

Finally, care must be exercised when transporting and storing samples. Samples must be transported upright and exposed to as few vibrations as possible. Drying out of samples must be prevented both during transport and during storage. The temperature of samples originating from the Netherlands should preferably be kept at around 10 °C. In Japan samples for long-term preservation are generally stored at a temperature of 20° C. When storing for a long period, samples usually have to be sealed off with paraffin wax.

Sample disturbance
It is virtually impossible to obtain undisturbed samples in natural condition for any type of soil, including peat. It is known that the degree of disturbance varies with the sampling technique and type of soil, and that it shows differences even in the same sampler.

The following lists factors that may cause disturbance of samples:

1. Shearing deformation due to friction between the inner face of the tube and soil while forcing the sampler into the ground.
2. Expansion due to pressure release or negative pressures at the lower end of the tube during withdrawing of the sampler.
3. Vibration, conditions of preservation, and time required for transportation.
4. Disturbances while the sample is being prepared for tests.

There are additional factors in peat:

5. Compression when forcing the sampler into the ground (while cutting fibres).
6. Tensile resistance of vegetable fibres near the sampler edge when withdrawing the sampler.
7. Draining of water during sampling in peat with high water content below the groundwater level.

To quantitatively investigate the degree of disturbances due to these factors or the sampling method, the following methods are applied:

1. Comparisons of water content, loss on ignition, and density.
2. Investigation by the stress-strain curves of unconfined compression tests.
3. Investigation by the e versus log σ curves.

Helenelund [ref. 4.2] has investigated the disturbance due to sampling and has compared various sampling techniques. He concluded that in case of fibrous peat, a significant precompression of the peat occurs with most sampling techniques. Precompression also occurs when sharp edged thin walled samplers of 10-15 cm diameter are used. This precompression can be greatly reduced by using thin walled samplers with a saw-blade edge, which is rotated slightly (zig-zag) during penetration. According to Helenelund [ref. 4.2], in fibrous peat, the sample diameter should preferably be 20 cm or more. Square sample tubes do not seem very suitable for peat. Preferably, lined and/or splitable samplers should be used and the friction between peat and sampler should be reduced by enough inside clearance. The sample lengths should be limited to a length of 50 cm. For continuous sampling, the Begemann boring unit gives good results [ref. 4.2].

4.2.2 In-situ tests

Vane Tests

The commonly used vane tester is the strain controlled lever type in figure 4.3, and its specifications are given in table 4.1. The single-tube type is used with boring or for little decomposed fibrous peat, and the double-tube type is for well-decomposed peat which causes difficulties due to friction around the rod.

Vane shear tests of peat with this apparatus yields a vane rotation angle versus vane shear resistance curve as shown in figure 4.4. In clay substrata, this curve is nearly smooth, and often has a peak value. In peaty substrata, the resistance value of each rotation angle fluctuates strongly, and the result is a jagged line, often with remarkably high and low values. This is due to resistance against the rotation of the vane caused by the fibres in the peat where some are cut or others provide an elastic response. Extraordinarily high values may be caused by large diameter peat components such as subterranean stems of reeds or sedge. Helenelund [ref. 4.3] confirmed that peat was not cut along the edge of the vane with the vane test in fibrous peat, but only deformed around the vane, and recovered to the original condition after the vane had passed.

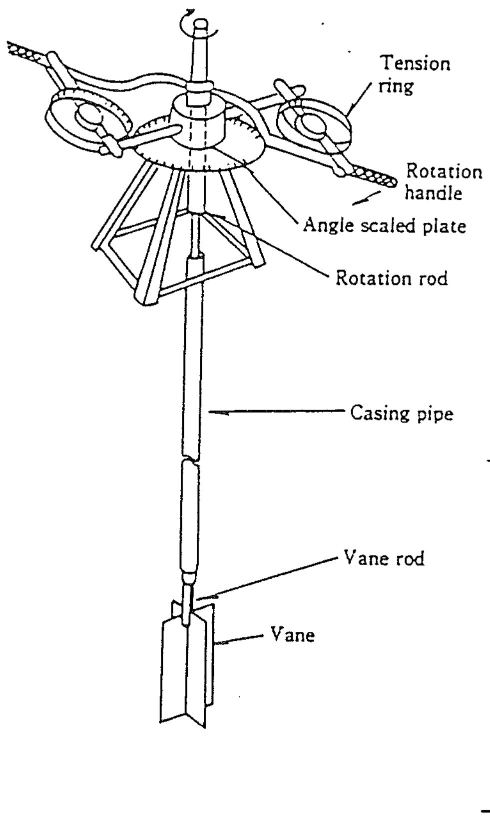


Figure 4.3 Strain controlled lever type vane.

Table-2.1 Vane specifications

Max. rotational force	2500kgf·cm
Proving ring	Capacity 50kgf
Angle scale	0~360°, min. 1°
Vane scale	$D_v = 5.5$cm, $H_v = 11$cm
Vane rod	Dia. 16mm, 1m length
Quality	Stainless steel
Rotation rod	Dia. 40.5mm, 1m length
Casing pipe	Dia. 40.5mm, 1m length

Table 4.1 Vane specifications.

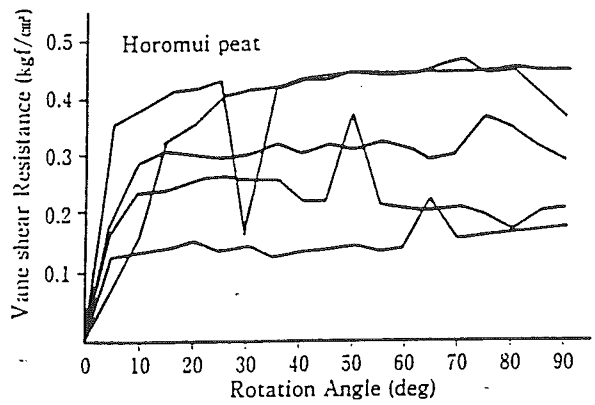


Figure 4.4 Example of vane test of peat.

He also reported that the maximum resistance value did not always give a reliable shear strength of peat. Fibrous peat is easily drained and compressed by external forces. In the vane shear test shown in figure 4.5, the front of the vane may cause compression early in the rotation, while the back of the vane may create a vacuum and drainage as indicated by the arrows. As a result the actual length of the vane shear face is shorter than the theoretical one, and the vane shear strength determined by the test is greater than the actual shear strength of peat. The degree of compression and drainage, and the actual length of the vane shear face are unknown, and it is difficult to estimate the actual from the obtained values. Landva [ref. 4.4] investigated the vane shear test in peaty substrata, and confirmed that voids were generated at the back of the vane. He also reported that failure of bog moss occurred at 20-40° rotational angles, and that failure occurred about 10 mm outside the edge of the vane. In addition, it was found that standard vanes caused no apparent failure face in fibrous peat, the measured values indicate the strain resistance of fibres, and do not show the real shear resistance.

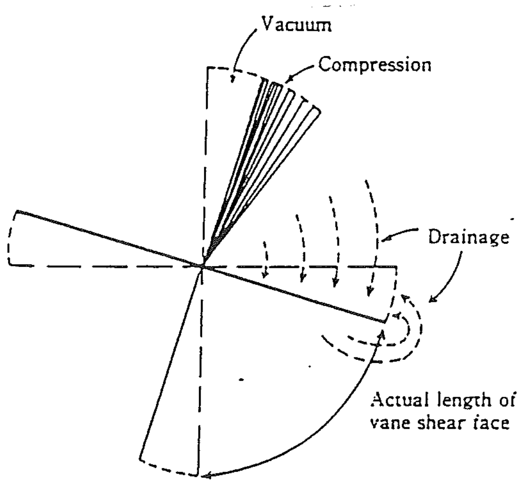


Figure 4.5 Phenomena at vane tests on fibrous peat.

Consequently, there is no way to simply change a maximum resistance value into the real shear strength of fibrous peat, and a representative value must be determined from an evaluation of the relationship between the rotational angle and the resistance values after excluding extraordinary values. It must be stressed that the vane shear strength obtained from the vane test can only be regarded as an engineering index.

Vane dimensions and vane shear strength

It has been reported that different vane sizes have little effect on the results of vane tests in clay, while this is not the case in peat [ref. 4.5 and 4.6]. Figure 4.6 shows reported values of vane shear strengths for different vane widths in peat deposits. The figure shows that a 5.5 cm-wide vane gives an average shear strength and a scattering of values that is two or three times those of a 10 cm-wide vane. That is, larger vanes give smaller measured vane shear strengths and less scatter of the values. The implication is that larger vane dimensions are preferable. Northwood et al [ref. 4.7] and Radforth [ref. 4.8], used similar research results to propose the use of the 10 cm (4 in)-wide vanes at H_v/D_v =2 (height/diameter). However, by experience large power is necessary to rotate a 10 cm-wide vane in fibrous peat, and there is much difficulty in handling the vane. Therefore, in most cases, the 5.5 cm vane has been used.

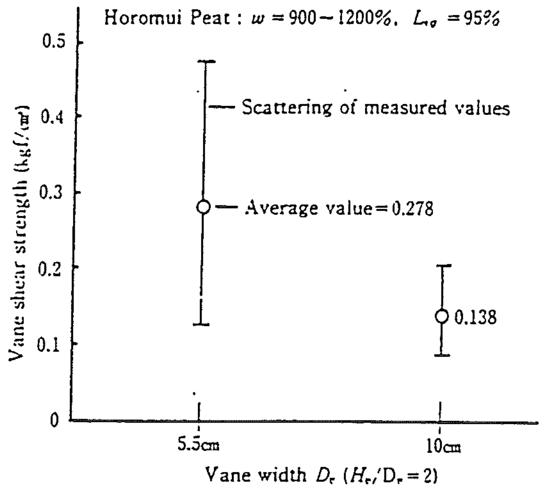


Figure 4.6 Vane dimensions and vane shear strength.

Vane rotation speed and vane shear strength

In vane shear tests of clay, the vane shear strength increases with rotational speed. A rotational speed of 1.4 deg/s results in about twice the shear strength values obtained at 0.017 deg/s [ref. 4.9]. Skempton's experiments show that vane shear strengths at rotational speeds of 0.15-0.5 deg's correspond to the shear strength obtained in unconfined compression tests. In addition, basic experiments in Sweden have indicated that there are no changes in shear strength values below a rotational speed of 0.1 deg/s, and that an apparent increase in resistance occurs above this value [ref. 4.10]. Nowadays, a rotational speed of 0.1 deg/s is widely used as a standard.

In peat, with high water content and high compressibility, vane rotation at low speeds may cause drainage and compression of the peat in the vicinity of the vane, and result in an increase in the apparent strength of the peat. It may also be expected that vane rotation at a high speed will result in an increase in strength of peat due to the tensile resistance of the peat, or due to cohesive and frictional resistance between peat fibres. These changes will vary with the organic content and the state of decomposition of the peat. Figure 4.7 shows changes in vane shear strength at various speeds of rotation for different investigations. Practically it is very difficult for a lever type tester to maintain a rotational speed below 0.1 deg/s, and comparatively smooth rotation is only obtained above 0.5 deg/s.

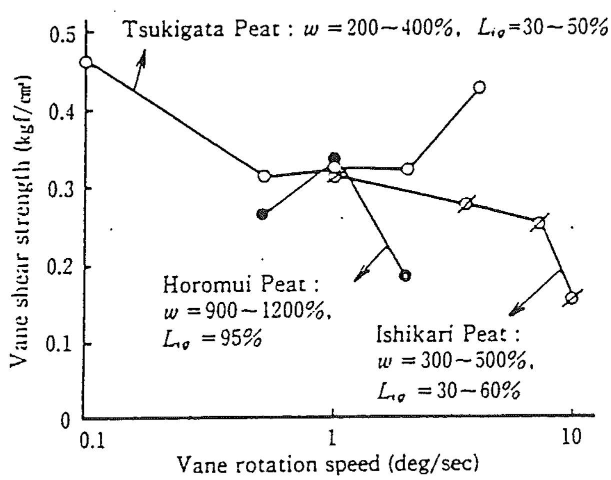


Figure 4.7 Vane rotation speed and vane shear strength.

From the above considerations, the vane shear strength of peat may be taken as an engineering index, and when converted by some factor it gives an indication of the design shear strength. In view of operational constraints and work efficiency it may be necessary to adopt a maximum rotational speed.

Static Cone Penetration Tests

Deep continuous testing with the vane shear test is expensive and requires much labour. Contrarily to the vane test, the Cone Penetration Test (CPT) is relatively fast and the obtained cone bearing capacity, q_c, gives a good indication of the strength of substrata.

The static cone penetrometers commonly used in Hokkaido is the WP-20 type cone, which may be used for very soft substrata of $q_c < 5$ kgf/cm^2. Measured values obtained from each test should be compared because of different specifications as indicated in the table 4.2. According to the research findings with the static cone penetration test in clay [ref. 4.11], the relationship between the cone bearing capacity by the WP-20 type cone penetration test, q_{cw}, and that by the Dutch cone penetration test, q_{cd}, is $q_{cd}=1.53\ q_{cw}$. The interrelationship between these two cone bearing capacities measured in peat deposits is shown in figure 4.8, and expressed by:

$$q_{cd} = 1.12\ q_{cw} \tag{4.1}$$

Type	WP-20	Dutch cone
rod form	double tube	double tube
cone form	with taper	taper & sleeve
cone section area	20 cm^2	10 cm^2
cone angle	30°	60°
symbol	q_{cw}	q_{cd}
gauge	proving ring	burden gauge

Table 4.2 Specifications of static cone penetrometers

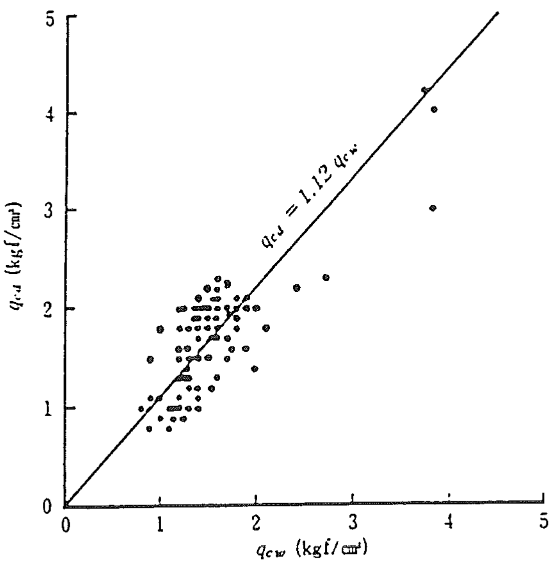


Figure 4.8 Relation between q_{cw} and q_{cd}.

Although the estimation of strength parameters from CPT's is a subject for discussion, for the dutch electrical friction cone, good correlations are available to establish soil types. The cone penetration tests is a fast and reliable method to determine the local thickness of peatlayers. In figure 4.9 the soil type classification based on the local friction (measured with an electrical dutch friction cone) is given.

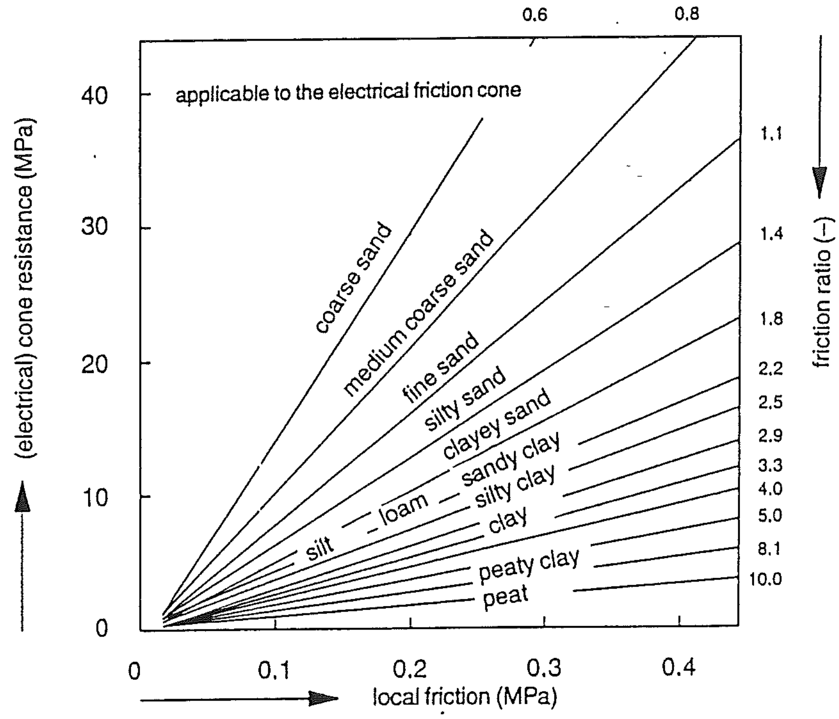


Figure 4.9 Soil type classification using the dutch electrical friction cone.

4.3 Parameters from laboratory tests

The geotechnical characteristics of peat show a remarkable heterogeneity in the depth direction as well as in the plane. It is difficult to accurately and completely determine properties of such soils through soil investigations. Because of the heterogeneity, a visual description of the soil samples shall always be made. Such a description may give an indication of anisotropy and may state how representative the test results are for the in-situ behaviour of the peatlayer. Sometimes incorrect test results are obtained, and it is necessary to investigate all obtained values thoroughly to determine their validity. The water and organic content of peat are much higher than ordinary soil, and the specific gravity, density, and void ratio correlate well with water content and loss on ignition. Research has found that the natural water content correlates well with loss on ignition, dry unit weight, and the natural void ratio. Accordingly, when the water content in peaty substrata is measured accurately, the validity of results of soil tests may be evaluated from these data. Results deviating from the range of established data, may then be excluded or revised before a final determination of soil constants.

The values for the stability or settlement calculations is the average or central value of test results of soil surveys. The average value is the arithmetic mean of all test results, and the central value is that of soil tests in the representative part of the substrata considering the other test results. It is preferable to adopt representative values through technical evaluations rather than just routinely determine average values. Especially where relatively lower values are used to determine the peat strength from vane and cone penetration tests [ref. 4.12].

Furthermore, one must be cautious with the translation of laboratory test results into in-situ behaviour, as (especially for the strength and deformation characteristics) the tests results largely depend on the stress and strain conditions used in the tests.

4.3.1 Unit weight and water content

After a ring of known internal volume is pushed into the sample and the weight of soil determined, the unit weight can be calculated. After drying, the water content is also known whence the unit weight of the dry soil can be ascertained.
Water content is determined at various temperatures between 60 and 105°C. Skempton and Petley [ref 4.13] prove that at 105°C during 24h, loss of organic matter is limited, while at lower temperatures, small amounts of free water are retained.
The number of samples undergoing the unit weight plus water content determination must be large enough to provide a representative picture of the magnitude and possible fluctuations is unit weight of the subsoil even when the soil structure is non-homogeneous.
For some applications, it can be relevant to observe shrinkage anisotropy after drying.

4.3.2 Loss on ignition

Organic content can be determined in several ways, but for high organic contents, it is sufficiently accurate to simply equate organic content to loss on ignition. Temperature and duration of firing in the loss-on-ignition test differ, but Delft Geotechnics has good experience with 550°C during 5h. Betelev [ref. 4.14] states that the loss on ignition method is acceptable in soils containing up to 15% organic matter, and advocates 550±50°C during 3h. Correlations are most simply cast directly in terms of ignition loss. Ash content is the complement of loss on ignition, and is preferred by some. Betelev [ref. 4.14] presents a method for determining the organic content of soils and rocks based on dry combustion at 500°C in an oxygen or air stream. The quantity of CO_2 production is measured by a gas analyzer. The method is economical and can also be applied to eg. oil-polluted soils.

4.3.3 Strength

An accurate depiction of strength as a function of deformation is supplied by the triaxial test. The triaxial test is available in a number of versions which differ in terms of apparatus and procedure. Essentially, the test is strain-controlled in that cell stress remains constant during the tests. The following distinction between. four types of triaxial tests is generally adopted:

- the unconfined compression test
- the unconsolidated undrained test (UU)
- the consolidated undrained test (CU)
- the consolidated drained test (CD).

The determination of unconfined compressive strength is considered unreliable for peat with a water content over 300 - 400 %.

In the simple shear tests, the sample is merely subjected to pure angular rotation. The test gives a good idea of strength as a function of angular rotation.

The direct shear test produces relatively inaccurate data insofar as strength parameters are concerned. In principle, in this test, the shearing plane is predetermined, while boundary phenomena also occur through wall friction. In consequence, measurement generally provides too high values. Water pressures cannot be recorded so that where the samples are cohesive, the test is only suitable for obtaining an indication of undrained shear strength via quick deformation.

The laboratory vane test, the manual vane test or torvane test and the pocket penetrometer test, all carried out on peat samples are not very reliable.

Triaxial Compression Tests
In NEN 5117 [ref. 4.15] the triaxial tests are described in detail in terms of equipment, implementation and results processing.

Results of the undrained triaxial compression test (CU Test) of peat give the angle of shear resistance due to the effective stress as $\varphi'=48°$ [ref. 4.16] or $\varphi'=34°$ [ref. 4.17], very high values. This has led to the suggestion that the strength of peat basically depends on the friction and follows the principle of effective stress. The large φ' results from an high pore water pressure during shear and the reinforcing effect of organic fibres. The stress - strain curve does not produce a peak, therefore the result varies strongly with the stress chosen as failure stress. In another report, the triaxial compression test gives the slope of rupture envelope on the Mohr's effective principal stress circle as 5°. Accordingly, some view the strength of peat as almost exclusively expressed by the cohesion [ref. 4.18].

Rate of Strength Increase
The rate of strength increase ($\Delta c/\Delta p$) by compression is 0.5-1 or more [ref. 4.19]. As shown in figure 4.10, according to the research on National Highway 12 at Toyohoro, Hokkaido, the relation between the stress increase Δp and the increase in vane shear resistance Δc in foundati-

ons due to embankments in nearly $\Delta c/\Delta p = 1$.
Tsushima [ref. 4.20] reports that a greater loss on ignition results in a greater angle of shear resistance and increase in undrained strength; and that these two correlate closely.

As to engineering characteristics of peat, the increment in strength due to depth and one-sided change in characteristics often found in common alluvial clays, is seldom observed or very small. This is a result of peat deposits mainly consisting of dead vegetable matter with low specific gravity, e.g. no increase of σ_v' with depth.

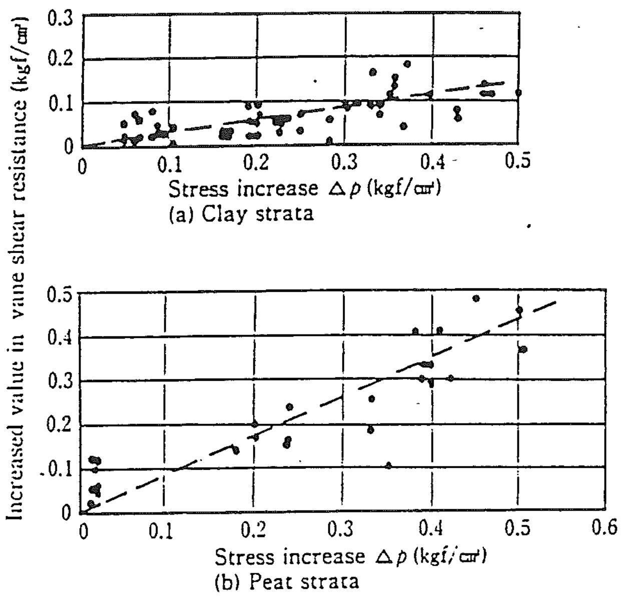


Figure 4.10 Δp versus Δc.

Simple shear-apparatus

The simple shear-apparatus is regarded as being extremely valuable as a practical instrument. The mean deformation state along a shear plane tallies more or less with the simple shear state and the stress rotation occurring there is also established in the unit.

Generally, the stress state in the active part of the shear plane (under the crown, beneath the foundation, as appropriate) is most closely approached by the triaxial compression test. Likewise, the stress state in the passive section of the shear plane (outside the toe or foundation) is best simulated in the triaxial extension test. The more or less horizontal sections of a slip surface come very close to stress state in the simple shear test. If a slip surface can thus be divided into three identical parts, a mean τ- γ- relationship can be constructed from three separate tests based on the idea that the sliding soil body will move as a whole. Ladd in [ref. 4.21 and 4.22] termed this the 'strain compatibility technique'. This technique is demonstrated in figure 4.11 for a soft marine clay ('offshore Maine'). It now appears that both the stress/strain curve and the shear strength averaged by the above procedure concur with the result of the simple shear test.

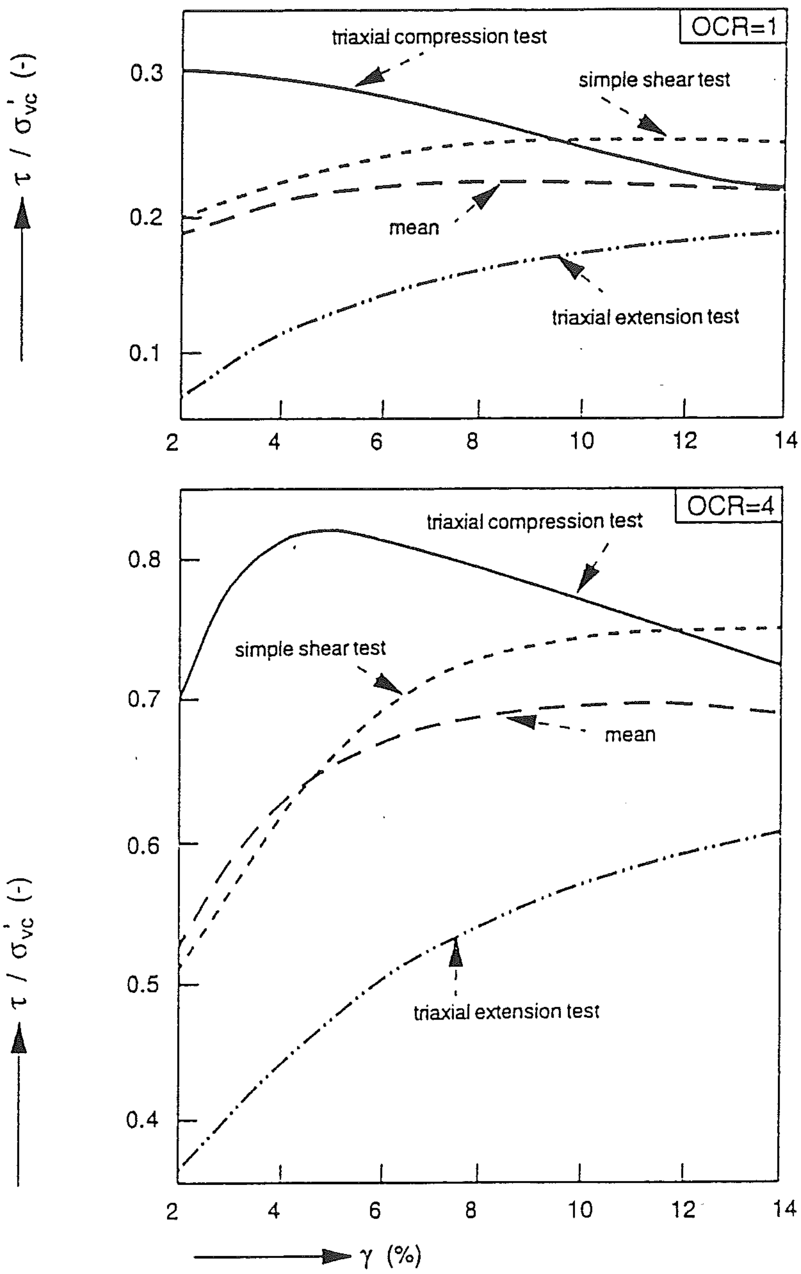


Figure 4.11 The Ladd 'strain compatibility technique'.

This test result can thus usually be regarded as immediately representative of the entire slip surface; consequently no separate triaxial tests are needed.

As an alternative to the simple shear test, the old-fashioned direct shear test can also be performed. The direct shear test produces relatively inaccurate data insofar as strength parameters are concerned. The direct shear test is only suitable for obtaining an indication of undrained shear strength via quick deformation.

4.3.4 Deformation

The compression test determines most accurately the one-dimensional deformation parameters needed to calculate the magnitude and rate of vertical deformation or settlement.
For practical reasons a sample of limited size certainly in relation to the thickness of the compressible layer, tends to be employed.

The most accurate and most thorough representation of stress/strain behaviour is recorded by the triaxial test and simple shear test. The elasticity moduli deduced from these are essential for certain types of calculations. The tests are however comparatively expensive while even in this case the samples used are comparatively small relative to the thickness of the highly compressible layer.

4.3.5 Other relevant laboratory tests

Beside the above mentioned tests, valuable information can be obtain from visual examination. A visual description should include:

- estimation of fibre content,
- preferred direction of fibres.

Further, for some applications, the von Post humification test may give valuable information.

4.4 Parameters from field tests

Besides all kind of test, performed in the field and/or in the laboratory, a visual description of all available samples shall be made. Especially in case of peat, a description of the soil texture is highly valuable. As already mentioned above, such a description is required to classify the peat and to obtain information about anisotropy in strength, deformation characteristics and permeability.

4.4.1 Unit weight and water content

After a core, drill pipe or a sample tube of known internal volume, has been inserted, unit weight can be calculated after weighing. Subsequently, before working out unit weight of the dry soil, the water content has to be determined, usually by drying the soil in the laboratory.

4.4.2 Strength

It is considered that peat texture changes in the depth direction and deposit horizon, and there may be differences in physical properties in the two directions. Reported research [ref. 4.23, 4.24 and 4.1] give an anisotropy of 1.3 - 2.2 in strength. When investigating the stability of the structures, care must be taken to account for this problem in designs and plans.

The resistance values obtained with vane tests, include the vertical shear stress, τ_v, and horizontal shear stress, τ_h, and separation of the two has been attempted (Manai and Kitago [ref. 4.24]). It has been confirmed that the vane shear strength τ with a vane of M (= H_v/D_v) = 2 for peat of about τ_v/τ_h = 2 is determined by the following formula with α = 1/3, and is nearly equal to the vertical vane shear strength τ_v [ref. 4.25]

$$\tau = \frac{2M}{\pi D \upsilon \ (M+\alpha)} \tag{4.2}$$

Figure 4.12 shows the relationship between the cone bearing capacity, q_{cw}, with a WP-20 type cone and the vane shear strength, τ_v (research number = 422) [ref. 4.26]. The regression line (solid) correlates comparatively well with the correlation coefficient γ= 0.64. A presentation of τ_v/q_{cw}, assuming q_{cw}= 0 at τ_v= 0, for each date and simply averaged is described by the dotted line in the figure. The broken line indicates the relation obtained in similar investigations in the past [ref. 4.27 and 4.28]. The mechanism of cone penetration and vane shear in peat are not clear, but possibly the organic content affects the resistance. With the vane shear test, increasing organic content gradually increases tensile strength - the breaking resistance of the vegetable fibres, while this is less pronounced in the cone penetration test. Consequently, the relationship between q_c and τ_v may vary with increasing organic content. A similar tendency is seen with increasing water content, closely related to the organic content.

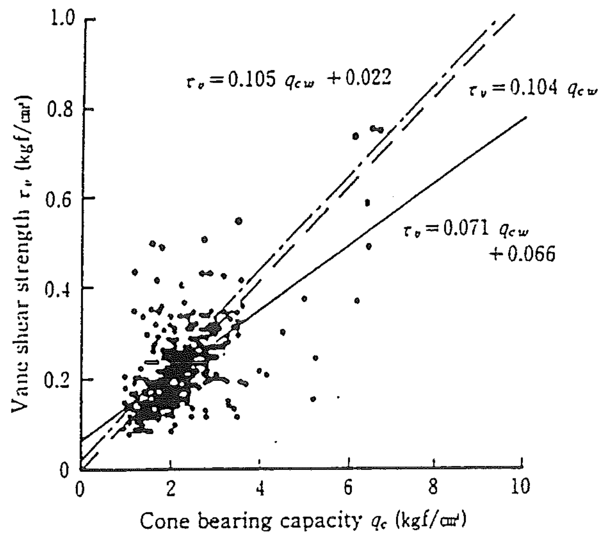


Figure 4.12 Relation between q_{cw} and τ_v.

The τ_v/q_{cw} value varies with the water content, but is widely dispersed in the range 0.05-0.25. Setting different values of τ_v/q_{cw} for each water content makes design complicated, and averages of these values may be used. The relationship between τ_v and q_{cw} (see figure 4.13) can be written as

$$\tau_v = (0.104 \text{ or } 0.114)\, q_{cw} = (1/9)\, q_{cw} \tag{4.3}$$

Using expression (5.3) above, the relationship between the cone bearing capacity, q_{cd}, and the vane shear strength, τ_v, is

$$\tau_v = 1/9\ (1/1.12\ q_{cd}) = (1/10)\, q_{cd} \tag{4.4}$$

Oikawa and Sato [ref. 4.29] report the relationship between the vane shear strength at a rotation speed of 0.1 deg/s with a 5 x 10 cm vane and cone bearing capacity using a 30° point angle cone with 20 cm^2 cross section as $\tau_v = 0.12\, q_c$, almost similar to the value above.

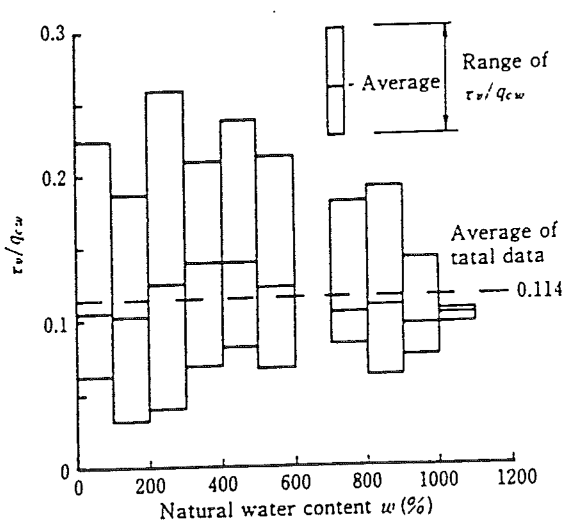


Figure 4.13 τ_v/q_{cw} classified by water content.

Regarding the in-situ vane test, it must be stressed that in peat, vane tests provides non-reliable figures for determining undrained strength. The shear strength obtained from the vane test can only be regarded as an engineering index.

4.4.3 Deformation

Deformation parameters are hard to measure in-situ because measurement entails simulation of the spread of loads applied and development of excess pore water pressure. For these reasons, there are hardly any in-situ methods which yield direct deformation parameters of peaty soils.

The horizontal stress/strain behaviour of highly compressible strata can be measured in-situ with the aid of pressure meter tests. Based on various types of tests (such as the Ménard pressure meter, self-boring pressure meter and the Camkometer test) and assisted by semi-empirical formulae, a horizontal modulus of elasticity and sometimes also the horizontal in-situ stress can be deducted. Compared to other types of pressure meter, the self-boring pressure meter provides results, which are better not only in terms of quality but also in that more parameters, such as strength and consolidation parameters, for example, can be deduced correlatively. However, execution of the test appears to be time-consuming and impractical. The dilatometer test is closely related to the pressure meter test. However, as the dilatometer is based on very small strains, this apparatus is considered inaccurate for application in peat.

4.5 Safety aspects

An earth structure located on peaty subsoil, must meet specific safety and usability standards throughout its service life. Safety and serviceability are generally established by reference to stability and deformation calculations. This usually entails resort to computer models in which the physical behaviour of ground is often highly schematised. Furthermore, the geometry of soil strata is schematized based on soil investigation results; specific values are often ascribed to the geotechnical properties of each layer. The natural heterogeneity of the subsoil plus the random nature of the soil investigation introduces a fairly large uncertainty into the calculations.

Thus, in the traditional deterministic safety approach conservative assumptions are applied to the parameters in the calculation whilst an overall safety factor is applied to the result of the calculation.

Over the last decades, the tendency has been to make increasing use of more advanced, so-called probabilistic methods which adopt a more subtle approach to safety. The 'Level 1' approach involves relying on so-called partial factors for the calculation model employed and for the material parameters.

4.5.1 General safety philosophy

Limit states are states beyond which the structure no longer satisfies the design performance requirements. In general, a distinction is made between:

Limit state 1 - Ultimate limit state: Failure mechanisms at which collapse occurs
Limit state 2 - Serviceability limit state: Failure mechanisms in which a specific criterion for serviceability is exceeded without collapse occurring.

In geotechnics, the serviceability limit state tends to be related to attainment of a particular deformation criterion such as the occurrence of a particular maximum settlement or a particular maximum differential settlement.

Generally, soil structures fail when the strength R of the structure is less than the load S acting on the structure, hence where R-S $<$ 0. The expression R-S is often termed the reliability function Z

$$Z = R - S \tag{4.5}$$

The reliability function Z has the property of $Z < 0$ corresponding to failure, $Z > 0$ to non-failure, $Z = 0$ being the limit state. The ration between the strength R and the load S is also well defined as the overall safety factor y_0

$$y_o = \frac{R}{S} \tag{4.6}$$

However, practice shows that neither the strength properties nor the loads can be calculated accurately beforehand. This can be ascribed to the following non-exhaustive sources of uncertainty:

- the natural heterogeneity of the soil strata
- the statistical uncertainty connected with soil properties because only a limited number of measurements (borings, cone penetration tests, laboratory tests) is available
- the uncertainty surrounding the geotechnical computer model used owing to schematisation of actual physical behaviour
- the defects caused by human mishandling, such as negligent execution or wrong geotechnical design and/or calculations.

4.5.2 Deterministic methods

In conventional deterministic methods, conservative assumptions are made in regard to the magnitude of the variables for strength (x_1, ... x_n) and for load (y_1, ... y_m). Thereupon an overall safety factor (y_0) creates a standard margin between the calculated strength (R) and the load (S). In formula form, the requirement is

$$\frac{R(x_1, \dots x_n}{S(y_1, \dots y_m)} \geq y_0 \tag{4.7}$$

4.5.3 Probabilistic Level I methods

Although neither the strength properties nor the loads are known precisely, in practice it is definitely possible to estimate these so-called basis variables, by assigning a certain probability distribution to them. The most common is the normal distribution characterised by the mean value μ and the standard deviation σ; sometimes a log-normal distribution is more suitable. The estimation of the probability distribution becomes more accurate when more data are available.

The fact that the procedure involves using estimated values for strength R and load S implies that the overall safety factor obtained itself is also little more than an estimate. It should therefore be remembered that the true safety factor can be smaller than the estimated value and possibly even less than 1.0. In practice, this means that there is always the chance, however small, of the structure falling, so the following definition applies

$$P_f\ (\ Z<0\ )\ >0 \tag{4.8}$$

in which

P_f = probability of failure

With the more advanced, probabilistic methods, structural safety is expressed in terms of a probability of failure, P_f, e.g. 1:2000, 1:10000 and so on.

With probabilistic methods, allowance is made for the potential deviations in the value of various parameters. The parameters are interpreted as stochastic variables identified by the following non-exhaustive list of parameters:

μ = the expected value or (mathematical) expectation; in a normal distribution the arithmetical mean of a sample test is an estimate of the expected value,

σ = the standard deviation,

V = the coefficient of variation, this being the quotient of σ and μ.

For a general description of statistical concepts, the reader is referred to literature sources.

In practise the Level I method is used for calculation purposed. The Level I method uses a series of partial safety factors by which representative values of the variables are divided or, where this would be more disadvantageous, multiplied so that safe, so-called design values are obtained. The structure must then meet the requirement

$$R_d\ (\frac{x_{kl}}{\gamma_{rl}}\ ,\ \ldots\ \frac{x_{kn}}{\gamma_{rn}}) \geq S_d\ (\gamma_{kl}\cdot\gamma_{sl}\ ,\ \ldots\ \gamma_{km}\cdot\gamma_{sm}) \tag{4.9}$$

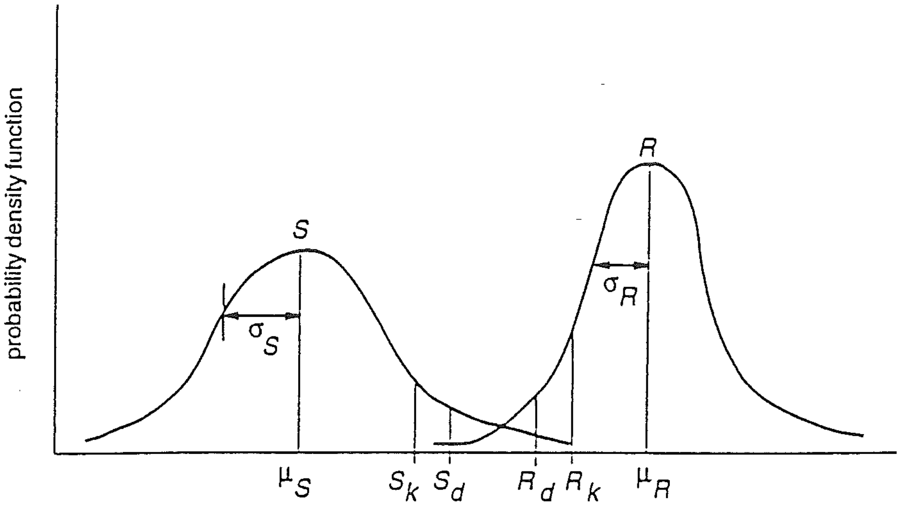
The principle of Level I calculations is explained in figure 4.14. One representative value of the basis variable for strength and one for load are denoted R_k and S_k respectively in the figure; the

clay	clean	soft	14	0.5	0.013	0.452	17.5	0	25
		medium	17	1	0.006	0.121	17.5	10	50
		stiff	19 or 20	2	0.004	0.056 or 0.042	17.5 or 25	25 or 30	100 or 200
	slightly sandy	soft	15	0.7	0.009	0.253	22.5	0	40
		medium	18	1.5	0.005	0.079	22.5	10	80
		stiff	20 or 21	2.5	0.003	0.042 or 0.014	22.5 or 27.5	25 or 30	120 or 170
	very sandy	-	18 or 20	1	0.004	0.063 or 0.025	27.5 or 32.5	0 or 2	0 or 10
	organic	soft	13	0.2	0.015	0.550	15	0 or 2	10
		medium	15 or 16	0.5	0.012	0.250 or 0.140	15	0 or 2	25 or 30
peat	not preloaded	soft	10 or 12	0.1 or 0.2	0.023	2.530 or 0.600	15	2 or 5	10 or 20
	moderately preloaded	medium	12 or 13	0.2	0.016	0.600 or 0.300	15	5 or 10	20 or 30
variation coefficient				-	0.25		0.10	0.20	

The table presents the low representative value for the soil properties of the layer average of the concerning soil type. If the most unfavourable situation in practice is created by application of the high representative value of the layer average, a value based on consistency/relative density of the soil type concerned must be chosen from the next row (i.e. from denser, respectively stiffer material) and in case of dense, respectively stiff material the value after 'or' must be chosen. The same applies for the properties C_α and C_{sw} on the understanding that in those cases a lower value leads to lower settlement.

[1]) C_α-values are valid for a range of pressure increase of at most 100%.

Table 4.3 Relation between soil type and soil parameters according to NEN 6740 [ref. 4.30] design values are denoted R_d and S_d respectively. The representative value is either a characteristic value or a nominal value. The characteristic value adopted is usually the value which in 5% of cases is under or overshot; in normal distributions with expectation μ and standard deviation σ, the characteristic values are (μ - 1.64.σ) and (μ + 1.64.σ). In practice, characteristic values must be determined using a sample test. A nominal value is a value laid down in a standard, manual or specification. Table 4.3 gives an example of nominal values according to NEN 6740 [ref. 4.30].



$$S_d = S_k \cdot \gamma_S$$

$$R_d = \frac{R_k}{\gamma_R}$$

criterion: $S_d \leq R_d$

Figure 4.14 Principle of the Level I probabilistic method of calculation.

System of partial factors

Description of this method can be found in the CUR-book C162 [ref. 4.31].

New Dutch Standards in the building construction field [ref. 4.32], adopting the example of the Euro Code and ISO Standards, include confirmatory methods which rely on partial factors, hence in line with the Level I probabilistic method.

Determination of the characteristic value

According to NEN 6740 [ref. 4.30] the characteristic value X_k is a 5% fractile. Hence, the 5% lower limit is used for parameters affecting stability positively or limiting deformations (such as, for example, cohesion c' and the angle of internal friction φ'); the 5% upper limit for parameters influencing stability or deformations negatively (such as, for example, the retaining height or load). In the case of a normally distributed stochastic parameter of known expected value (μ) and variation coefficient (V), the 5% lower limit is given by

$$X_k = \mu \ . \ [1 \ - 1.64 \ V] \tag{4.10}$$

the 5% upper limit by

$$X_k = \mu \ . \ [1 \ + 1.64 \ V] \tag{4.11}$$

In reality, within a layer of soil a spatial variation occurs (in directions *x*-, *y*- and *z*-) for a particular property which could be expressed in terms of an auto-correlation function.

If the range in values adopted must be derived from test results, this means that in case of a soil investigation of normal scope (i.e. 2-3 samples per soil layer, for example) a very large margin between mean and characteristic values should be allowed for.

The values of V to be adopted can depend on the type of soil or formation. For instance, as might be imagined, peat has much higher scatter than clay. Basically, however, the uncertainty in the mean value of a layer tends to decrease with increasing number of test results. If the number of test findings is infinite, the entire soil volume has been sampled and the mean strength is known precisely. Consequently, the variation coefficient requires correction for the number of samples (n) used in the design cross-section.

n	3	4	5	6	8	10	∞
t	2.92	2.35	2.13	2.02	1.89	1.83	1.64

For V - 0.10 and n = 3 or 4 this means: $X_k = 0.83 \ . \ \mu$ or $0.88 \ . \ \mu$ respectively.
For $V = 0.10$ and n = 5 or 6 this means: $X_k = 0.90 \ . \ \mu$ or $0.92 \ . \ \mu$ respectively.
For a very large sample size $n (n \rightarrow \infty) \ X_k = \mu$.

Table 4.4 Test factor as a function of sample size

$$\bar{V} = V \ . \ \sqrt{(\frac{1}{n})} \tag{4.12}$$

In this way , in NEN 6740 [ref. 4.30], the characteristic mean value in a layer for a soil parameter is determined from

$$X_k = \mu \ . \ [1 \ - V \ . \ t \ . \ \sqrt{\frac{1}{n}} \ ] \tag{4.13}$$

In this formula, t is the test factor for a reliability level of 95%. The value of t is shown in table 4.4. The above relationship is only considered applicable to three or more test results, i.e. $n \geq 3$. For less than three test results, a safe value must be assumed for the characteristic value [ref. 4.30]. In case of only one test result, according to [ref. 4.30], a safe value can be taken from the most adverse of the following two values:

- the most adverse representative value per table 4.3
- the test result.

In the case of two test results to [ref. 4.30], a safe value can be adopted by taking the most adverse representative value from table 4.3 on condition that at least one of the test results here is identical or higher (lower respectively). If such is not the case, then the most adverse test result must be employed. When using formula (9), the true variation in a sample test essentially is completely ignored. Furthermore, in small samples (in a statistical sense) the t-value to be used is very large and the characteristic value therefore small. Under the authority of TAW [ref. 4.33 and 4.34] a Bayesian method is developed in which both the a priori assumed variation coefficient and the variation coefficient as derived from sampling are involved. The test results are thus used to test the a priori assumption. Where the sample size (n = 2-3) is very small, confidence in the a priori assumption in regard to the variation prevails; the larger the number of tests, the more decisive the true variation from the sample test will be for estimation of the characteristic value.

Building stage

Apart from evaluating the macro-stability of a soil structure in the fully consolidated end situation, the macro-stability whilst building is in progress also needs to be taken into account. This is essential for two reasons: firstly in order to investigate whether stability during the building phase is completely guaranteed and secondly in order to establish whether the design is in fact technically feasible and at what rate the various activities can be carried out.

As regard the building phase, a higher failure probability may be acceptable temporarily because better control is then exercised, experts are present and because the phase is short-lived. The reliability index β chosen should then, for example, be one category lower.

Consequently, the building phase usually needs to be considered as a separate structure for which a separate safety level must be selected. This safety level may or may not deviate from the safety level for the end phase. In TAW reports [ref. 4.33 and 4.34] the safety approach for specific, temporary load situations during the building phase for water defences, fills and excavations is qualitatively assessed. This approach is explicitly based on the 'Bishop' slip circle stability method.

For the building phase, in certain circumstances other partial factors can be used. The β value can be determined for this purpose from an analysis in which also cost aspects are taken into account. Consequently the required partial factors can be calculated.

> > > > > > to be added (if available) a list of partial and overall safety factors for various constructions on peat < < < < < <

4.5.4 Variability of peat properties

Both laterally and vertically, peats show large variability. Figure 4.15 shows zones identified within a single block sample of fibrous Wisconsin peat exhibiting clearly different creep parameters [ref. 4.35]. Borings taken at 2 m centres in a peat area in Delft yielded strong variations in loss on ignition N as shown in figure 4.16 (X: location of borehole, Y: depth below surface). This extreme variability on the decimeter scale probably also exists at larger and smaller scales as well. Predictions of peat behaviour will have to account for this variability.

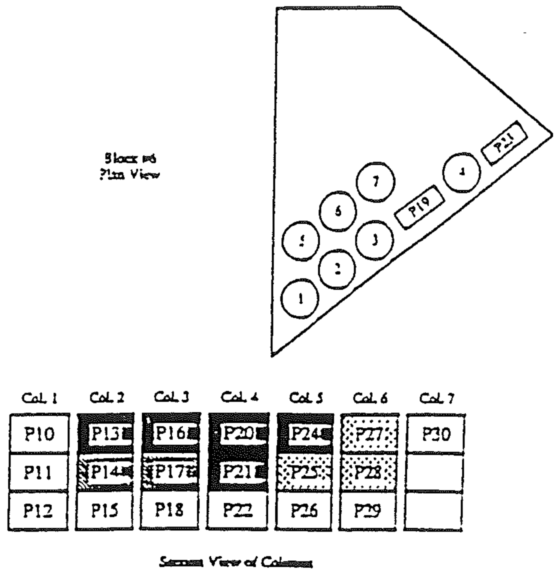


Figure 4.15 Zones of microstructure within a peat block [ref. 4.35].

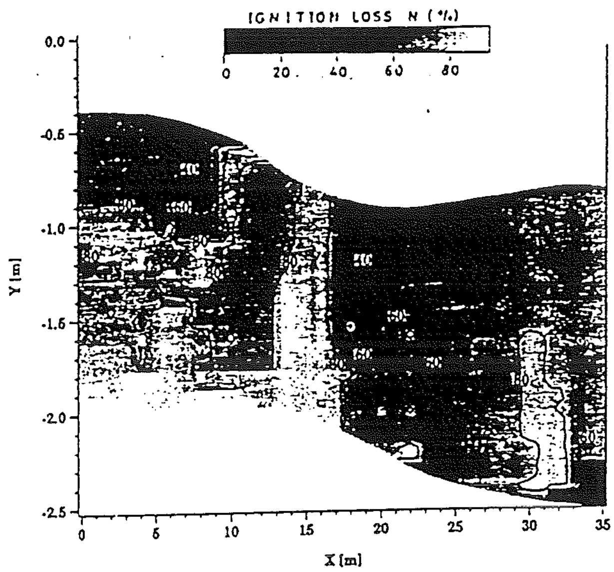


Figure 4.16 Contours of loss on ignition in a peat deposit near Delft (Sikder, 1994).

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5 Design methods

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The concepts of behaviour of inorganic clays have often been applied to peats and organic soils, with varying degrees of success. In the main it is possible to apply traditional soil mechanics theory to these soils, but there are important abnormalities that need special consideration. In this section special attention is given to constitutive models and methods of design, which use for peats and organic soils is regarded.
The described methods are adapted for the application to peat and organic soils. In cases where no specific calculation method for peat exists, the limitations of the available and commonly employed calculation method are discussed.

In section 5.1 methods are given to calculate one-dimensional compression. Section 5.2 deals with the stability of structures built on peat. As an alternative for, or in addition to the traditional calculation methods, section 5.3 discusses finite elements calculations. Sections 5.4 descibes the influence of new construction activities on existing civil constructions in peat area's.

5.1 Compression of peats and organic soils

Methods of design and calculation for one-dimensional compression generally follow those used for inorganic clays, but it is necessary to take account of some abnormalities in the behaviour of peats and organic clays. The main abnormalities concern the anisotropy and non-linearity of permeability, the strong stiffening of the material at large compression, and the strong secondary (or secular) compression. Some remarks will be made on each of these issues in this section.

Simple formulae for the final settlement of peat have been developed by many researchers. These formulae will be discussed in section 5.1.1 and may be used for a first and rough indication of the settlements. This first approximation may help to chose which method (e.g. with common or natural strain, see section 5.1.2 and 5.1.4 respectively) has to be choosen for a more detailed settlement analysis.

The conventional method of one-dimensional settlement calculation is described in section 5.1.2. This method uses parameters such as the (primary) compression index C_c to calculate the primary compression and the vertical coefficient of consolidation c_v to calculate the time to end of primary (EOP) compression. The secondary compression index C_α is used to calculate the secondary compression, which is assumed to proceed linearly on the logarithm of time scale after end of primary.

The permeability affects the consolidation process during primary compression. In section 5.1.3 special attention is given to the permeability of peat which is highly non-linear to the normal stress due to large void ratio changes during settlement. Furthermore the permeability usually is anisotropic in peat.

In case of large strains the soil behaviour seems to follow a natural strain law rather than the common (Cauchy) strain law. Therefore section 5.1.4 introduces the natural strain in the primary compression which uses the natural (primary) compression index *b*.
Much debat has been on the question whether creep, i.e. secondary settlement, starts directly after loading or at EOP. The creep behaviour model based on C_α, as described in section 5.1.2, assumes EOP. The so-called abc-model describes primary and secondary strain integrally, which is theoretically more sound. The model introduces the direct compression index *a* and the natural secondary compression index *c*. The abc-model is based on natural strain and relates strain rate $\dot{\varepsilon}$ to vertical effective stress σ'_v and to voids ratio *e* (e.g. density). Some correlations for the parameters in the abc-model will be given.

Finally section 5.1.5 deals with the topics: unloading/pre-loading, submerge of fill, sustained height and the so-called tertiary compression.

5.1.1 Simple formulae for estimation of final settlement

Simple formulae for final settlement may render a first and rough indication of the settlements. The use of simple formulae is recommended, because it helps to make the right choice of a more detailed settlement method (e.g. small or large strain).
Many simple formulae have been developed by different researchers to enable designers to easily estimate final settlement on the basis of a few simple parameters. Among these are Al-Khafaji and Andersland [ref. 5.1], Flaate [ref. 5.2], Carlsten [ref. 5.3], Noto [ref. 5.4], Meyer and Dereczenik [ref. 5.5] and Den Haan and El Amir [ref. 5.6]. These formulae require knowledge of initial and final stress, and parameters such as organic content, natural water content, and relative volumes of water and organic matter.
Den Haan and El Amir's [ref. 5.6] method relates natural water content normalized by ignition loss (*w/N*) to final stress and will be explained later on in this subsection. A comparative study of all the available formulae for final settlement in peats has not yet been made. However, comparisons of the Den Haan and El Amir and the Al-Khafaji & Andersland methods are executed [ref. 5.6] and results show only small differences. If this would be true for all methods, the most simple could be chosen for use in international handbooks and guidelines for construction on peats and organic soils.

Fokkens/Den Haan method
For very highly compressible peat subsoil, the correlation method Fokkens/Den Haan is used in the Netherlands. This correlation method estimates the final settlement of peats in terms of a

relationship between water content, ignition loss (which represents the organic material content) and effective vertical stress. The method is applicable to normally consolidated virgin peat. With overconsolidated peat, the method is only applicable if the load is such that the stress in the peat after loading exceeds the preconsolidation pressure.

The Fokkens/Den Haan method is based on a combination of an ageing law from pedology and the logarithmic compression law from soil mechanics [ref. 5.7]. The ageing law establishes a correlation between organic material content and water content for a specific degree of ageing. The organic material content is determined by the loss on ignition of a peat sample measured after five hours combustion at 550°C. The compression law describes the relation between the extent of compression (which is related to the change in water content) and the effective stress. Fokkens [ref. 5.7] found that a specific degree of ageing corresponded to a certain effective stress or capillary stress above the ground water level. From soil measurements he discovered a unique correlation between water content, organic material content (loss on ignition) and effective stress. Den Haan later modified the Fokkens correlation on the basis of additional laboratory research [ref. 5.8] (the Fokkens method was based on a unit weight of peat of the order of 10 kN/m^2; this value is not always correct and has been rectified by Den Haan).

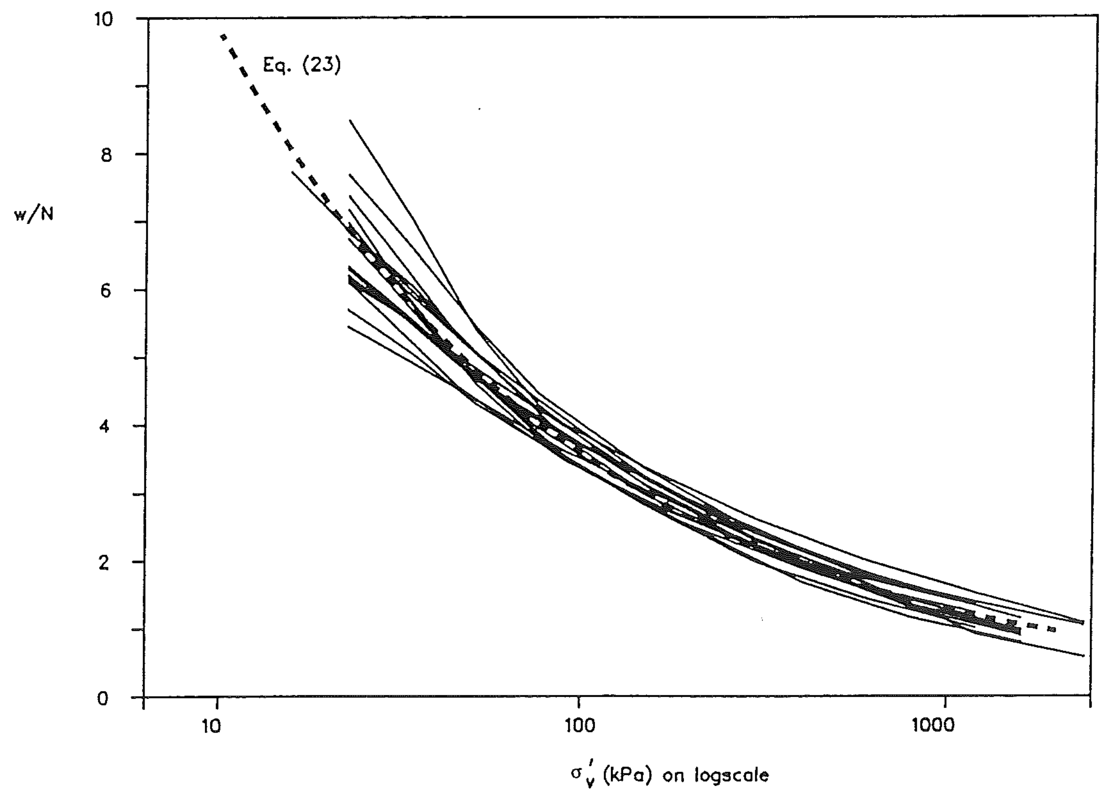


Figure 5.1 Polder Zegveld peats, *w*/*N* - log σ'_v for 10^4 days loading.

Figure 5.1 shows that a fairly unique curve is obtained for about 25 oedometer tests performed on Polder Zegveld peat, the fit curve is given by

$$\frac{w}{N} = 26.7 \cdot \left[\frac{\sigma'_v}{\sigma'_u}\right]^{-0.437} \tag{5.1}$$

in which

w = water content after load (%)

N = loss on ignition after 5 hours at 550°C (%)

σ'_v = vertical effective stress in normally consolidated peat (kPa)

σ'_u = unit effective stress (σ'_u = 1.0 kPa)

As a result of a load increase, the effective stress increases. With of the resulting reduction in water content, the final peat layer compression can be determined.

Den Haan describes the final compression as a function of loss on ignition and water content before and after load for loads in the virgin compression range as

$$\frac{\Delta h_e}{h} = \frac{w_0 - w}{w_0 + 37.1 + 0.362 \cdot N} \tag{5.2}$$

in which

Δh_e = final compression of peat layer (m)

h = thickness of peat layer (m)

w_0 = initial water content (%)

where w can be estimated from equation 5.1. The curves in figure 5.1 and equations (5.1) and (5.2) are valid for a loading duration of 10^4 days, i.e. final settlement.

5.1.2 Compression models based on common (Cauchy) strain

The total settlement is assumed to be the sum of the so-called primary and secondary settlement. The time period of the primary settlement ends when the extra water pressure due to a load increase has vanisched. The secondary settlements represents the creep period, which is practically completed after an arbitrarily chosen period of 10^4 days (approximately 30 years). The sum of the primary and secondary settlement after 10^4 days is defined as the final settlement.

primary settlement

From consolidation tests, where no lateral deformation occurs, Terzaghi concluded that there was no linear relationship between compression and load. This means that Hooke's law is invalid [ref. 5.9]. However, Terzaghi found a direct relation between the compression and the logarithm of the load. This relation, which is widely used internationally (see the 1-day line in figure 5.2) is

$$e_0 - e = C_c \log\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right) \tag{5.3}$$

in which

e_0 = initial void ratio (-),
e = void ratio (-),
C_c = primary compression index (-),
σ_0 = initial effective stress (kPa),
$\Delta\sigma$ = increase of effective stress (kPa).

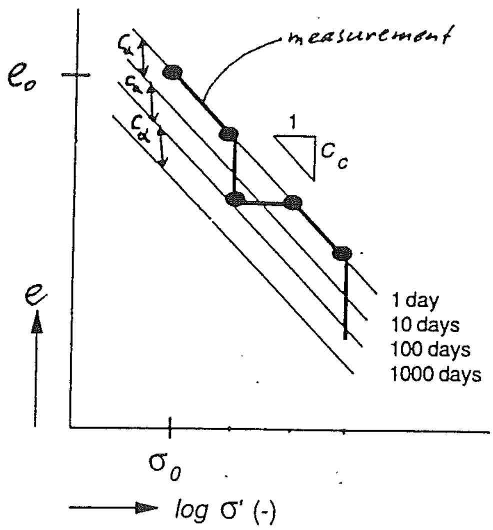


Figure 5.2 Relationship for primary and secondary compression index.

Formula (5.3) gives the primary compression due to an increase of the load on the soil sample. With dry soil, this compression or settlement occurs almost immediately. For peat or organic soils that are saturated with water, settlement is achieved only after the entire load has been transferred through the grain structure, i.e. when all excess pore water pressure has vanished by expulsion of water under simultaneous compression of the sample. The phenomenon of the gradual decrease in the pore water pressure with resulting soil compression is known as consolidation and is strongly dependent on the permeability of the soil. In section 5.1.3 some special aspects of the permeability of peat are discussed. The consolidation period (or hydrodynamic period) is usually represented by the consolidation coefficient c_v, which can be determined from the oedometer test results by the so-called Casagrande method [ref. 5.10]. The time needed for 100% consolidation is also called the "time to end of primary".

Virgin soil, i.e. normal consolidated soil, is more compressible than soil under unloading or reloading. For unloading, the primary compression index C_c is replaced by the swell index C_s for unloading and by the recompression index C_r for reloading. Under a greater load than the previous load, the original primary compression index again applies The value of the earlier load is known as the preconsolidation pressure p_g.

Formula (5.3) is based on so-called Cauchy strain (i.e. strain is defined by the quotient of the settlement and the initial layer height) and applies to relatively small deformations e.g. maximum strain in the order of 20%. If deformations are large, the formula is only valid if the value of the (primary) compression index C_c is determined for the loading trajectory which occurs in practice. If such is not the case, then a solution can be found by updating the layer thickness h and the void ratio e_0 after a specific deformation, but a better solution is to turn to other strain definitions as for instance the so-called natural strain, which will be further elaborated in section 5.1.4.

secondary settlement
In addition to primary settlement, there is secondary settlement. Secondary settlement is related to the creep behaviour and in principle it proceeds indefinitely. In practice, the increase in settlement after a specific period is negligible, and it is usually assumed that the settlement is practically completed after 10^4 days (approximately 30 years).

Consolidation tests [ref. 5.11] show that, after the consolidation phase or hydrodynamic period has ended, there is usually a direct correlation between compression and the logarithm of time, see figure 5.3, which is in formula

$$e_0 - e = C_\alpha \log\left(\frac{\Delta t}{\Delta t_d}\right) \tag{5.4}$$

in which

e_0 = initial void ratio (-),
e = void ratio (-),
C_α = secondary compression index
Δt = duration of load (days),
Δt_d = duration of one day (= 1.0 day).

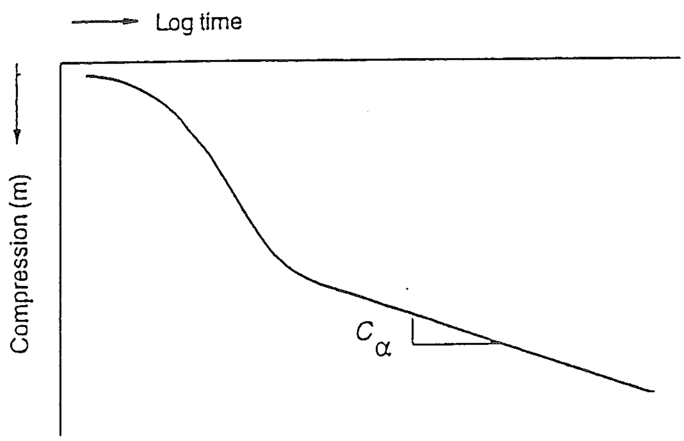


Figure 5.3 Compression versus logarithm time.

Various concepts have been developed concerning the start of the time duration Δt in Formula (5.4). Some researchers assume that Δt begins as soon as the load is applied; others believe that the end of the consolidation period, i.e. the end of primary EOP, is equal to the start of Δt. As a result, there is some confusion about the magnitude of the secondary settlement contribution to the final settlement. In figure 5.2 it is assumed that secondary compression starts after 1 day. Secondary strain is severe in peats and cannot be as easily ignored as is often done when dealing with more firm soils. The secondary compression according to formula (5.4) is independent of the size of the load. Each small load increment will reset the secondary settlement time scale. In other words each small load step will generate substantial secondary settlement. Esspecially in peats, secondary settlements are large and results in case of phased loading may become unrealistic. Mesri solves this problem by assuming for load steps that are smaller than the preconsolidation pressure the primary settlements are equal to the end of primary settlement values without a consolidation process. Furthermore for these small load steps the secondary settlements time scale will not be resetted, but follows a new C_α that coresponds with the new vertical effective stress.
Note that due to creep deformations, or ageing, a so-called quasi preconsolidation pressure builds up. As a result of this, on a later increase of stress, the soil initially behaves as if it was overconsolidated, even though stresses exceed their previous maximum loads.
In section 5.1.4 the so-called abc-method of Den Haan is introduced, which method describes the primary and secondary compression integrally, which gives a theoretically more sound solution for the creep and consolidation problem as well as large strains. Den Haan defines lines with equal creep strain rate, which are similar to the lines of 1, 10, 100 and 1000 days in figure 5.2 that also show a constant creep strain rate de/dt or $\dot{\varepsilon}^C$.

Correlations with the C_α/C_c Concept
Mesri *et al.* [ref. 5.12 and 5.13] has stated that for most clays C_α can be correlated with C_c, see figure 5.4.

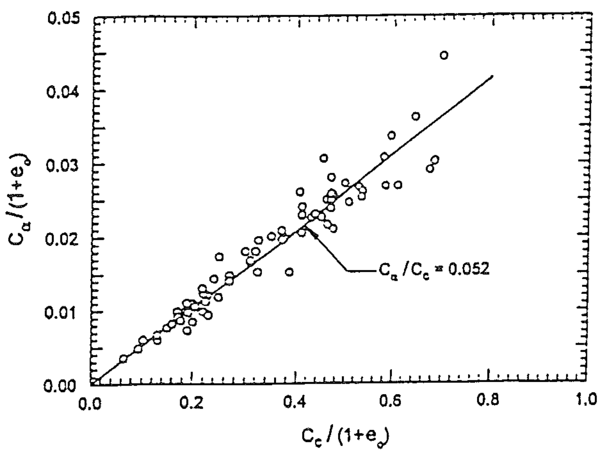


Figure 5.4 Relationship between secondary compression index and (primary) compression index for Middleton peat.

Mesri *et al.* [ref. 5.12 and 5.13] found that for a majority of inorganic soft clays:

$$\frac{C_\alpha}{C_c} = 0.04 \pm 0.01 \tag{5.5}$$

and for highly organic plastic clays:

$$\frac{C_\alpha}{C_c} = 0.05 \pm 0.01 \tag{5.6}$$

and for peat deposits:

$$\frac{C_\alpha}{C_c} = 0.06 \pm 0.01 \tag{5.7}$$

in which he assumes that C_c is determined from end of primary (EOP) values. The value of C_α is obtained from the linear segment of the e - log$t$ curve immediately beyond the transition from primary to secondary compression (thus the slope directly after EOP). This means that in equation (5.4) the value of Δt_d is not 1 day but is equal to the time to end of primary t_p. Mesri *et al.* further shows that the ratio of C_c and C_α is constant for each time point, so

$$\left(\frac{C_\alpha}{C_c}\right)_{t_p} = \left(\frac{C_\alpha}{C_c}\right)_{1\mathrm{day}} \tag{5.8}$$

Thus the value of C_α in equation (5.4), which corresponds to $\Delta t_d = 1$day is related to the values of C_c in equation (5.3) that also correspond to 1 day.

Mesri *et al.* [ref. 5.12] further shows that whether the behaviour is normally consolidated or over-consolidated, the ratio between C_α and C_c is more or less the same.
If this ratio for the creep behaviour is the case for all peats and organic clays then less time consuming creep tests may suffice.

The total settlement for multiple layers
The total settlement of multiple layers is calculated by the sum of the settlements of individual layers. For each individual layer, constants are used for soil parameters which apply to the layer centre. Consequently, the compression is calculated for each layer taking into account any preliminary loading or preconsolidation pressure. The initial effective stress in the centre of the layer concerned is determined from the mass of the soil column above and the pore water pressure present prior to load application. When determining the pressure increase caused by the load in the layer concerned, lateral stress distribution may be taken into account. The total settlement for all layers at a certain time is calculated by adding together the primary and secondary compressions of the different soil layers at the time concerned.

A question that remains is how to handle the development of the consolidation in multiple layers, because EOP (i.e. start of creep) is defined as the time after which 100% consolidation has been reached.

5.1.3 Permeability

Permeability in peat is strongly anisotropic, with generally much higher horizontal permeability than the vertical permeability. Furthermore the permeability of peat is highly non-linear, which may be shown to be a result mainly of the large decrease in voids space during compression. Figure 5.5 shows an example of measurement of Mesri *et al.* [ref. 5.12]. It shows that the horizontal permeability is larger than the vertical permeability. Furthermore it clearly illustrates the dependency of the vertical permeability versus the void ratio.

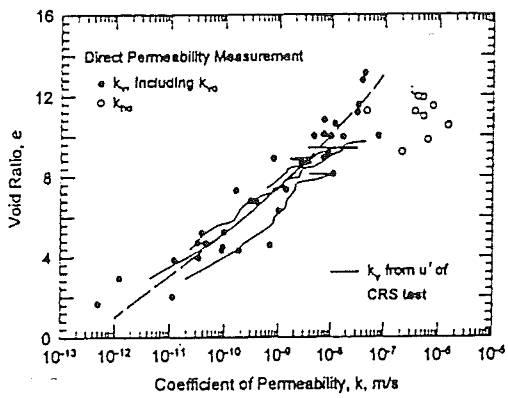


Figure 5.5 Permeability of peat in vertical and horizontal directions versus void ratio.

The relationship between voids ratio e and logarithm of permeability often used for clays is

$$e_o - e = C_k \log\left(\frac{k}{k_o}\right) \tag{5.9}$$

in which:

- e_0 = initial voids ratio (-)
- e = voids ratio (-)
- C_k = coefficient of permeability decrease (-)
- k = coefficient of permeability (m/s)
- k_0 = initial coefficient of permeability (m/s)

In an oedometer test the value of e_0 is known. The value of k that corresponds with e is found from Taylor's $\sqrt{t}$ method (for e values in the virgin compression range). Then the two unknown variables C_k and k_0 follow from curve fitting equation (5.9) with the k and e values.

In clays, C_k may approximately be estimated from $C_k \approx 0.5\, e_0$. There are indications that in peats C_k values are usually higher than in clays. Mesri *et al.* [ref. 5.12] compared values of C_k for peats and clay and silt deposists, see figure 5.6, and found values of $C_k \approx 0.25\, e_o$ for peats. Usually, the initial coefficient of permeability k_o in the vertical direction of peats is typically 100 to 1000 times larger than the initial permeability of soft clay and silt deposits

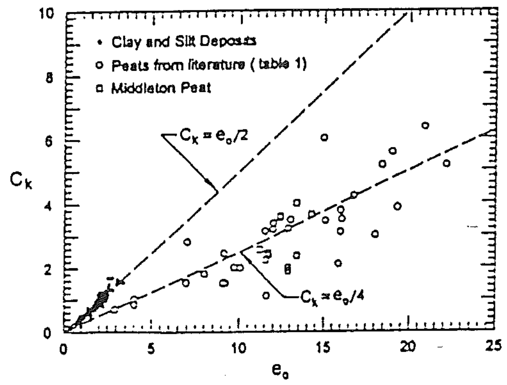


Figure 5.6 Values of C_k for peats as compared to those of soft clay and silt deposits.

Scaling from sample to field
Values of permeability measured on small oedometer samples cannot be trusted to accurately forecast the development of primary compression in the field. Peat is extremely variable, mass structural features are not represented by small samples, the anisotropy of permeability is not accounted for, and gas contents may not be equal in the sample and in the field. The initial permeability can be quite high in peat, and is enhanced by the generally much higher horizontal than vertical permeability.

There has been much debate about the validity of the scaling law for time during primary compression

$$\frac{t_f}{t_s} = \left[\frac{H_f}{H_s}\right]^i \qquad (5.10)$$

in which s denotes sample and f denotes field and

t = time (sec)
H = drainage distance (m)
i = constant, which is 2 in the conventional Terzaghi theory

Comparison of field and laboratory measurements has indicated that i may be smaller than 2 for peat, a value of 1.5 often being mentioned. The irregularity could be due to any of the features mentioned above, while Hobbs suggests that the vertical permeability may be much higher in the field than measured in the oedometer. The large decrease in permeability with increasing compression (non-linear permeability) is further not generally accounted for when considering equation (5.10).

There exists a widespread pessimism about the possibilities of accurately forecasting the development of settlement and pore pressure dissipation during the primary phase in peats and organic soils. Modern settlement models however have no difficulty in accounting for anisotropic and non-linear permeability and it should be possible to combine field and laboratory tests to evaluate anisotropy and non-linearity of permeability. Clearly though, much remains to be improved. In this respect recent (to be published in 1997) Canadian data of Lefebvre shows to be promissing.

5.1.4 Compression models based on natural strain, abc-model

The e versus $\log\sigma'_v$ graph is not linear but convex for large strains in peats and soft organic clays. This makes the (primary) compression index C_c (see section 5.1.2) inefficient, because it is useful only over a predetermined, limited stress range. By simply replacing strain by so-called natural strain, the virgin relationship is again rendered linear in many cases. Then the natural compression index b, which is the slope of the virgin curve in natural strain versus $\ln \sigma'_v$, describes the complete virgin range adequately.
Natural strain also can be applied to the secondary settlement, but the question: when secondary strain starts and how to deal with phased loading is solved by introducing creep starin rate.

Natural Strain
Natural strain is obtained by the integration of increments of deformation relative to the momentary dimension. Thus

$$\varepsilon^H = \int_h^{h_0} \frac{dh}{h} = \ln\left(\frac{h_0}{h}\right) \tag{5.11}$$

in which

ε^H = natural strain (-)
h = thickness of layer (m)
h_0 = initial thickness of layer (m)

and the superscript H commemorates Hencky as the first to apply this measure of strain.

Common engineering strain (or Cauchy strain) differs from natural strain in that deformation is related to the initial layer thickness h_0 instead of the actual layer thickness h. Cauchy strain is denoted by ε^C and is related to natural strain through

$$\varepsilon^H = -\ln\left(1 - \varepsilon^C\right) \tag{5.12}$$

Note that

$$\varepsilon^C = \frac{h_0 - h}{h_0} = \frac{e_0 - e}{e_0 + 1} \tag{5.13}$$

Substitution of equation (5.13) into (5.12) renders

$$\varepsilon^H = \ln\left(\frac{e_0 + 1}{e + 1}\right) \tag{5.14}$$

primary settlement based on natural strain

Lefebvre *et al.* [ref. 5.14] made use of natural strain to obtain linear relationships between tangent modulus and effective stress for James Bay peats. They showed that this leads to a linear relationship between natural strain and logarithm of effective stress. Earlier, natural strain had been introduced in soil mechanics by Lundgren [ref. 5.15], Juárez-Badillo [ref. 5.16] and Butterfield [ref. 5.17]. Noto [ref 5.4] shows a similar kind of scaling with straight lines on a log$e$ versus log σ'_v graph.

In case of natural strain equation (5.3) leads to

$$\ln\left(\frac{e_0 + 1}{e + 1}\right) = b \ln\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right) \tag{5.15}$$

in which

e_o = initial void ratio (-)
e = void ratio (-)
b = natural compression index (-)
σ_0 = initial effective stress (kPa)
$\Delta\sigma$ = increase of effective stress (kPa)

Den Haan [ref. 5.18, 5.19 and 5.20] shows that a very wide range of soil types are adequately formulated by the natural strain of equation (5.15) rather than the common strain of equation (5.3). Esspecially for large strains, i.e. larger than 20%, the formulation of equation (5.15) is in favour of equation (5.3).

A consequence of using natural strain is that ultimately, at infinite stress, volume becomes vanishingly small. This is expected in a hypothetical sense, because not only the voids, but also the solids compress at high stress. It is likely that solids volume decreases before all voids are compressed, and voids ratio would probably also approach zero only at infinite stress.

In case of small strains

$$\ln\left(\frac{e_0 + 1}{e + 1}\right) = -\ln\left(\frac{e + 1}{e_0 + 1}\right) = -\ln\left(1 + \frac{e - e_0}{e_0 + 1}\right) \approx -\frac{e - e_0}{e_0 + 1} \tag{5.16}$$

and substituting equation (5.16) into equation (5.15) renders equation (5.3) for

$$\frac{C_c}{e_0 + 1} = 2.3\ b \tag{5.17}$$

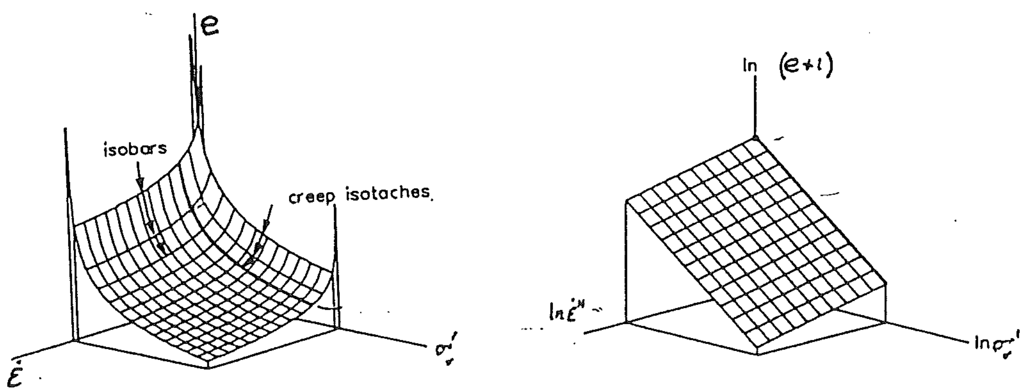
Thus equation (5.17) shows the relation between b and C_c for small strains. Note that in equation (5.17) the factor 2.3 is due to the difference between the functions log and ln.

Secondary settlement based on natural strain and creep strain rate

Secondary strain is severe in peats and cannot be as easily ignored as is often done when dealing with more firm soils. Secondary compression occurring after the hydrodynamic primary period, is conventionally described by a linear voids ratio e - log$t$ relationship with slope C_α, see section 5.1.2. When natural strains are used equation (5.4) could be replaced by

$$\ln\left(\frac{e_0 + 1}{e + 1}\right) = c^*\ \ln\left(\frac{\Delta t}{\Delta t_d}\right) \tag{5.18}$$

in the same way as for C_c and b of equations (5.3) and (5.15) respectively. However, it is easy to show that both secondary settlement relations of equations (5.3) or (5.18) do not reflect a fundamental property of the soil, because each small load increment will reset the secondary settlement time scale. In other words each small load step will generate full secondary settlement. Esspecially in peats, secondary settlements are substantial and results may be unrealistic. In the following it will be shown that this ploblem can be solved by chosing creep strain rate as the creep parameter in stead of time.



(a) (b)

Figure 5.7 a unique relationship between void ratio, effective stress and creep strain rate.

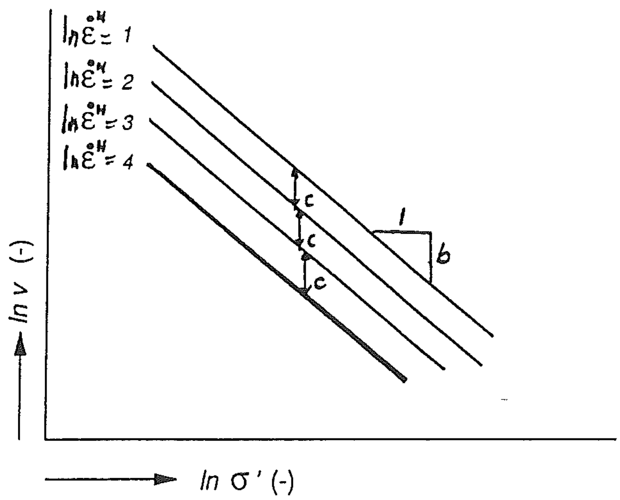


Figure 5.8 Projection of constant creep lines to the lnν - lnσ'_v -plane.

Most recent philosophy with regard to creep behaviour in peat and organic clay assumes a unique relationship between void ratio, effective stress and creep strain rate. This means that in the e - σ'_v - $\dot{\varepsilon}$ -space, see figure 5.7a, a unique curved plane exists. Note that $\dot{\varepsilon}$ is de/dt.
Den Haan shows that for the transformed axis of ln$(e+1)$, lnσ'_v and ln$\dot{\varepsilon}^H$ this plane may be represented by a flat plane, see figure 5.7a. A projection of the constant creep rate lines in figure 5.7b to the lnν - lnσ'_v -plane ($\nu=e+1$) is shown in figue 5.6. The angle of the constant creep rate lines is defined by b, which is in fact the (primary) natural compression index as given in equation (5.15). The distance between the constant creep rate lines is defined by the natural secondary compression index c

$$\ln\left(\frac{e_0+1}{e+1}\right) = c\,\ln\left(\frac{\dot{\varepsilon}_0^H}{\dot{\varepsilon}^H}\right) \qquad (5.19)$$

in which

e_0	=	initial void ratio (-)
e	=	void ratio (-)
c	=	natural secondary compression index (-)
$\dot{\varepsilon}_0^H$	=	initial creep strain rate (day^{-1})
$\dot{\varepsilon}^H$	=	creep strain rate (day^{-1})

Note that $\dot{\varepsilon}_0^H$ corresponds with e_0, i.e. the initial creep strain rate corresponds with the initial void ratio. The thick line in figure 5.9 represents an oedometer test result with two load increments. The constant creep rate lines in this figure are found by curve fitting the two creep parts of the measurement (time and void ratio) to equidistant lines. Then the initial creep strain rate may be found by extrapolating these equal creep strain rate lines to the initial void ratio. With the parameters b and c and the initial point (e_0, σ_0 and $\dot{\varepsilon}_0^H$) all other combinations of void ratio, stress and creep strain rate are defined.

Note that Mesri, see figure 5.2 and section 5.1.2, also uses lines of constant creep strain rate, but he assumes that creep always starts from the 1-day line. Den Haan shows that creep not necessarily has to start from this 1-day line, which is esspecially the case for small load increments. Den Haan further assumes that consolidation and creep occur at the same moment, while Mesri assumes both behaviours occur consecutive.

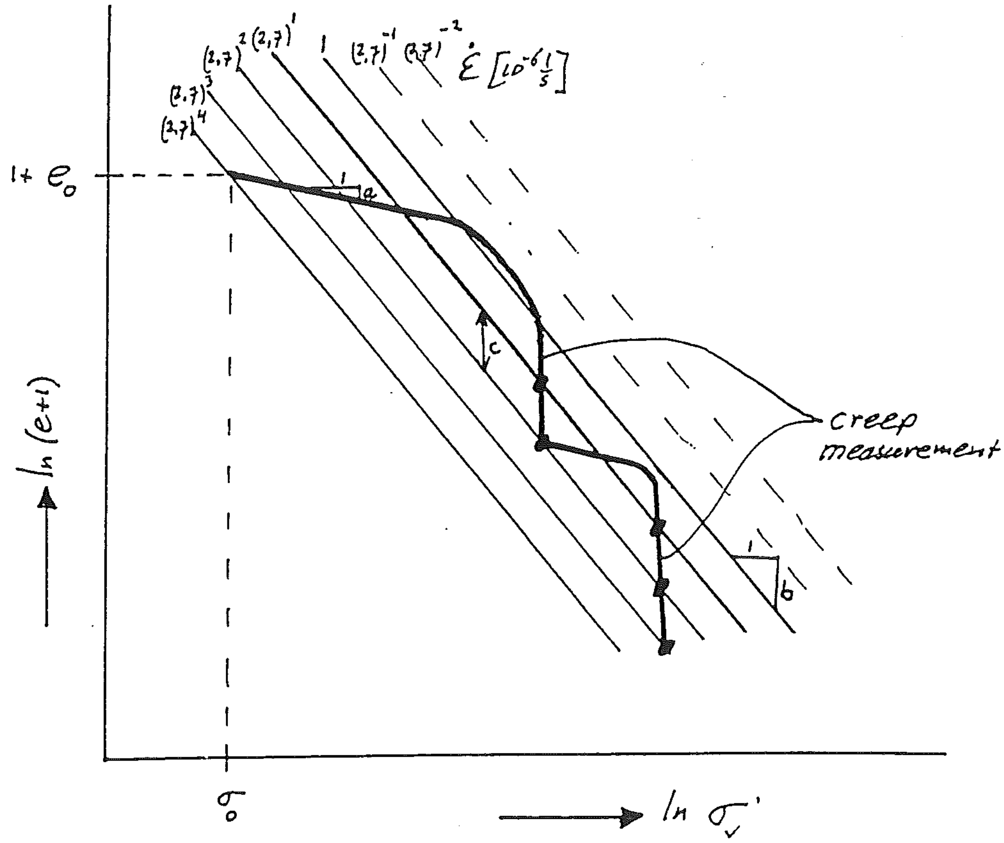


Figure 5.9 Fictive oedometer test with two load increments plotted on the constant creep lines in the lnν - lnσ'_v -plane.

A natural creep strain rate $\dot{\varepsilon}^H=10^{-7}$ s^{-1} often corresponds roughly to the rate at the end of 24h. Much compression data has been reported in terms of 24h, and because 24h curves are essentially parallel to the constant creep rate lines (see figure 5.9), b can be determined fairly accurately from standard 24h curves.
In stead of the initial point correponding to e_0 also another reference point may be chosen. Den Haan [ref. 5.20] suggests to chose a reference point that corresponds with $\sigma_0=1$ kPa and a creep strain rate corresponding to 24h, i.e. 1-day. He denoted the void ratio of this reference point in terms of specific volume and named it ν_1.

The system of parallel lines determine creep rate $\dot{\varepsilon}^H$ as a function of natural strain and effective

stress, see figure 5.9. They form a background pattern to the development of stresses and strains which is assumed to be valid in the primary (or hydrodynamic) period as well as the secondary period.
In the primary period, strain rate then is assumed to consist of the sum of creep rate and rate due to compression induced by an increase in effective stress. The latter is the so-called direct settlement. For the direct settlement, see figure 5.9, it is assumed

$$\ln\left(\frac{e_0 + 1}{e + 1}\right) = a \ \ln\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right) \tag{5.20}$$

in which

a = natural direct compression index (-)

Total settlement

The total settlement during primary and secondary compression is

$$\ln\left(\frac{e_0 + 1}{e + 1}\right) = a \ \ln\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right) + c \ \ln\left(\frac{\dot{\varepsilon}_0^H}{\dot{\varepsilon}^H}\right) \tag{5.21}$$

Note that in the total settlement of equation (5.21) the parameter b is defining the shift in figure 5.9 between $\dot{\varepsilon}_0^H$ and $\dot{\varepsilon}^H$. Equation (5.21) is dubbed the so-called abc-model. Some parameters for a Middleton peat sample are for example: a=0.05, b=0.384, c=0.043, $\dot{\varepsilon}_0^H$=-1.408 and σ_0=1 kPa.

Consolidation and phased loading

Combining the chosen stress - strain - strain rate equations with Darcy flow and the continuity equation, it is possible to calculate combined primary and secondary deformations following surface loading of peat deposits. The permeability of the soil must be known to calculate Darcy flow, and may be taken as a function of voids ratio by means of equation (5.9).

A compression model utilizing the equations described here, has been programmed. A full code is available in Den Haan [ref. 5.20], and a demonstration version together with a short description and a number of worked exercises, is available on the internet at *hhtp://www.delftgeot.nl/so* by downloading the program *consef.exe* and WordPerfect file *consef.wp*.

Correlations of v_1 and b with ignition loss

Den Haan [ref. 5.18] shows that under virgin loading of clay and peat, a unique relationship exists between specific volume v and the vertical effective stress σ'_v. This means that so both the slope and intersection of the virgin-compression line in the plane ln v - ln σ'_v are known. Den Haan proposes to locate this intersection by reference to the specific volume v_1 at a stress of 1 kPa. With this reference specific volume, equation (5.15) is restructured as follows

$$\ln\left(\frac{v_1}{v}\right) = b \ln\left(\frac{\sigma'_v}{\sigma'_u}\right) \tag{5.22}$$

in which

v_1 = specific volume at a stress of 1 kPa (-)

v = specific volume ($v=e+1$) (-)

σ'_1 = unit effective stress (σ'_u = 1.0 kPa)

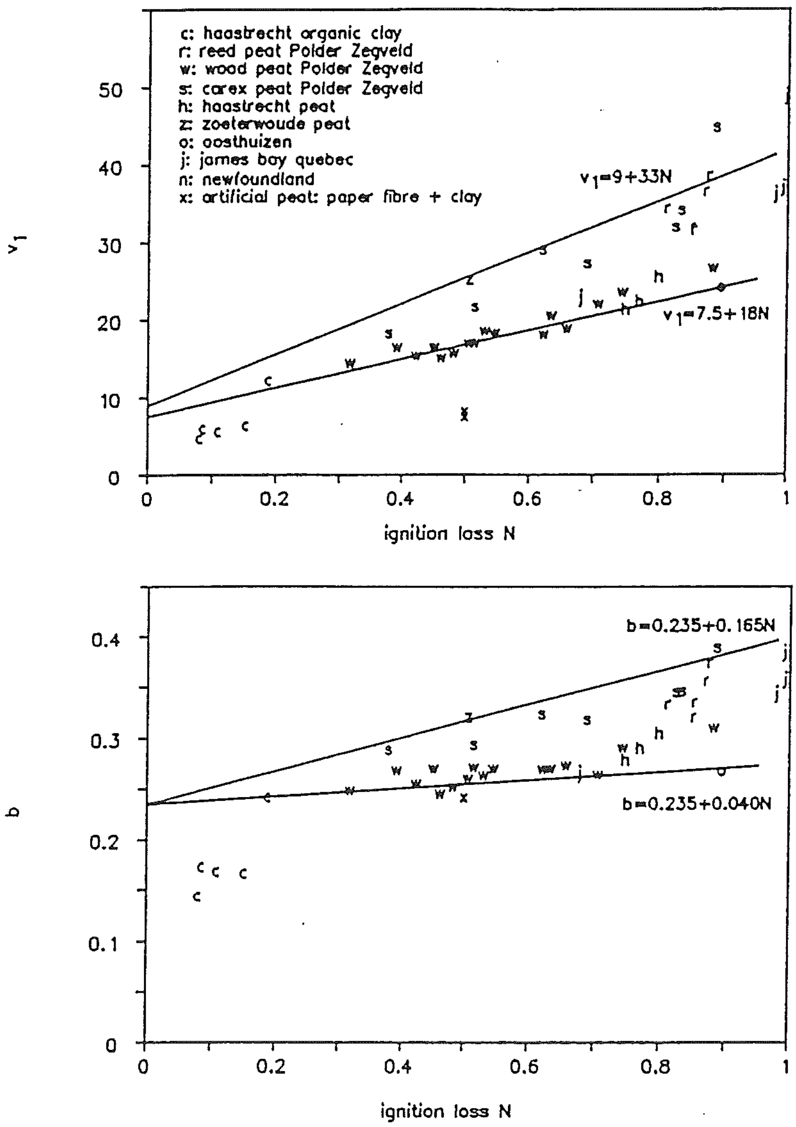


Figure 5.10 Correlation of compression parameter v_1 and b with ignition loss N for peats and organic soils.

Note that equation (5.18) is not valid for very high values of v because a certain density is needed before it is a soil type of material instead of a liquid.

Values of v_l and b determined for Dutch peats in the Polder Zegveld, and for other oedometer tests on peats reported in the literature, are correlated with ignition loss in figure 5.10. The degree of correlation is reasonable, but could be improved by developing local correlations, and diversifying the relationship for different botanical peat types. The correlations in figure 5.10 apply primarily to peat containing considerable wood residues.

The settlement can now be calculated merely by comparing the specific volume for the new load with the original specific volume. At the same time, a check can be made on the initial state. The initial volume based on the Den Haan relationship yields a preconsolidation stress σ_c. The degree of over-consolidation is found by comparing this preconsolidation stress with the stress in the field σ_{vo}, which quotient is referred to as the over-consolidation ratio OCR

$$OCR = \frac{\sigma_c}{\sigma_{vo}} \tag{5.23}$$

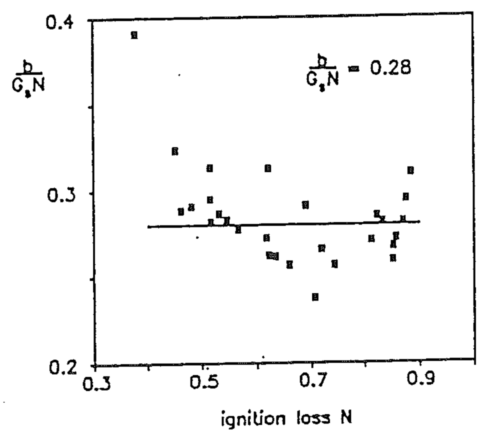


Figure 5.11 Polder Zegveld peats, correlation formula for natural compression index b.

Figure 5.11 shows that the equation (5.24) yields reasonably accurate results for Polder Zegveld peats.

$$\frac{b}{G_s\, N} = 0.28 \pm 0.028 \tag{5.24}$$

in which

G_s = specific gravity
N = loss on ignition (-)

The specific gravity G_s can be taken as a function of N according to equation (5.25), see figure 5.12.

$$\frac{1}{G_s} = \frac{N}{1.365} + \frac{(1 - N)}{2.695} \tag{5.25}$$

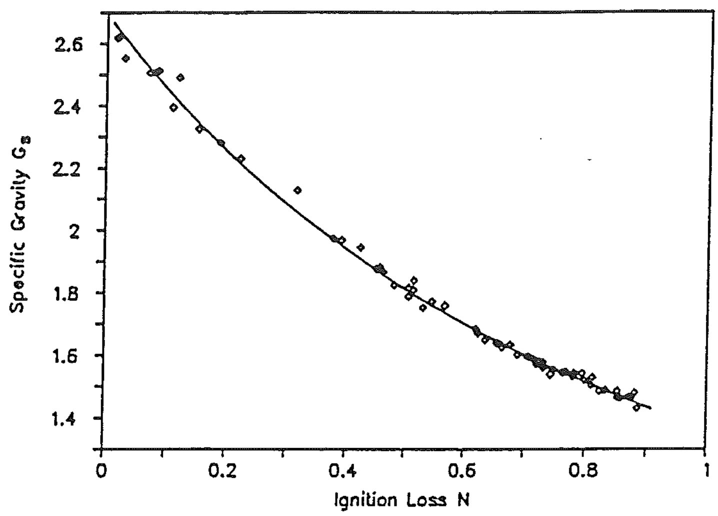


Figure 5.12 Correlation of specific gravity and ignition loss, Dutch organic soils.

c/b correlation

The creep rate index c is related to the creep stress index b by the same interrelationship as given by Mesri for C_α/C_c, see section 5.1.2. As with Mesri, values of approximately 0.05 - 0.06 are found for c/b in peats and organic soils.

5.1.5 Submergence, sustained heigth and tertiary compression

Unloading/ pre-loading

It is suggested that on reloading, the stress - strain - creep strain rate relationship is different than during first loading. The laws governing the change of the relationship are not yet clear. It is necessary to obtain formulations for the change in the relationship at any arbitrary stress or strain, to enable a general model to be constructed in which loading stages can be alternated by unloading stages. The preloading technique often used over soft soils and peats is an obvious example calling for extension of the theory.

Submerge of fill

Compressions in peat are often severe, with as consequence that a large portion of the fill settles below the groundwater level. Bouyancy then reduces the load exerted by the fill. In any realistic compression analysis this must be taken into account.

Sustained height
Dutch software exist which allows the option of so-called 'sustained height'. Using this option, the level of the fill is regularly adjusted to correct the designed final elevation for the incurred settlement.

Tertiary compression
Wilson *et al.* [ref. 5.21] postulated that, due to the intricate and complicated cellular structure of fibrous peat, it would exhibit not only "primary" and "secondary" consolidation characteristics, but also tertiary and quaternary etc. characteristics. Edil and Dhowian [ref. 5.22] and Dhowian and Edil [ref. 5.23] found that in standard oedometer tests on fibrous peat at low stresses, the normally straight secondary tail of the ε - log-time curve eventually steepened. They named this tertiary compression. Note that on a linear time scale settelement rate still decreases.

To avoid confusion with the well-known continuous steepening *towards* the secondary tail on application of small load increments (so-called Type III curves), the following definition should be adopted: *after* a long secondary stretch during which $\log\dot{\varepsilon}$ decreases equally with increasing log-time, $\dot{\varepsilon}$ gradually decreases less quickly. Usually, $\dot{\varepsilon}$ still decreases during tertiary compression, only less so than expected from further extrapolation of the secondary tail. Bishop and Lovenbury [ref. 5.24] showed examples of creep tests on London clay and Pancone clay where $\dot{\varepsilon}$ actually suddenly increases in the secondary stage. Kabbaj *et al.* [ref. 5.25] have ascribed this to the passing of the minimum voids ratio previously experienced in the geological history of the clay.

Hansbo [ref. 5.26] found a steepening ε - log-time plot but showed that no special transition appears when plotted on ε - t scales. However, the criterion to consider is whether the final slope is equal to or larger than C_α for large load increments. In the former case, steepening is probably associated with small increments, as described earlier. A case of the latter may be seen in tests by Krieg and Goldscheider [ref. 5.27] on a Dutch peat. The final slope of the test at 40 kPa is a factor 4 higher than at higher pressures.

Field occurrences of tertiary compression have also been noted (Candler and Chartres [ref. 5.28]; Simic [ref. 5.29]). Edil and Fox [ref. 5.30] observed both field *and* laboratory tertiary compression for a test fill in Middleton, Wisconsin.

The existence of tertiary compression has become somewhat controversial. However, from other materials it is known that creep curves sometimes exhibit a wavy course, possibly due to structural perturbations Freda [ref. 5.31]. There is clearly a need for careful study of this topic.

5.2 Stability

The stability of a construction is often ensured by control of stresses in the soil, to prevent occurrence of failure. Failure is the last phase of deformation behaviour, where the stresses have reached a level at which deformations increase continuously. Without further increase in stress, there appears to be insufficient resistance in the form of friction and/or cohesion between the solid particles of the grain structure to prevent them from moving relative to one another; thus the grain structure disintegrates. The stress level at which soil fails is known as the ultimate bearing capacity, or failure bearing capacity.

This ultimate bearing capacity is usually reached or exceeded in the first instance locally where the shear strength is too low. When the ultimate bearing capacity is exceeded, ground equilibrium or stability is destroyed and failure occurs. The calculation methods used to determine whether or not the construction is threatened by loss of equilibrium or stability, predict the shearing capacity on the basis of sometimes rough approximations. A distinction should be made between the following forms of failure:

- Failure by slip along a sliding surface;
- Failure by swelling/heave;
- Failure by squeezing.

5.2.1 Slip along a sliding surface

If the ground can no longer mobilise the shear strength required for equilibrium along a specific plane, a slip surface occurs along which the volume of ground shears. The critical shear stresses generally occur at places where the ground must bear finite loads, i.e. at the edges of extensive fills (terrace loading), under narrow fills (strip loading) or under berms (point loading).

For several decades, calculation methods have been developed in many countries to determine the stability of slopes and excavations on or in soil of low bearing capacity. Instabilities mainly occur beneath the edge of fill and at the edge of an excavation. Therefore, at these locations, the equilibrium of a specific mass of ground must be taken into account when calculating the stability. Usually, a hypothesis is established with respect to the planes along which the soil mass will probably shear, assuming that, at each point of such a shear surface, the available strength of the material is mobilised. The stability of slopes can be analyzed by trial and error in this way. With complex cross-sections, in particular, manual stability calculations are very time consuming. Over the years, numerous computer calculation methods have therefore been developed to determine the least stable situation rapidly and accurately.

The calculation methods for determining macro stability all use the stability factor SF. As design values of the parameters are used in the calculations, a stable slope must always have a stability factor greater than or equal to unity.

Peat behaves anisotropically. In practice, this anisotropic behaviour can probably be ignored in case of clay, but in the case of peat, such behaviour predominates [ref. 5.32]. The anisotropy of peat is caused by its fibrous structure. Under loading, the anisotropy is further intensified because the fibres are pressed further in the preferred direction. To investigate the effect of anisotropy in practice, this effect is incorporated into a finite-element model, using an anisotropic 'cap' (see for instance [ref. 5.33].

The anisotropic behaviour of peat also probably is accompanied by directionally- dependent strength (φ' and c'). In the stability calculations using the 'Bishop' method, when an embankment is constructed on peat an improbably low equilibrium factor is found in a number of cases. This low equilibrium factor is possibly caused by the fact that only shear strength parameters from triaxial tests are used which consequently are assumed to be representative of the shear strength over the entire slip surface, independent of local load vectors. The shear strength parameters from triaxial tests are possibly only representative of the active part of the slip surface. In the neutral and possibly also in the passive zone of the slip surface, shear strengths are assumed to be measurable by simple or direct shear tests and triaxial extension tests respectively (see figure 5.13). Furthermore, in the 'Bishop' stability calculation, circular slip surfaces are used which usually do not tally with the findings for shear planes in case of fills on peat and with the results of finite-element calculations. A good alternative consists in using 'Spencer' stability programmes in which not only circular but also non-circular user-specified slip surfaces can be involved in the calculations. At the moment, the methodology for direction dependent shear stress and non-circular slip planes is undergoing practice trials. Investigations are in hand to establish whether substantial differences exist relative to existing practice.

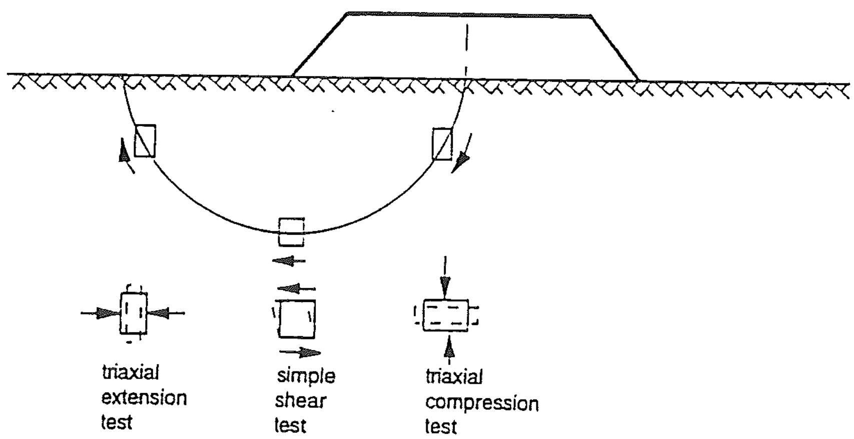


Figure 5.13 Slip surface and the required laboratory investigation

The stability of a slope is assessed by comparing the driving moment for a given sliding surface with the moment of resistance. A surface load has the following, mutually partly compensating effects, on stability:

- Increase in failure probability by increasing the driving moment;
- Reduction in failure probability by increasing the shear strength and resisting moment by increasing the effective stress (provided that the ground behaves as though drained) within the area of influence of the surface load.

With a concentrated load on or near a slope, not only is the load spread in the drawing surface, but also in the direction perpendicular to this surface. The increase in the effective stress and hence the shear strength in the subsoil is thus smaller than with a strip load.
In almost all cases found in the literature [ref. 5.34], the stability factor calculated with a three-dimensional model exceeds that of the corresponding two-dimensional model. This applies both to slopes with and without surface load. The greater the longitudinal dimension of the surface under load, the smaller the difference between the two stability factors.

In this context, different researchers have come to different results, depending amongst other factors on the shape of the three-dimensional slip surface chosen. A three-dimensional slip surface calculation or a three dimensional finite element calculation is not always available in normal consultancy practice. Therefore, in accordance with German guidelines, it is recommended that a conventional two-dimensional slip surface calculation be carried out in the case of a concentrated load and the concentrated load should be regarded as a strip load with the same force per unit surface.

The stability factor is thus probably underestimated, but the underestimation can be regarded as a hidden reserve. The greater the degree of cohesion contribution to shear strength, the greater this reserve will be.

The Bishop method
The Bishop method is a slip surface calculation method which assumes loss of equilibrium of a slope by the creation of a circular slip surface.
This method is often used in practice to determine the stability factor of a slope [ref. 5.33, 5.35, 5.36 and 5.37]. The circular shearing soil mass is divided into a number of vertical slices. A shear stress τ and a normal stress σ_n act along the circular slip surface.
The shear force is found by multiplying the shear strength along the slip surface by length b_i of the piece of slip surface of a slice.
Adding together the shear forces of all slices and multiplying this by the sliding circle radius R gives the total restraining moment generated by the existing shear strength. The driving moment is determined by multiplying the weight of the body of soil for each slice by the horizontal force component relative to the centre point of the circle, and totalling these for all slices. If applicable, an additional moment term is added to account for the presence of free water.
The stability factor SF is defined as the quotient of the restraining and driving moments.
The angle x_v is fixed at $45° + \varphi'_i/2$ for angles exceeding than $45° + \varphi'_i/2$. This is called 'cut off'. The value of $45° + \varphi'_i/2$ agrees with the maximum dimension of the circle in the Mohr-

Coulomb criterion.

The stability factor can be determined only for circular slip surfaces. By varying both the centre point and the radius of the circle, the minimum stability factor must be found iteratively. The limitations of the Bishop method are:

- The method is suitable only for circular slip surfaces.
- Only external moment equilibrium, external vertical equilibrium and vertical equilibrium per slice are satisfied.
- Horizontal equilibrium is calculated neither externally nor per slice; the sum of all interslice forces is non zero.
- The method makes an incorrect assumption with regard to the inter slice forces; this assumption has little effect however.

Implementation of the Bishop method to a computer program offers the following special opportunities:

- Automatic search for the minimum stability factor in a given pattern of centre points of slip circles.
- Use of a stress dependent c' and φ' in the soil layers.
- Simulation of an evenly divided top load or a linear load on the soil mass.
- Calculation of adjustment percentages for pore water pressures.

The Spencer method

Bishop's assumption that the resultants of the interslice forces are horizontal, is not generally correct. Spencer assumed that the resultant force makes a certain angle to the horizontal as extrapolated from the equilibrium conditions.

For circular slip surfaces, Spencer's method results differ by only a few percent from those using the Bishop method.
The Spencer method can also be applied to non-circular slip surfaces.

The Janbu method

In the Janbu method, a plane strain state is assumed, although any slip surface can be taken. The shearing soil mass is divided into vertical slices.
For each slice, equilibrium of vertical and horizontal forces and moment equilibrium apply. For the entire shearing soil mass. equilibrium must also apply.

A simplification of the method is known in which the resultant of the slice forces have no vertical components; this method is known as the Simplified Janbu Method [ref. 5.38].

The Morgenstern and Price method

The Morgenstern and Price method is suitable for any slip surface. The principle is the same as with the Spencer method where the three equilibrium conditions for each slice are satisfied, namely horizontal and vertical force equilibrium and moment equilibrium. Unlike the Spencer method, the Morgenstern and Price method assumes that the gradient angle of the interslice forces is variable.

Probabilistic approach

In a probabilistic approach to stability, assessments of the probability of failure are calculated with respect to the stability of a slope.
In the slip circle calculation method, such as the Bishop method, strength properties are introduced as stochastic variables; uncertainties concerning excess pore water pressure can be applied to the model in an indirect manner. Spatial variations of layer thicknesses, bulk density of the soil layers and the soil geometry are disregarded for the sake of simplicity. In addition, restrictions are applied to the random occurrence of a certain value in a layer, i.e. layer parameters having the same distribution and a continuous variation pattern.
The extent to which values within a layer correlate is described by an autocorrelation function. This function describes the rate of fluctuation of soil parameters in a longitudinal direction.
If it is assumed that the shear strength parameters vary stochastically within the soil layers, the stability factor computed by the Bishop method can also be regarded as a parameter undergoing a continuous stochastic change in the longitudinal direction of the slope. As a result, using uncertainties in the shear strength properties, the expected value of a stability factor, including the standard deviation, can be calculated by the Bishop method. The autocorrelation function of the shear strength properties can also be used to determine the autocorrelation function of the stability factor.
The statistical characteristics denoting variation in stability factor act as input for the actual probability calculation which is characterised by three stages. In the first stage, the probability of the Bishop stability factor being less than 1.0 at any specific point is calculated. In the second stage, the length effect is calculated and the third stage takes account of the effect of adjacent shear surfaces. For a further description, reference should be made to the literature [ref. 5.39].
In practice, probability stability analyses are used exclusively for special situations. The method is used for determining partial stability factors.

5.2.2 Uplift/heave

If the contact pressure, i.e. the effective stress between layers of low bearing capacity and a sand layer beneath this, fails due to high water pressure in the sand relative to ground water level, major deformations or loss of equilibrium can occur in the section of low bearing capacity and the layer will locally heave. Such a phenomenon can occur if the piezometric level in the sand layer rises, if for example this is dependent on a certain external water level at a river, the bed of which cuts into the sand layer concerned.

Vertical equilibrium

Bottom heave of an extensive excavation occurs where there is a loss of vertical equilibrium i.e. as soon as the water pressure in the aquifer below the excavation is equal to the weight of the ground above. The effective stress at the interface between aquifer and ground above is now zero. The resulting piezometric head in the aquifer is called the maximum piezometric head H_g. This is derived from the vertical ground stress and the water pressure as follows:

$$H_g = \frac{\sigma_v}{\gamma_w} + H_p = \frac{\sum_{i=1}^{n}[\gamma_i \cdot h_i]}{\gamma_w} - d_z + H_p \qquad (5.26)$$

where:

H_g = piezometric head in the aquifer layer in relation to a reference level (datum); the piezometric head is positive or negative if this lies above or below datum (m),

σ_v = vertical effective stress at the interface of the aquifer and the ground above (kPa),

γ_w = unit weight of water (kN/m^3),

H_p = piezometric head in the vertical concerned in relation to datum; the piezometric head is positive or negative depending on whether this lies above or below datum,

i = number of the soil layer concerned; runs from l-n (-),

n = number of soil layers above the aquifer concerned, including the layers above ground water level (-),

γ_i = unit weight of layer i including water (kN/m^3),

h_i = thickness of layer i (m),

d_z = thickness of layers which lie below ground water level and above the aquifer section concerned (m).

Figure 5.14 illustrates the above.

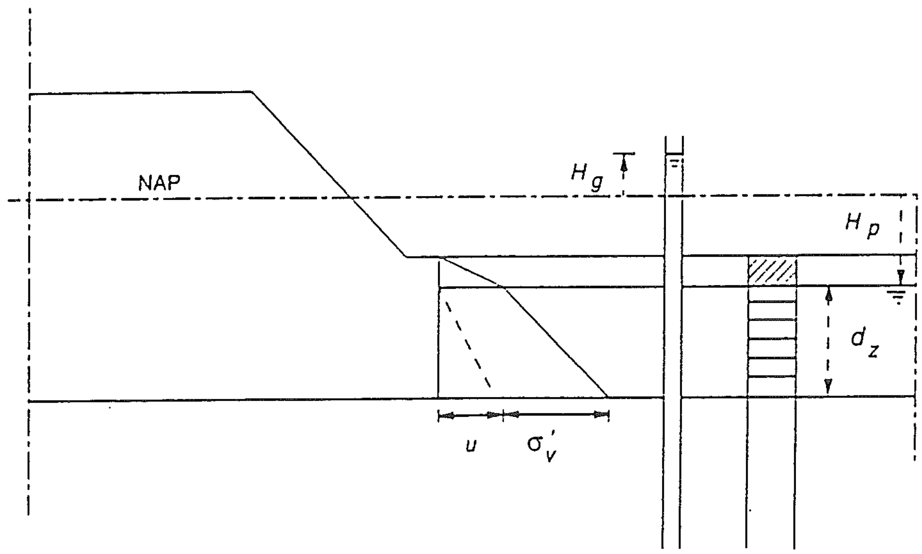


Figure 5.14 Vertical equilibrium with excess water pressure in the aquifer.

In the case of a narrow building site or excavation, stress distribution produces a favourable effect.

Uplift near dykes

The phenomenon of uplift can also occur near dykes resting on a layer of highly compressible soil of low bearing capacity with a subjacent sand layer aquifer, in which the piezometric head is dependent on the external free water level. The loss of vertical equilibrium with an excessive piezometric head in the sand layer can lead to unacceptable vertical and horizontal deformations and even loss of soil mass equilibrium.
The problem can be divided into three separate parts:

Ground waterflow behaviour.
: Using geohydrological data and geometry, the piezometric head in the sand layer beneath the low bearing capacity layer can be determined. For a description of the method for calculation the ground water flow behaviour, see [ref. 5.40].

Vertical equilibrium.
: By comparing of the weight of low bearing capacity layers and the piezometric head in the sand layer, equilibrium can be checked; uplift occurs if the effective stresses become zero for high piezometric head in the sand layer.

Stability and deformation behaviour.
: Depending on the calculated effective stress and the stiffness, either the stability, or the stability and deformation behaviour, are checked. Here the simplified stability method based on the Spencer method is used, supplemented with manual load and deformation calculation. This method first analyses the equilibrium behaviour followed by the deformation behaviour. For a description of the calculation methods for equilibrium and deformation behaviour, see [ref. 5.40].

5.2.3 Squeezing

If a highly compressible soil layer of low bearing capacity is sandwiched between a more or less solid upper layer and a solid lower layer (for example, sand), and the surface layer is subject to local load, there is a chance of the intermediate layer being squeezed away. This phenomenon is generally known as squeezing. Squeezing is caused by excess pore water pressure occurring locally relatively quickly in a layer of low bearing capacity, with the result that the shear stress exceeds the shear strength present.
Potential squeezing of the low bearing capacity layer is checked using the 'IJsseldijk' method. Here the partial factors described in section 4.5.3 are applied. In addition to the 'IJsseldijk' method, this section also gives a design rule and a general calculation method which allow approximate calculations to be carried out.

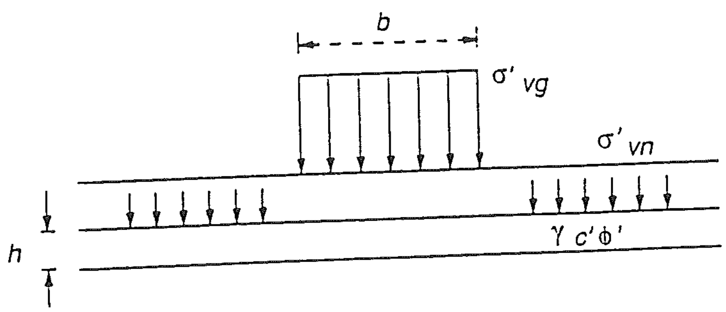


Figure 5.15 Squeezing of an intermediate layer as a result of local filling

'IJsseldijk' method

In the 'IJsseldijk' method, the vertical equilibrium in the compressible layer directly adjacent to fill is taken into account (see figure 5.15) [ref. 5.33, 5.35, 5.41, 5.42, 5.43 and 5.44]. The shear stress is assumed to be linear over the height of the layer of low bearing capacity. Thus the following relationship can be established for the allowable fill height:

$$\sigma_{vg} = \left(\sigma_{vn} + K + \gamma \cdot \frac{h}{2}\right) \cdot \left[\frac{e_{\lambda} - 1}{\lambda} - 1\right] \tag{5.27}$$

with:

$$K = c' \cdot \cot \Phi' \tag{5.28}$$

where:

$$\lambda = \frac{b}{h} \cdot \frac{\sin\Phi' \cdot \cos \Phi'}{1 + \sin^2 \Phi'} \tag{5.29}$$

σ_{vg} = vertical effective stress beneath the fill (kPa),
σ_{vn} = vertical effective stress adjacent to the fill (kPa),
c' = cohesion (kPa),
φ' = effective angle of shearing resistance (°),
γ = unit weight of the layer of low bearing capacity (kN/m^3),
h = thickness of the layer of low bearing capacity (m),
b = width of the fill (m).

If the passive pressure in the low bearing capacity layer is also considered, then a greater fill height is allowable. If the passive pressure is assumed to peak when horizontal equilibrium is attained in the squeezing layer then:

$$\sigma_{vg} = \left(\sigma_{vn} + K + \gamma \cdot \frac{h}{2}\right) \cdot \left[\frac{e^{\lambda} - 1}{\lambda} \cdot \frac{(1 + \sin \Phi')^2}{1 + \sin^2 \Phi'} - 1\right] \tag{5.30}$$

'Matar-Salençon' calculation rule

Using the Matar Salençon rule, allowable fill heights can be determined on the basis of the undrained cohesion of the foundation material [ref. 5.45]. The associated formula is:

$$\Delta p = \frac{1}{SF} \cdot \left(4.14 C_u^b + \frac{C_u^{\circ}}{h} \cdot x\right) \tag{4.32}$$

where:

Δp = allowable load (kPa),
SF = stability factor (-),
C_u^b = undrained strength of the upper side of the low bearing capacity layer (kPa),
C_u^o = undrained strength of the lower side of the low bearing capacity layer (kPa),
h = thickness of the low bearing capacity layer,
x = distance from the toe of the filling (m).

The formula gives the 'envelope' of the maximum load on the ground; this is clearly shown in figure 5.16. The allowable filling is the load resulting from fill which on average remains below this envelope.

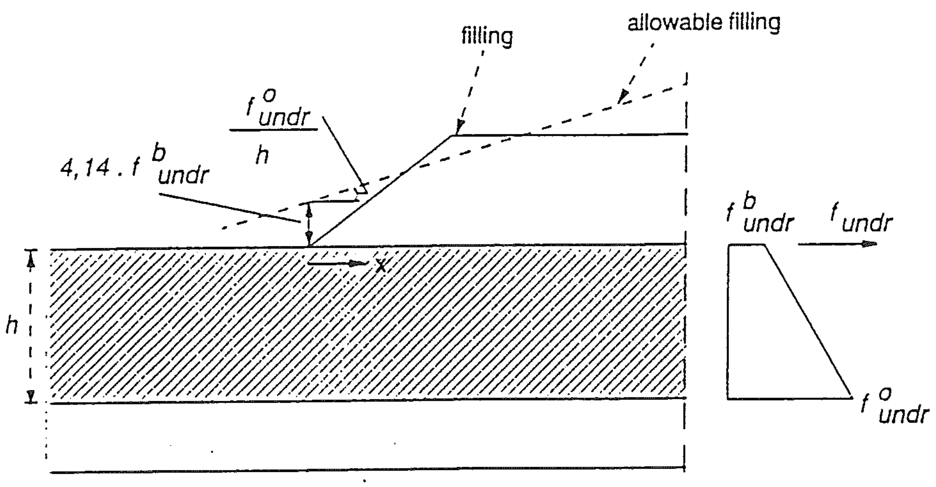


Figure 5.16 Permissible filling for horizontal equilibrium according to Matar Salençon.

5.2.4 Stability during construction

In addition to assessing the fully consolidated state of the slope, stability during construction must also be considered. This is necessary to determine whether the construction can be completed within a reasonable time and at which rate the various fill layers can be added.

Critical situations during construction are when fill layers must be added. These generate additional excess pore water pressures in the subsoil while excess pore water pressures resulting from previous fill layers have not fully dissipated. In general, determining the embankment fill schedule involves a number of steps.

- First it is calculated at what excess pore water pressure dissipation the planned total fill will be of sufficiently high stability; this allowable excess pore water pressure can then be expressed as a consolidation degree.
  In practice, this calculation is done by calculating the stability factor of the total fill for a number of consolidation degrees (for example 30%, 60% and 90%). The minimum consolidation degree required (for example 50%), relative to the desired stability factor for the total fill, is determined by interpolation.

- The hydrodynamic period is calculated, after which the development of consolidation is established; the minimum time period required to achieve complete filling is then established using the minimum consolidation percentage required.
  For example: say that a consolidation percentage of 50% is reached in 10% of the hydrodynamic period. The minimum period required is then compared with the performance time available.

- If the calculated minimum period is longer than the available performance period, the effect of vertical drains or other specific measures should be investigated (see section 6.1.2).

- The total fill is normally applied in a number of layers. The time of application of a fill layer depends on the height of the fill layer and the degree of consolidation in the subsoil as a result of previous fill. Slip surface calculations are used to establish percentage consolidation requirements for the subsoil in order to achieve a fill layer of a certain height. The safety factor required is taken into account. The consolidation percentage for the subsoil can then be translated into performance time or waiting time. By varying the thickness of the fill layers and the waiting periods, the performance period is optimized. If this minimum performance period is still too long, due to the various waiting times, then special measures can be taken such as those listed in the previous section.

In calculations for dyke improvements in the Netherlands, it must also be remembered that only a limited period is available each year for the performance of such work, namely from mid-March to mid-October. Also, in the first performance year, usually only a limited period is available for actual filling operations as in practice some months will be required for preparation. In the final performance year, time must also be reserved for finishing work, the application of a road surface etc.

5.3 Finite element calculations

The finite-element method is a good tool for determining two- dimensional behaviour. Generally, a compromise should be accepted in regard to the material model.

Because of the anisotropic character of peat, finite element calculations should be made with extra care regarding the input parameters and interpretation of the calculation results.

Most of the programmes available in practice use elastoplastic soil models. In so-called 'two-phase models' the combined behaviour of solid particles and water are simultaneously taken into account so that non-linear consolidation can be modelled.
The more realistic 'hardening' behaviour of soil (i.e. plastic behaviour prior to complete mobilisation of available friction or stiffness dependent on stress level) is not offered as an option in most programmes and in real situations is a definite shortcoming.

Currently, under the auspices of TAW research, experience is being acquired with the Cam-clay model and the Adachi-Oka creep model. These models are described in [ref. 5.33].

The use of a finite-element model calls for geotechnical expertise insofar as choice of parameters, schematization and interpretation of the results are concerned. In the TAW research programme, an investigation is in hand to evolve a guideline for identifying the parameters needed for the finite-element models.
Verification and validation projects are extremely important backups to correct use of finite-element models. Future developments in the field of finite-element modelling are:

- The anisotropy of stiffness and strength parameters
- The effect of stress rotation.

< < < < < < to be extended > > > > > >

5.4 Influence on objects

Soil constructions on highly compressible subsoils generate horizontal and vertical deformations which in turn cause stresses in foundation piles in or near the soil structure. Piles at the edge of a soil structure are horizontally loaded due to horizontal deformations in the compressible layer; this phenomenon is known as 'piles horizontally loaded by soil'. As the pile tip is generally firmly anchored in the underlying sand layer, bending stresses are introduced in the pile which can lead to pile failure. As a result of vertical compression of the soil layer, the soil in the vicinity of the pile acts as a vertical load on the pile; this phenomenon is known as negative skin

friction.
Because of the negative skin friction between soil and pile, extra vertical stresses are induced in the pile which can lead to settlement of the pile tip.

5.4.1 Piles under horizontal load from moving soil

In the 'De Leeuw' method for determining the behaviour of piles under horizontal load from soil pressure, first the soil behaviour and then the pile behaviour is calculated [ref. 5.46]. The stresses on the pile and the resulting pile deformations are determined from combining the two behaviours, after which the pile bending moments are found.

For simple situations, for calculation of the horizontal displacement of the ground, the IJsseldijk and De Leeuw tables can be used; see [ref. 5.47] of the bibliography. The assumptions are:

- Soil deformation does not cause volume changes (Poisson's ratio = 0.5);
- The highly compressible soil layer is homogeneous;
- A plane strain situation is assumed;
- The deep sand or foundation layer is incompressible.

A distinction is made between strip and terrace loading, and where the top of the layer of highly compressible ground can be represented by a stiff crust. A stiff crust can for example consist of a sand layer lying on the highly compressible layers. Where such layers are present above the highly compressible layer, the results of calculations with and without a stiff crust can be regarded as the two extreme values.

Using analytical expressions for horizontal stress and horizontal displacement, these values can be calculated at the pile location.

The interaction between the soil and the pile can also be calculated using finite element programs. With these both the soil and pile stiffness can be modelled (see section 5.3).

5.4.2 Negative skin friction

Over the years, many calculation methods have been developed to predict negative skin friction. In the Netherlands, for negative skin friction on piles the national standard NEN 6740 [ref. 5.48] should be used. For other applications, for example negative skin friction on diaphragm walls the Zeevaert/De Beer and the Slip method are amongst the better known and most common. These methods are described extensively in reference [ref. 5.49], the Slip method is summarized below.

Slip method

The slip method is an analytical method for calculating negative skin friction. It is assumed that complete pile/soil slip occurs when the ground exerts a downwards shear force on the pile. The shear force is calculated from the vertical stress using the formula:

$$\tau = c' + K_o \cdot \sigma_v \tan \delta \tag{4.33}$$

where:

τ = shear stress (kPa),
c' = cohesion (kPa),
K_o = coefficient of earth pressure at rest (-),
σ_v = vertical effective stress (kPa),
δ = angle of friction at ground/pile interface (°).

The formula does not take into account the reduction in vertical effective stress due to negative skin friction in the overlying layers. An extensive description is given in [ref. 5.50], which also covers multi layer systems.

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6 Construction techniques

6.1 Special techniques

To compensate for or to deal with the low density and the high compressibility of peat and organic soils, this section describes some special techniques. These include acceleration of consolidation, weight compensation and soil strengthening.

6.1.1 Lightweight materials

In certain situations, the use of lightweight materials may be contemplated. Generally, there is no way of defining where application of lightweight materials is definitely more advantageous. The findings of a costs/quality comparison between a structure with and a structure without light-weight materials are influenced considerably by the following factors, although these are not necessarily exhaustive:

- The sensitivity of the soil to settlement and the extent of settlement which has already taken place;
- The nett heightening required;
- The time available between fill and completion of the final structure;
- The permissible residual settlement after completion of the final structure;
- The possibility of damage to the surroundings through settlement and/or stability problems;
- The fill material availability and cost;
- The necessary extra arrangements for pavement to cater for use of lightweight materials;
- The space available and the angle of the embankment slope.

Since lightweight material can provide a watertight barrier between the subsoil and the top layer of soil, particular attention must be paid, during structural detailing, to the channelling away of rain water. This applies particularly if the structure has an open surface.

Where structures incorporate lightweight material, essentially the durability throughout the anticipated lifetime must be identical to that of a structure made of conventional material. In such cases, aspects such as sensitivity to vibration, temperature sensitivity or isolation effect and risk of chemical and biological attack must be taken into consideration. In the event of repair or any other types of activities on the structure (for instance the installation of pipes) it is important to prevent possible heave and any replacement material must be equally lightweight to avoid exerting any extra weight.

Volcanic tuff sand ('Flugsand')
Volcanic Tuff Sand ('Flugsand') is a material of volcanic origin extracted from areas such as the

Eifel on either side of the Rhine near Coblenz.
The non-cohesive material is coarse-grained, angular and porous and naturally contains a percentage of fine components; in terms of grain size distribution, the material is comparable to gravelly sand or sandy gravel. Flugsand naturally contains small quantities of heavy metals.

The Flugsand solid particles themselves have a porous structure and are thus classifiable as a lightweight material. A smaller or greater quantity of capillary water is retained by the solid particles. Reference is made to the apparent density which is semi-dependent on the quantity of water. The apparent dry density of the solid particles is 1300-2200 kg/m^3. These numbers are used as a quality index.

Flugsand deposited at site has, above water, a unit weight variable by compaction between 11-13 kN/m^3. The effect of Flugsand as a lightweight material is somewhat less in underwater applications than in above water (saturated volume weight: 15-16 kN/m^3).
It must be noted that the availability of good quality Flugsand has decreased dramatically in the last years.

One useful property of the material is the low coefficient of active ground pressure. Because of its low density, Flugsand is advantageous for applications as fill material behind retaining structures.

During construction, Flugsand is mainly subject to requirements in regard to density, permeability, CBR value and (for use in top strata) frost susceptibility.

Flugsand is used predominantly as a fill material and as a load distribution layer in road construction. Flugsand should preferably not be used in the top half meter beneath the upper side of pavement, owing, amongst other things, to possible compaction effects.

Natural materials of vegetable origin
Products of plant origin usually have a low mass density compared to other types of soils. In certain circumstances, they can be used successfully as a lightweight material.

Straw, turf, heather and timber fibre balls, shredded wood, saw dust and branch bark are obtained by processing products of plant or vegetable origin. The general material properties of such materials are: no cohesion, an angle of friction greater than 30° (upto 50°), a dry unit weight of 300-400 kg/m^3 and a saturated unit weight of 900-1000 kg/m^3. The comparatively high permeability of this material must also be taken into account.

It is a precondition for use of this material that its final location should be underwater to prevent oxidation. Turf, straw and heather are only delivered in compressed bales. When applying bales, the substrate or base of water courses must be cleaned up and smoothened. Through riding over

Paper- and synthetic drains

> > > > > > experience to be supplied < < < < < <

6.1.3 Vacuum consolidation

Vacuum consolidation can be defined as the expulsion of water from a soil mass by the application of a partial vacuum or air under reduced pressure in the uppermost layers of soil. In this section, air under reduced pressure will also be referred to as a 'vacuum', although, strictly speaking, this designation is inaccurate. Relevant reference material includes [ref. 6.9, 6.10 and 6.11]. The purpose of vacuum consolidation in combination with vertical drainage, is to achieve accelerated subsoil settlement. The vacuum is created by aspirating the air beneath an airtight film or membrane spread over the ground. A pre-load is thus created resulting in water discharge i.e., the pore water pressure falls and the effective stress increases (see figure 6.1).

To improve dewatering beneath the film, a layer of drainage sand is applied into which a set of horizontal drainage pipes is laid. Also, a system of vertical drains is applied to accelerate consolidation. Where the water can be channelled away quickly enough, pore water pressures in the vertical drains decrease immediately by an amount equivalent to the reduction in air pressure applied.

Since, during vacuum consolidation, no extra weight is applied at surface, vacuum loading can be carried out without time restrictions.
With respect to the effective stress in the subsoil, it makes no difference whether traditional or vacuum consolidation has been carried out, provided that an infinitely thick compressible layer and an infinitely extended site are present and surface loading at conventional consolidation is identical to the reduced air pressure. In practice, however, there are never infinite dimensions so that differences certainly do exist at the edges.

Furthermore, where a fill layer is placed after application of pre-loading under reduced air pressure, pre-loading has already initiated a stabilizing action so that the fill can proceed more quickly and usually with steeper slopes. The diameter and spacing of horizontal drains depends on the anticipated flowrate and length of the pipes. To illustrate the point, we can cite the use of drains with an equivalent diameter of 50 mm, arranged 3 m centre to centre in a sand bed of 0.5 m thick. It is a good idea for the drains and pump to be installed as close as possible above the natural groundwater level in order to maximize the reduced pressure effect. Furthermore, this effect, as consolidation proceeds, is favourably influenced by settlement of the system. The membrane with for example a thickness of 1 mm is spread out over the site in strips; joints are either heat bonded or glued.

The extremities of the membrane are connected on the edge of the area to be consolidated with a poorly permeable layer for instance by excavating a ditch in this layer or by constructing a vertical sealing wall. In the event of the membrane terminating in a sandy (permeable) layer, the groundwater table on the outside is also lowered and 'false' air entrained beneath the membrane resulting in loss of underpressure.

The membrane has to be pierced to accommodate vacuum pipes and piezometers. These piercings therefore need to be flexible such that even at the anticipated ultimate settlements, impermissible stresses in the membrane are avoided. The preferred technique therefore must be to pass the pipes out sideways under the membrane. The sand in the sand bed needs to be free from coarse particles that might damage the membrane once this is under tension.

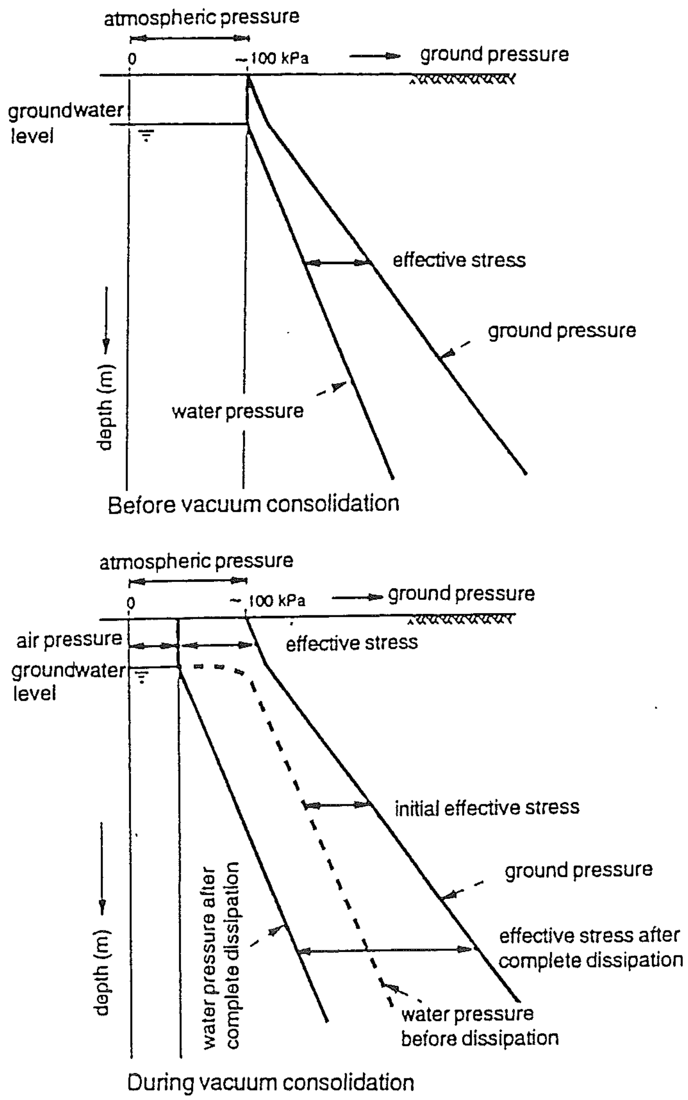


Figure 6.1 Ground, water and air pressure before and during vacuum consolidation.

The results of measurements in two test areas using vacuum consolidation [ref. 6.12] have led to the following observations:

- In laboratory testing of undisturbed samples, pre-loading in the form of reduced air pressure should already be taken into account. Around the drain a substantial vacuum will be created very quickly and this may result in a marked drop in permeability and possibly the release of air dissolved in the groundwater. These aspects may lead to a prolongation of the consolidation period.
- The changes in pore water pressure readings are readily explicable in terms of the consolidation theory. The hydrodynamic period deduced from the measurements was however decidedly shorter than the pre-determined value, particularly in peat strata.
- The first test area, operating mainly under vacuum loading, consolidated more slowly than the second test area subjected both to vacuum loading and a sand fill. The explanation for this may be reduced air bubble formation in the second test area.

The back-calculated hydrodynamic period for the first test area was approximately 60% higher than that of the second test area, yet the soil stratification was virtually identical.

Often a combination of vacuum consolidation and filling is used. In figure 6.2, ground, water and air pressures are presented for such a case. Although the precise change in pressure is dependent on local conditions, the figure certainly provides a good insight into the influence that simultaneous filling has upon the vacuum consolidation effect.

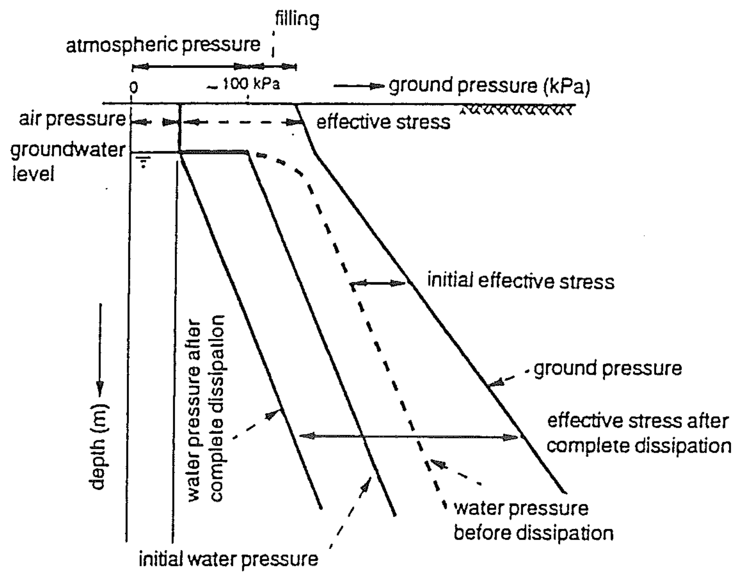


Figure 6.2 Ground, water and air pressure during vacuum consolidation with simultaneous filling

Deformations and stability

When vacuum loading is employed, the surrounding soil tends to move towards the vacuum loaded area. After a filling, the soil should tend to move away from the area. By combining vacuum consolidation with filling, the resulting deformations should therefore be smaller than use of fill alone.

Thanks to the application of underpressure, no plastic deformations occur at the edges. With fast simultaneous filling a plastic zone can be produced in the soil beneath the fill and particularly close to the edge. Just how the soil behaves in the plastic area depends largely on the properties of the material. Certain types of clay are known to display volume-resistant behaviour after they have attained the ultimate bearing capacity. This means, upon subjection to load, further compression of this particular material can only occur if lateral deformations take place. If these deformations are prevented by the surrounding soil mass, the plastic area is not very compressible. If the lateral deformations are not impeded, squeezing phenomena can occur which can lead to discontinuities in certain verticals. The effect of the vertical drainage system may be detrimental.

Where vacuum preloading is combined with filling, changes in pore water pressures must be taken into account. Consequently, for the periods immediately after application of a fill layer, stability is controlled with the aid of a slip circle method. Where fill is designed to cater for a temporary situation, a lower safety level is applied.
However, if the underpressure system fails for one reason or another, the effective stresses in the sand layer between the membrane and groundwater decline very rapidly, which is also the case with the stability contribution of this layer. It is therefore essential also to perform a stability analysis in the case of loss of air under pressure in this layer.

Pore water pressures

When using vacuum consolidation plus simultaneous filling, the total pore water pressure is higher than without the fill although there can still be a nett reduced pressure relative to the original pore water pressure. A higher overall pore water pressure has the advantage of accelerating reduction in excess pore water pressure in kPa (not in percentage terms). The quantity of air released is also not as great so discharge is less hampered; furthermore, less water will flow in from the environment. The deeper the drainage system is installed, the greater reduction in hydraulic head achievable by vacuum installation. Of course, precautionary measures will need to be taken at the edges of the loading area in order to prevent air leakage. Influx of water from the surroundings is bound to be higher. Generally, it is advisable to position the horizontal pipes as near as possible to the existing polder water level. Deeper installation is usually impractical and hence non-economical.

6.1.4 Columns formed in the ground

The use of columns formed in the ground will result in increasing the load bearing capacity of the foundation layer, reducing settlement, accelerating the consolidation process and improving the stability of the earthworks. Columns consist of materials such as gravel or sand or a mixture of the existing soil and lime or cement. Such columns are used in a triangular or square pattern. The type of column, column diameter and the layout pattern largely determine the end result [ref. 6.13].

As regards the load transfer from the earthwork via columns to the subsoil, a distinction can be made between two alternative modes of load transfer from the earthwork via columns to the subsoil:

- The foot of the column is installed in or on a bearing stratum. In the event of the bearing capacity of the foundation layer needing to be improved substantially and/or the settlement being greatly reduced, the foot of the column is often installed in a firm layer. For geohydrological or environmental reasons, when using gravel or sand columns, it may be desirable or necessary to prevent short circuiting between the free water table and deeper seated groundwater. This is achieved by keeping a layer of compressible soil (clay or peat) between the foot of the gravel/sand column and the bearing layer. By resorting to a number of techniques, measures can also be taken in the foot of the gravel or sand column such that no vertical water flow occurs.
- The foot of the column is located at a comparatively great depth in the same poor bearing layer.

Even in such circumstances, there should be less settlement since settlements are smaller at greater depth; the overburden pressure here is always greater than near the surface. Furthermore, the ground around the column is densified by in situ formation of the gravel column, the stiffness of the ground being increased locally. The most important reasons for the use of columns in this particular case is not settlement reduction but increased stability. The location of the anticipated slip circle determines the location and the depth of the columns.

Gravel or sand columns

The diameter of gravel or sand columns varies between 0.5-1.5 m. This variation depends on the stiffness of the weak subsoil. Thorburn [ref. 6.14] stipulates that the undrained cohesion of the poorly bearing layers needs to be over 20 kPa, whereas according to Greenwood [ref. 6.15] application is already possible if the undrained cohesion is over 7.5 kPa. However, with such low cohesion, a comparatively large quantity of sand or gravel is required.

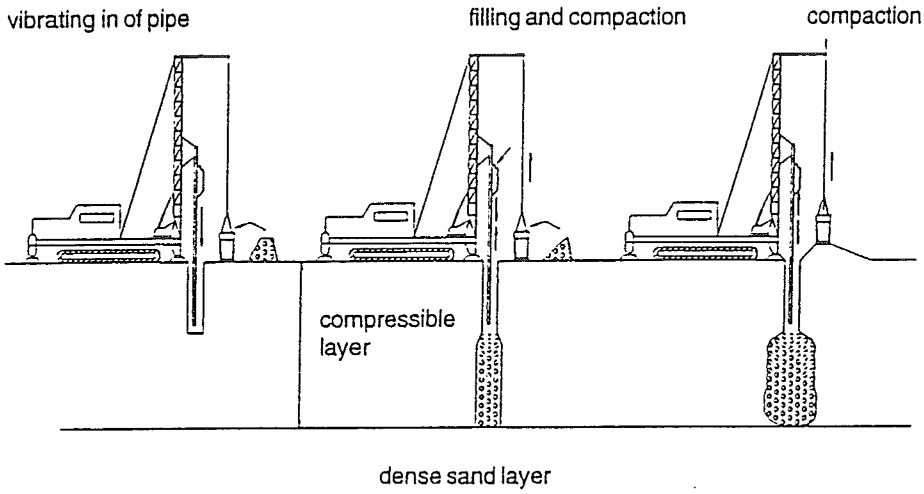


Figure 6.3 Principles of the composer system

As far as gravel and sand columns are concerned, a distinction can be made between a total of four important methods of installation.

The composer system

In the composer system, a limited quantity of dry, coarse sand, for instance, to a height of 1.0 m is poured into a pipe at ground level via an airlock system (see figure 6.3). An air overpressure is created above this sand column after which the pipe is vibrated to depth. A quantity of sand is then added and the pipe raised to a height of 1.0 m, for example, so that a quantity of sand is discharged from the pipe under the action of air overpressure.

Afterwards, the pipe is gradually vibrated downwards, air overpressure and a constriction in the pipe ensuring that no sand is forced back into the pipe. The diameter of the target sand column is determined by the length over which the pipe is vibrated downwards. If short circuiting of a column with the bearing sand layer must be avoided, the initial pipe charge can be a sand cement mixture.

The Franki system

In the Franki system, a gravel or stone plug is formed at the base of a pipe. The pipe is then driven down to the requisite depth by an internal drop-weight, after which the pipe is gradually filled and withdrawn while the fill material is rammed out by means of the internal drop-weight. Soft soils are thus displaced by a gravel or stone column. The Franki system is illustrated in figure 6.4.

square pile spacing	β
2.0 m	1.0
1.8 m	0.9
1.5 m	0.8

Table 6.1 pile spacing and settlement reduction rate.

Several field investigations have found that ground improved by sand compaction piles show slight displacement after the fill has been placed. However, attention must also be paid to displacement in the surroundings that may occur during the execution of sand columns.

To determine the influence of compacted sand piles on the strength and stability of the ground, two different assumptions can be employed:

1. The soil and column complex is introduced at the mean shear strength of both components. In so doing, the difference in stiffness between both elements and the possibility of 'progressive failure' should be taken into account.

2. The load on compressible subsoil is reduced by the proportion supported by the stiff columns and only the strength parameters of the original soil layers are taken into account, as well as accelerated soil drainage by the columns.

The strength of a composite foundation can be calculated with the equation stated in the CUR-166 publication [ref. 6.16].

Lime and cement columns (deep mixing method)
Lime and cement columns are installed in the ground using a hollow tube installed to the required depth by a drilling rig; instead of a drilling head, an auger is employed. Once the pipe has been installed at the appropriate depth, finely ground unslaked lime (CaO) or cement is blown to the bottom of the pipe. At the same time, the pipe is withdrawn using the auger, the rotating motion of the auger ensuring good mixture of lime or cement and soil. The quantity of unslaked lime or cement pressed into the column depends upon the water content of the clay. In the case of unslaked lime, this quantity must remain limited to 10-20% of the dry matter content of the clay; should this percentage be exceeded, there is a strong chance of the groundwater boiling.
Lime column diameter can vary but is generally no more than 0.5 m.

Limitations are imposed on use of lime and cement columns insofar as the organic content of the soil is concerned; application in peaty areas is therefore inadvisable.

In the installation process of lime or cement columns, no soil displacement takes place. Some increase in strength may be observed due to the extraction of porewater for the hydratation process of the lime/cement. Apart from this, the calculation principle for lime or cement columns is the same as for the gravel column. Through soil settlement and slight column deformation, the weight of the fill transfers increasingly via the column to the subsoil.

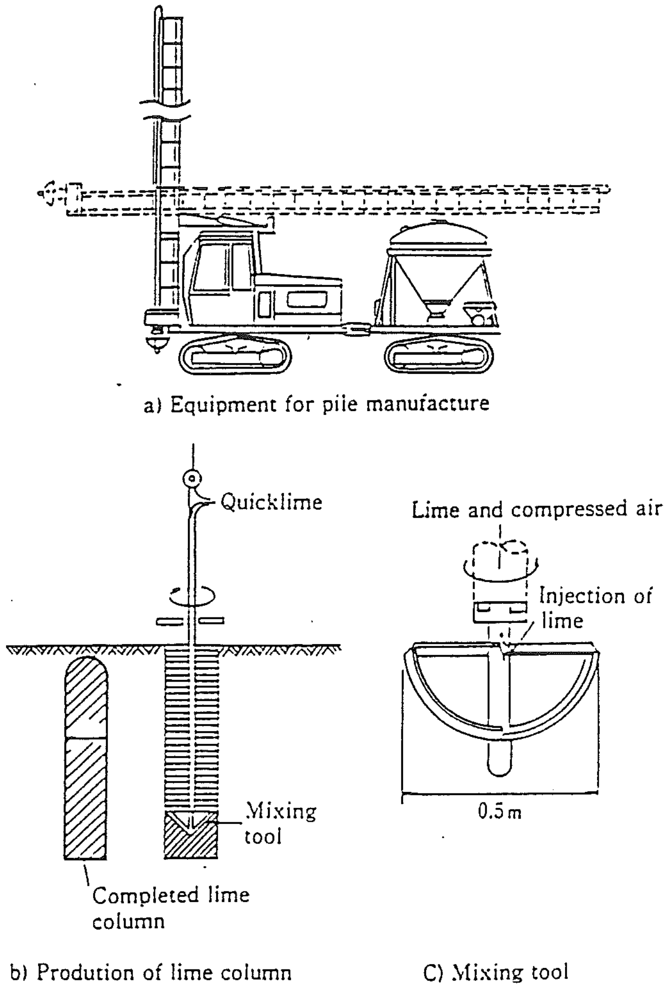


Figure 6.7 Lime column method (Sweden).

Sweden has developed a method to form lime piles called Lime Columns [ref. 6.17]. As shown in figure 6.7, an auger is inserted to a specified depth by rotating a stirring blade with a special edge, lime powder is injected to force mixing with soil at the withdrawal of the auger, and columns of this pile type are created. The method employs lightweight, portable construction equipment. In general, it has been applied to layers down to 10 m, with 50 cm diameter piles as a standard.

The method developed in Japan is similar to the Swedish method, but as shown in figure 6.8, it uses a large-scale equipment with a variety of stirring blades of complicated structure. It can work down to a maximum depth of 60 m and with a finished pile diameter up to 2 m.

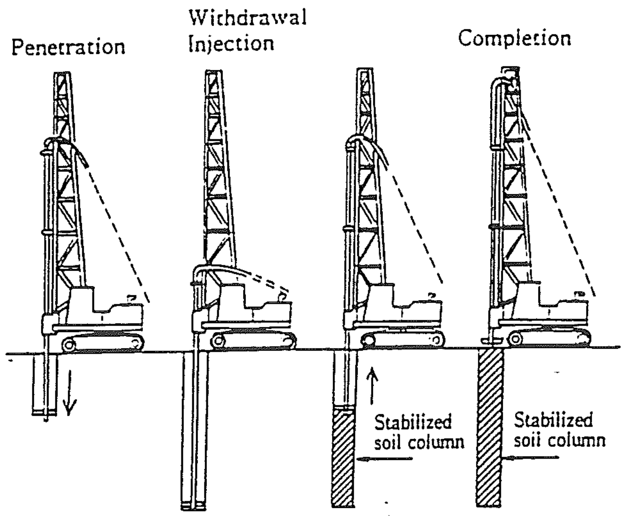


Figure 6.8 Deep mixing method (Japan).

Settlement of lime- or cement column reinforced soil

Settlement of foundations improved with the deep mixing method is calculated by the following simplified equation of the concentration of stress in improved columns.

$$S_1 = \frac{1}{1 + a_p(n-1)} \cdot S_0 = \beta \cdot S_0 \tag{4.35}$$

where:

S_1 = Settlement of improved foundations
S_0 = Settlement of unimproved foundations
β = Reduction rate of settlement
n = Stress ratio.

The stress ratio is usually 10-20, from the ratio of the two coefficients of volume compressibility, and assuming that the improved columns and unimproved ground settles evenly.

Strength of lime- or cement column reinforced soil

During laboratory investigation into lime/soil mixtures with a high organic content, it became apparent that the shear strength of the material did not increase as a function of time [ref. 6.18]. Also it was discovered that combinations with gypsum to this end produced no major improvement [ref. 6.19]. Therefore, there seems little point in using lime or cement columns in peat layers.

The strength properties of cement columns generated similarly to lime columns are better than those of lime columns. Cement is sometimes mixed with lime; the mix ratio depends, amongst other things, upon the type of soil and the strength required.

When the deep mixing method is applied to road embankments, pile like stabilities are usually expected in the treated soils. Designs usually consider improved columns and soft foundations as one, a so-called composite foundation.
Figure 6.9 shows the relation between the unconfined compressive strength and failure strain of peat stabilized with cement.

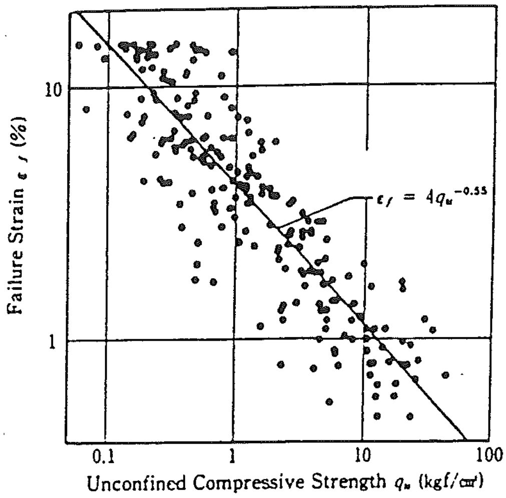


Figure 6.9 Relation between q_u and ε_f of stabilized peat.

Here ε_f is above 2% for q_u over 2 kgf/cm^2 in well stabilized peat, while ε_f is above 10 % for q_u = 0.1-0.5 kgf/cm^2 in unimproved peat. There are apparent differences in the stress-strain characteristics of the improved columns and the unimproved foundations, design methods considering stress-strain characteristics in both have not been established. At present, the strength of a composite foundation is determined by calculating areal averages of strengths, using the equation:

$$c = c_p \cdot a_p + c_o(1-a_p) \tag{4.36}$$

where:

c_p = Shear strength of improved columns
(= unconfined compressive strength q_u/2)
c_o = Shear strength of original foundation
(=cone bearing capacity q_c/20)
a_p = Improvement ratio (=Area of improved columns/total area).

In many projects a_p=50%, and at low a_p, settlement of foundations between improved columns may occur. In foundations under uneven loads such as fill slopes, this kind of settlement may result in horizontal forces acting on the improved columns and the result may be failure.

Ground does not have homogeneous engineering properties, and peat deposits in particular have very heterogeneous properties both vertically and horizontally, as may be presumed from their process of formation. Therefore, there is great variation in the strength of improved columns

after mixing and stirring.
Strength standards in the deep mixing method requires that the 28-day strength of improved soil should generally be 'above the design strength'. However, it would require very large amounts of improving material to make all improved soils satisfy design strength due to the local dispersion of strengths, and so uneconomical. A standard 70 % pass ratio of the average settlement and frequency distribution of strength is generally acceptable for sufficient stability.

Jet grouting
In the above described method to produce lime/cement columns, the peat is mixed in place with lime and or cement. Using the jet grout technique, the soil will be cut away and replaced with grout. The 1-phase jet grout technique is based on the cutting capacity of a single grout jet, injected at a pressure of 300 - 500 bar. In the so-called 2-phase jet grout technique, the penetration of the grout is aided by compressed air. The 3-phase system is based on two injection points. Here the soil is cut away by a water-air jet which is placed just above the jet nozzle of the grout. Figure 6.10 shows the jet grout process using the 3-phase system.

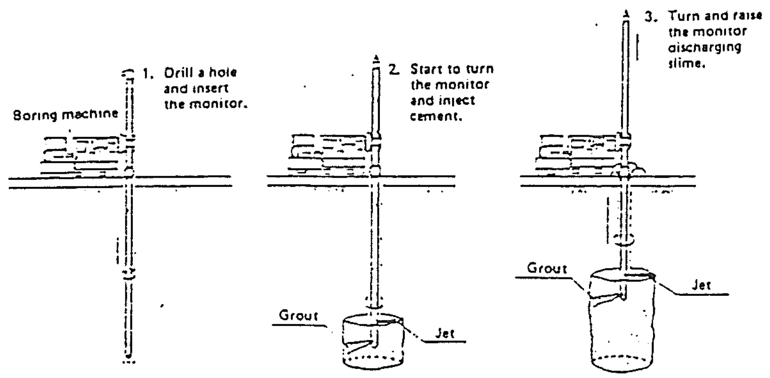


Figure 6.10 Schematic procedure for the 3-phase jet grout technique.

With jet grouting, depending on the chosen (1, 2 or 3-phase) technique, columns with a diameter from 1 to 3 m can be made in sand and (organic-) clay. In peat layers, due to the fibrous structure, the realised column diameters are smaller (1 to 2 m).
When jet grout columns are installed down to a good foundation layer, they can be used as foundation elements, to relieve compressible peat layers.

6.1.5 Geosynthetics

Geosynthetics can be used for reinforcing, separating, filtering and draining subsoils [ref. 6.20].

These functions are dealt with separately below.

Reinforcement function

In a reinforcement function, geosynthetics are used to prevent loss of stability or impermissible deformation of soil structures. They are employed primarily for fills on a soil with a low bearing capacity and for the construction of steep slopes, as well as for access on unhardened surfaces.

Separation function

A geotextile can be used to prevent material loss and undesired mixture of materials.

Filter function

The transport of fine particles from a soil mass can be prevented by use of a geotextile. This is especially important for dynamic structural loads ('pumping').

Drainage function

Geosynthetics are also employed for drainage of surplus water and for accelerating the consolidation process.

A distinction is made between the following geosynthetics used in civil engineering:

- Unwoven membranes consisting of randomly oriented inter-connected filaments;
- Fabric made of threads or strips arranged in a regular pattern;
- Grid consisting of net-like or gauzy structures with a 40-90% open surface;
- Membranes or membranes offering permeability as the prime characteristic;
- Certain combinations of membrane, textile and grid.

The materials used for producing geosynthetics are thermoplastic polymers. The target function imposes special requirements on the material properties and the structural form of the geosynthetic concerned.

Ultimate strain is likewise temperature dependent primarily for the PE and PA types of material. As the temperature rises, so does the ultimate strain; when the temperature falls, so does the ultimate strain.

Since the strength of polymer materials is time dependent, the length of time for which the load is applied is also important. Hence, temporary structures demand a lower material safety than permanent ones do.

The effect of moisture also plays a part with polyamide (PA) in the form of a lower tensile strength of the order of 600-800 N/mm^2 and a higher ultimate strain ranging from 18-30% (values at T = 22°C). The tensile strength of textiles is higher than that of membranes. In the case of textiles, load transfer is influenced considerably by component structure and orientation. Textiles are often stronger in one direction than in another.

A reinforcing geosynthetic is expected to offer both strength and stiffness. Deformation of the geosynthetic is a prerequisite in order to achieve certain load transfer; the entire structure should however deform as little as possible. Consequently the reinforcing function requires the load is transferred at a strain not exceeding 5-6%, resulting in a high elasticity modulus. PET fabrics, particularly, which possess high tensile strength and stiffness and low creep, are particularly noteworthy. In the event of lower stiffnesses being required, PA fabrics can be applied. PP and PE fabrics are usable if loads are lower and higher strains are regarded as permissible. Membranes can be used if the loads are lower still.
Creep is the deformation which continues to occur over the period following a certain initial deformation under constant load. This particular phenomenon occurs in geosynthetics via re-orientation of the molecular chain. Depending upon the length of time and material involved, the creep occurring can be as high as approximately 150% of the initial deformation. PE and PP material are particularly creep-sensitive. In situations where high load levels lasting for long periods occur, PA and PET materials would therefore be more suitable. A major part of the total geosynthetics extension, i.e., an important part of structural deformation, is occurring whilst execution is in progress.

The need for geosynthetics to be both permeable and to act as a ground seal are often conflicting. Water permeability places a lower limit on pore size, while the soil seal function defines the upper limit. Furthermore, both aspects are affected by the way in which the geosynthetic is used in the structure; existing ground pressure, particularly in the case of membranes, can result in material compression.

The friction between the ground and the geosynthetic which is important for load transfer depends on the geosynthetic application and on soil properties such as cohesion and angle of internal friction.

In normal circumstances, geosynthetics are extremely resistant to chemical attack. In extreme situations, resistance appears to be material and environment dependent. A small reduction in tensile strength may occur as the result of normal biological activity in the soil. Resistance to any acid or alkaline solution present in the soil may also be important. Furthermore, geosynthetics are limitedly resistant to UV radiation in sunlight. Resistance to mechanical damage such as puncture, tearing or wear depends on the manner of application and on the geosynthetic not to mention the soil concerned. Geosynthetics offer a fairly high mechanical wear resistance.

Possible applications of geosynthetics in connection with soils of low bearing capacity are:

- In steep slopes, as a reinforcement;

- In fills in the form of a reinforcement for less suitable fill material;
- In road foundations as a reinforcement and also as a separation between the foundation material and subsoil, as a filter or as a drainage agent;
- In the subsoil and foundation layers as a reinforcement for load spreading purposes or as filter and drainage materials for draining excess pore water accumulated during the settlement process.

During execution, a distinction can be made between the following activities for geosynthetics:

- Transport;
- Storage;
- Laying;
- Connection of strips;
- Soil filling.

The geotextile may be mechanically, chemically or physically damaged whilst these activities are in progress. Contact with sharp objects, harmful chemicals, not to mention exposure to fierce sunlight, must be avoided. Geotextile is supplied in rolls. The orientation of roll connection via overlapping, stitching or sewing needs to be parallel to the direction of the load. Geotextiles usually have different strengths in length or width direction. Obviously, the load direction should be as close as possible to the largest strength direction of the textile.

When using geotextile below water, wherever practically possible, the strips of textile need to be coupled together in order to limit the risk of inadequate overlap as far as is possible. During filling, a so-called 'mud wave' may occur by local application of too much material onto the canvas in one go. Thereafter, the geotextile will no longer lie horizontally.

Geosynthetics in structures on weak subsoil largely act as reinforcements. Below is a description of methods used for calculating such reinforcing structures. The methods involve determining the ultimate bearing capacity of the structure and working out what strength of geosynthetic is required. Apart from the methods listed below, finite element calculations can also be carried out, although there are relatively high attendant costs and in addition soil parameters need to be known very accurately.

Generally, failure is ascribable to one of five modes:

- Exceeding the bearing capacity;
- Internal loss of stability;
- Complete stability loss in a circular slip surface;
- Complete loss of stability along a straight slip surface;
- Loss of stability as the result of squeezing;

The reader is referred to [ref. 6.21 and 6.22] for the formulae to be applied in the calculations.

6.1.6 Reinforced earth

Reinforced earth can be described as soil to which another material such as plastic or steel has been added [ref. 6.23]. The added material gives the soil greater stiffness and/or higher strength.

Reinforced earth can be produced in the form of a structure consisting of:

- Soil plus geosynthetics (see section 6.1.5);
- Soil mixed with plastic filaments (for instance, Texsol);
- Soil with steel reinforcing strips and a covering of concrete panels or steel panels (for instance, Terre Armée).

The most significant property of reinforced soil is its ability to create steep or even vertical embankments. Apart from this, the construction time for reinforced soil structures is short and costs are much lower than for traditional structures such as a retaining wall.

When using reinforced soil on highly compressible subsoil, deformation and stability need to be looked at much more closely. Without sustaining damage, reinforced soil can tolerate higher differential settlement and/or angular rotation than a reinforced concrete L-wall, for example.

6.1.7 Load relief structure

Timber load relief floor
At one time, timber load relief flooring used to be in common use. It involved creating a timber pile foundation below groundwater level and surmounted by a timber frame and a floor on top. To ensure longevity, the structure had to be installed below water. Ground fill was then applied to the floor. The load relief floor was primarily intended to reduce the horizontal load on the pile foundation of an abutment.

With the help of present day materials such as steel and pre-stressed concrete and use of raked piles such horizontal loads can be assimilated somewhat more readily. Timber load relief flooring has therefore virtually passed into disuse.

Cakar Ayam System
The Cakar Ayam System (also known as the 'Chickenfoot System') involves placing reinforced concrete pipes vertically in the ground and inter-connecting them via a reinforced concrete raft placed over the pipes. Depending upon the condition of the soil, the length of the pipes is 1.5-3.5 m, the diameter 1.2-1.5 m and the wall thickness typically 8 cm. The pipes are spaced

2-2.5 m apart.
The pipes are normally filled with soil; if the pipe has a concrete plug, the fill can also be omitted [ref. 6.24]. The system is designed to improve the load bearing capacity of the surface layer, the bearing capacity of the subsoil is increased and the settlements reduced. Also, the pipes impede horizontal deformation of the subsoil. The structure can be used for both static and dynamic loads. The system has been used a number of times in Indonesia; no applications are known in the Netherlands.

Reinforcing mat on concrete piles

The reinforcing mat on concrete piles is applied as a load relief structure. Its effect is similar to the columns described in the previous section; it is also used in an attempt to reduce ground pressure [ref. 6.25].

This stress-reduction is achieved by most of the surface loading being carried by the deeper-seated strong bearing layers via the reinforcing mat and piles. The reinforcing mat can transfer load to the pile heads only if a certain settlement of the soil has taken place.

The tensile stresses generated in the reinforcing mat must be lower than the permissible tensile strength. In the event of no reinforcing mat being used, larger concrete caps are required on the columns or much tighter column spacing. Raked piles are then also necessary (see figure 6.11).

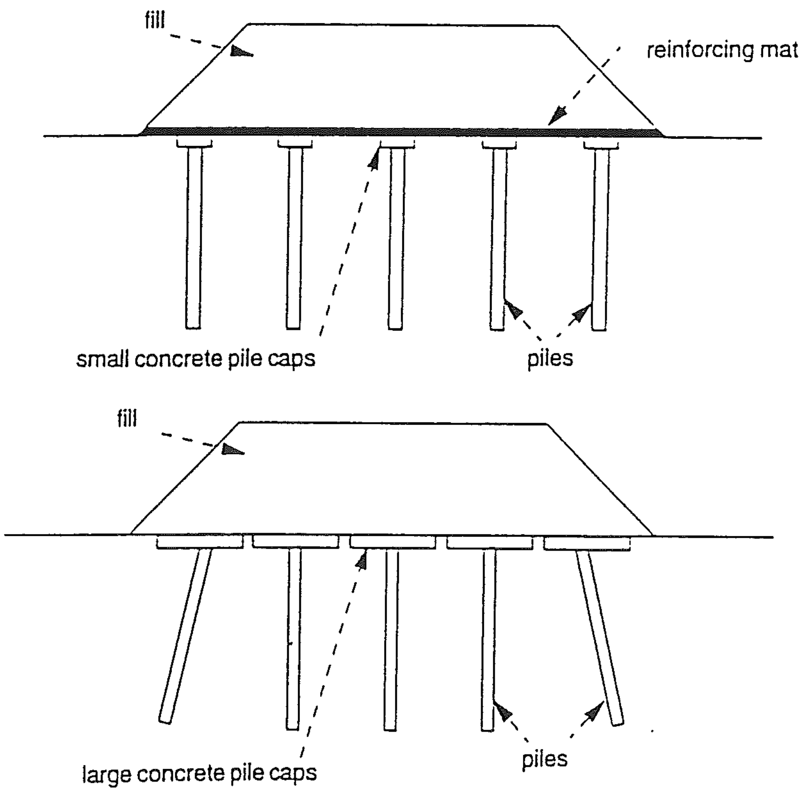


Figure 6.11 Piled soil structure; top with and below without reinforcing mat [ref. 6.26].

6.1.8 Replacement method

Replacement by excavation

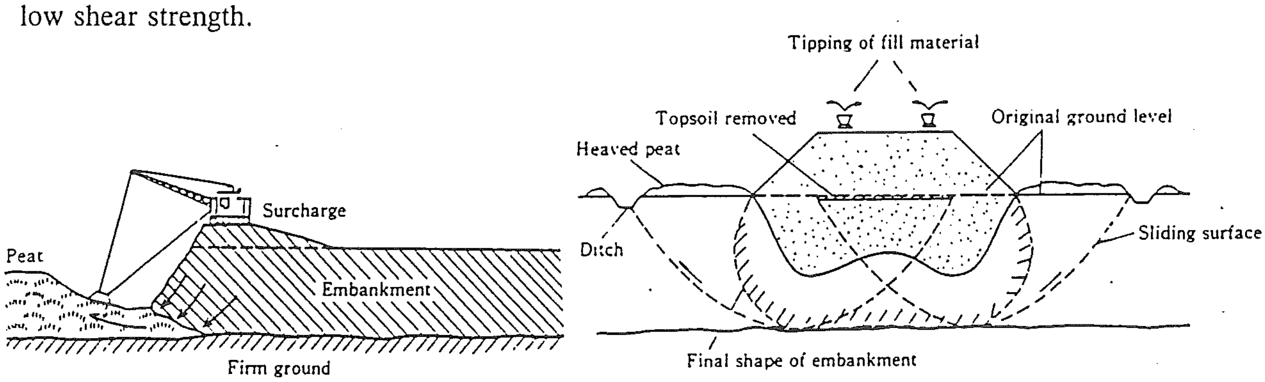
There are several problems with the excavation and replacement method: one is that small pockets of peat may remain unexcavated, as excavation is usually below the groundwater level and this makes it difficult for the operator of the excavation equipment to know when all soft material has been removed. When a decision to excavate completely has been made, an extensive combined boring and sounding programme must be made over the entire length of construction. Very low shear strength peat may cause the side slopes of excavations to fail, and result in significant increases in quantities excavated. Down to 1.5 m, 1 to 1 slopes are frequently specified.

Below 2.5 m, side slopes vary considerably and may be as flat as 4 to 1. The excavated peat should be removed as quickly as possible. If the excavated peat is pushed past the edge of the excavation, the weight of this material may cause it to slide back into the excavated area.

In peat deposits over 3 m, partial excavation is often used for economic reasons, and severe settlement can then be expected. The weight of the replaced fill is usually greater than the weight of the excavated peat, and this contributes to increases in settlement. If the physical characteristics of the peat is not uniform, differential settlement may occur after the completion of the embankment. With partial excavation, construction of the final pavement should be delayed as much as practical to let settlement take place.

Displacement by embankment

This method involves the displacement of soft deposits by the gravity of fill, and is used on many projects particularly in the United States. Figure 6.12, shows forced displacement longitudinally in a) and transversely in b), by placing extra fill on the slope and so displacing the peat at the bottom of embankments. This method is most successful where the peat is 3-6 m deep and has low shear strength.



a) The 'End Tipping' Method of Gravity Displacement

b) The Symmetrical Method of Gravity Displacement

Figure 6.12 Gravity displacement.

Displacement by blasting
This includes two techniques: underfill blasting and toe shooting.

In underfill blasting, fill is first placed on the peat, and then explosives are placed at or near the bottom of the peat.
The location and amount of explosives needed will depend on the depth of the peat, its shear strength, and the height and width of the fill. The explosion displaces the peat under the fill and creates a cavity beneath the embankment, causing it to settle rapidly. A schematic plan of blasting is shown in figure 6.13a).
This method promotes gravity displacement and has been successful in 6 m deep peats.

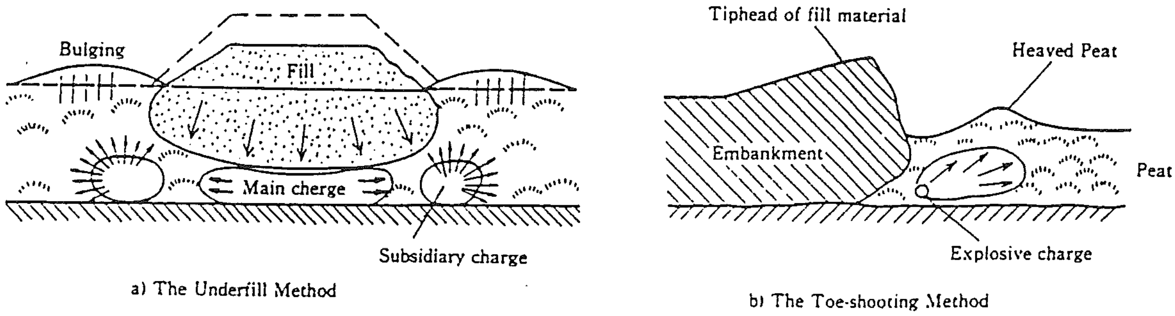


Figure 6.13 Displacement by blasting.

The major problem with blasting displacement, like with gravity is that not all peat is displaced and that some differential settlement can be expected. Reportedly the blasting displacement is less accurate than displacement by gravity, but the cost of blasting displacement is 25 % lower than by mechanical excavation.

6.2 References

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7 Site investigation and project management

To be added

7.1 References

8 Field investigation and project management

8.1 Stability control during construction

In the Netherlands, most methods use piezometers to monitor the stability of fills. For the sake of completeness this section discusses three methods based on deformation measurements. These stability control methods are used for some fills and excavations in addition to conventional methods using piezometers. Common stability control methods are given below.

1) **The method using settlement plates.** This method is based on the fact that the settlement rate increases significantly at a 'critical' level. Due to consolidation and creep effects, interpretation of these measurements is difficult. The disadvantage of this method is that it is only possible to establish afterwards that excessive persistent displacement occurred while such displacement should have in fact been prevented. With precise monitoring, it is however possible to establish the onset of deformations; on the basis of this, work can be stopped. When, and above all at what rate, filling can thereafter be continued, cannot be determined.

2) **The method using pegs.** This method is based on measurement of the horizontal ground displacement and has the same difficulties as monitoring using settlement plates. Both methods can be used to supplement each other.

3) **The method using inclinometers.** This method allows the detection of horizontal displacement below ground level. In addition to the high costs associated with this type of measurement, interpretation of these measurements can cause problems.

If piezometers are used, two approaches are possible. These approaches are discussed below.

1. The excess pore water pressures measured are used in slope stability analyses, from which an equilibrium factor can be determined (Method 1).

2. The excess pore water pressures measured are checked using previously calculated maximum permitted excess pore water pressures. This method will be referred to as the 'total and effective stress path method' (Method 2).

The two methods are explained in more detail below.

Method 1: The method using slope stability analyses

Slope stability analyses made during the construction phase can be carried out both on the basis of either total stresses using undrained shear strengths, or on effective stress basis using effective strength parameters and pore water pressures. As consultancy practice in the Netherlands makes almost exclusive use of the effective stress approach, the first calculation method is not discussed further.
Calculations based on effective stress and strength are similar in nature to those carried out in the design phase, except that measured excess pore water pressures are now included in the calculation. This can be done in two ways:

1. **The method using adjustment percentages.** In this method, the initial effective stress on the lower edge of the slices in the Bishop calculation is supplemented with an additional effective stress. This additional effective stress is determined by the weight of each fill layer multiplied by the adjustment percentage (degree of consolidation) relevant to the layer and derived from measurements. Instead of the adjustment percentage per layer, a global adjustment percentage is often used for all fill layers applied at that time. Sometimes isochrones are compiled on the basis of the measurements; these are lines of equal pore water pressure. The introduction of isochrones in the calculation gives a more realistic picture. As the stress distribution is neglected in this method, it is implicitly and erroneously assumed that the effective stress outside the toe of the slope remains equal to the original effective stress.

2. **The method using a water pressure line.** A water pressure line can be used to introduce excess pore water pressures outside the toe of the slope. In the Bishop analysis however, no increase in ground pressure outside the toe of the slope is assumed. With the introduction of pore water pressure alone, this no longer complies with the most important law of soil mechanics $\sigma = \sigma' + u$.

Although the method using water pressure lines is attractive on first sight (the same calculations are used as in the design phase), there are a number of difficulties, namely:
- As the analytical methods available for calculation of total external stability are not subject to stress distribution, there are fundamental inaccuracies in the description of the construction stability. The pore water pressures measured are in fact influenced by the stress distribution effect and the manner in which the measurement values must be interpreted with this method of monitoring and introduced in the calculations, cannot be given definitively.
- The method is labour-intensive and not flexible, as a stability calculation for each performance stage, is carried out.
- If, during construction, it is decided that the filling programme will be changed, new calculations must be made. The time required can cause an unacceptable delay to construction activities.

Method 2: The method of 'Total and effective stress paths'.
The method of total and effective stress paths [ref. 6.1] is based on the fact that a soil mass does not fail suddenly over the entire slip surface when shear occurs. For example, as a result of widening a road embankment, a plastic zone develops below and next to the road, within which the shear forces occurring lie on the failure envelope: under the circumstances, no further shear stress can be absorbed.

The above shows that the term 'plastic' does not mean that no shear forces can be absorbed. The shear forces present and absorbed may even be very high, depending on the normal stress in the grain skeleton. The term 'plastic' indicates merely that the maximum absorption capacity has been achieved. As a result of deformation processes in this zone,. extra pore water pressures are generated which mean that the normal pressures in the grain skeleton, i.e. the effective stresses, decrease. The result is a lower level of ultimate shear stresses, which in turn leads to a further increase in deformation etc.

This effect, in combination with the continuing increase in load, means that the plastic zone increases steadily in extent. It is clear that the plastic deformation begins somewhere below the fill and spreads in the direction of the active and the passive pressure. It is known that, in the majority of cases, the active part is the first to fail completely. Towards the end of the filling activities, only part of the passive zone lying outside the toe of the embankment, has not deformed plastically.

The method of 'Total and effective stress paths' makes use of the fact that the plasticity front progresses from beneath the fill in the direction of the passive zone. If the approach and movement of this front could be measured, it is possible not only to check calculations but also prevent loss of stability. The low stability factors which often occur in practice during filling are probably the cause of the plasticity front often passing the toe of the slope at the end of the filling activities. This means that, in most cases, a high percentage of the plastic zone arc length required for stability loss has been developed. The zone required for stability monitoring usually lies outside the toe line of the slope.

In principle the following can be used in interpreting pore water pressures measurements:
- A plastic ground model (failure envelope and flow surface);
- A stress distribution model (magnitude and direction of principal stresses);
- An induction dissipation model (causes of pore water pressure changes).

The method has the advantage that it takes into account the volume changes resulting from pore water pressure induction under shear stress (see figure 8.1). From the time at which the ground changes from slightly overconsolidated to normally consolidated state, the shear stresses also play a role in the induction process. The transition from overconsolidated to a normally consolidated state takes place by an increase in load such as filling, and the consolidation process which occurs after the increase in load. Under further load increases, the pore water pressure increase is also related to the increase in shear stress.

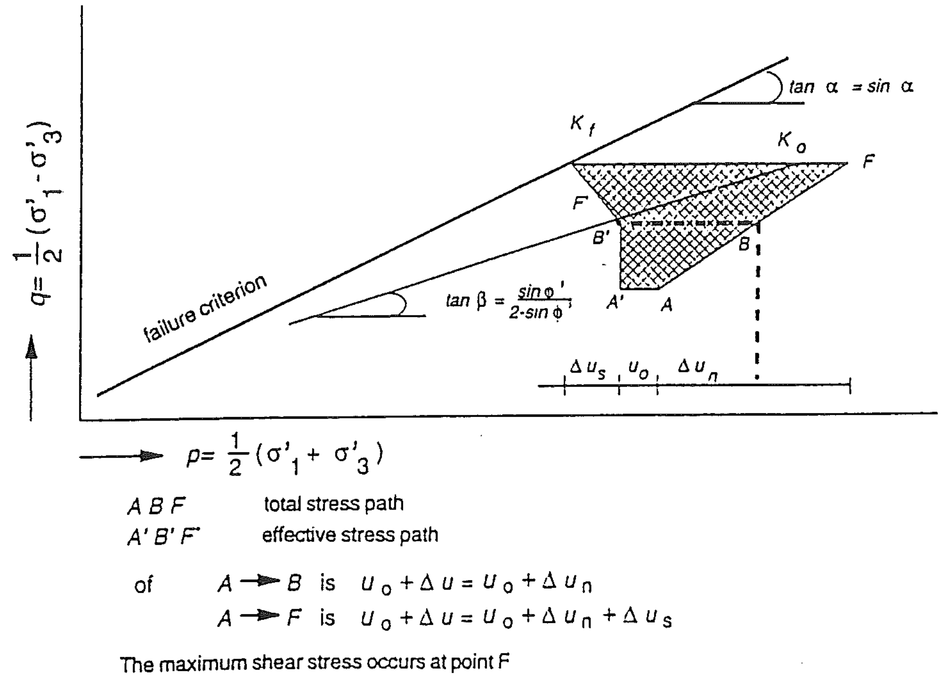

Figure 8.1 Developments in pore water pressures.

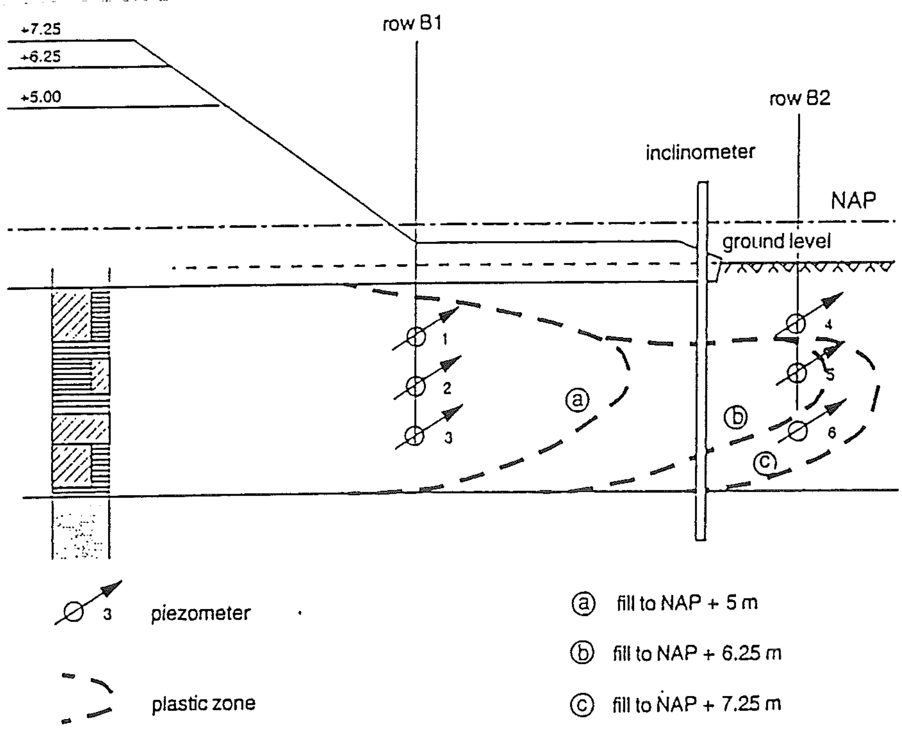

Figure 8.2 Progress of the plasticity front under the toe of the fill.

The excess pore water pressure which occurs due to stress redistribution in the subsoil at the time the actual local failure occurs is not taken into account in the model. However, knowledge of this redistribution acquired during the past years shows that the effects can be interpreted, for example:

- At the time when a much greater increase in piezometric pressure is found than was calculated according to the maximum expected line, it can be inferred that the maximum shear strength elsewhere in the soil mass has been mobilised and therefore all loads applied are being transferred to that part of the soil mass near the piezometer concerned;
- Reaching the maximum permitted line means that the soil at the piezometer location has failed; this need not mean however that the entire soil mass has collapsed.

Stability monitoring using the modified method has consequences for the location of piezometers. These must now be placed such that, due to the pore water pressure measurements, the presence of 'local failures' in a sufficiently large part of the area outside the line of the embankment toe can be established.

This situation can be achieved by installing a number of piezometers placed in vertical next to each other in a cross section (figure 8.2). A minimum arrangement would be: a number of piezometers in a vertical in or near the toe line and a number of piezometers in a vertical halfway between the toe and the exit point of the critical slip surface calculated using an analytical method. Figure 8.2 shows how, on the basis of measurement results from a number of piezometers, it can be concluded that a plastic zone is extending beneath an embankment fill.

8.2 Corrective measures

Deviations in the behaviour pattern described during the design phase may be found during construction monitoring. This can affect the magnitude and/or settlement rate, stability and affect the surroundings.

8.2.1 Settlement

If the settlement behaviour - the final settlement appears to deviate from the prediction and/or the rate of settlement or the consolidation process is different from that expected - then this can have be caused by various mechanisms.

If the prognosis based on measurements shows that the final settlement will be less than expected, this does not generally pose a problem. Any filling required can be reduced on completion. The result is that less fill will be required, meaning less work. Also if the settlement prognosis is good but settlement occurs more quickly than expected, in general less temporary extra height of fill is required; the overall quantity of filling material needed then largely corresponds.

If site measurements indicate a greater settlement than predicted, the consequences can be more extensive. For dykes, the fill height must be higher and the quantity of fill required increases. Any consequences for stability must also be studied. For roads, the problems can be even greater, since excessive residual settlement on completion are usually undesirable. In theory, this can be compensated for by the application of extra load (surcharge) for some time; however, experience has shown that this is rarely a feasible option. The required surcharge load is so high that this usually causes financial and technical problems. Accelerating settlement by the installation of drains is usually possible and often more promising.

A third method may be excavation of part of the fill already placed and replacement by lighter material. In this way, the expected final settlement and residual settlement can both be reduced.

8.2.2 Stability

If, on the basis of the interpretation of the pore water pressures measured, it is concluded that it is feasible to place the fill more quickly than expected, there is generally no reason not to do this. If however it is concluded that the schedule in the specifications must be delayed, this often causes problems. Where possible therefore the specifications for rate of fill placement should state 'except for unacceptably high pore water pressures' such that no filling can take place if pore pressures have not reduced sufficiently. The specifications should also stipulate that in such a case activities may be suspended or postponed. The possible consequence of exceeding the delivery date can however often not be compensated by such stipulations. Instead of suspending the work, alternative solutions should sometimes be considered. The possibilities must be assessed from case to case. A summary of solutions with greater or lesser consequences are given below.

Stability can be improved by the use of extra berms or the widening of existing berms, the installation of (extra) vertical drainage, extension of the drainage, or fill excavation and replacement by lighter material.

The first method assumes that each increase in weight on the passive side of the slip surface means an improvement in the stability. Despite the fact that the total weight of the berm is converted into pore water pressure, the resulting driving force decreases such that stability improves. As the berm can also cause shear deformations in the passive zone, the positive gain of changing the effective stress can be lost. This is the case in particular for short but high berms which cause relatively high shear stresses.

In general, it is not possible to create wider berms for a project under construction. Therefore it is often more effective to ensure that the shear strength increases by reducing the excess pore water pressures as quickly as possible. In real terms, this means adapting or relocating vertical drains. It is particularly useful to install drains to the passive side of the critical slip surface and where possible at some distance outside the toe of the slope.

If sand is used as fill and large settlements occur, part of the sand fill mass may have become saturated. This can be confirmed by monitoring a standpipe placed in the fill. By draining the lowest part of the sand fill layer, weight is removed from the active side of the slip surface, so that stability improves. Drainage also increases the effective stress in the subsoil and the flow rate of the vertical drains.

Sometimes it can be useful to use a lighter material for the remaining part of the fill, for example 'flugsand' instead of ordinary sand. Where necessary, part of the fill already placed can be excavated and replaced by a lighter fill.

It goes without saying that all these measures are expensive. Therefore at all times a balance must be achieved with regard to the consequences of delay or too late completion of the work.

Although stability monitoring is primarily intended to prevent stability loss, intensive monitoring cannot totally exclude slides. If the geotechnical expert is faced with imminent slides on a construction project, there are no clear recommendations for future action; they differ in each situation.
One important point here is the question of whether the slide is taking place in a site in the open or in a built-up area. In the latter case in particular there are many problems which are totally or completely unconnected with soil mechanics. These may include the safety of residents, the safety of passing traffic, access from and to adjacent buildings, and maintenance of utilities. The technical measures which can or must be taken are also very dependent on local conditions. See [ref. 8.2] for an illustration of how serious sliding was treated in a practical case.

Finally, some general comments on stability are given below.

It is occasionally decided that the first phase of fill placement should be carried out quickly. The aim being to achieve a high consolidation rate as quickly as possible in a stage in which there is no risk of stability loss. The absence of high driving forces does not however mean that no plastic zones occur.

Even with sufficient external stability, if the failure limit is exceeded locally, the friction properties in each case should be considerably reduced for the remainder of the construction period. Regular filling with small quantities is thus preferable to placing large amounts of fill in one stage.

The phasing can be optimized in various ways. In addition to phasing as a function of time, the filling strategy or sequence is important. In the latter case, the pore water pressures can be restricted by keeping the shear stress levels as low as possible, for example by filling from the passive zone to the active zone, i.e. by first creating the berm.

8.3 Effect on surroundings

If the measurement results suggest that the expectations of the design phase with regard to possible damage will be exceeded, this generally represents a major problem. The interests of third parties, for example land and house owners, are always central. Sometimes it may be better to accept greater damage, including financial consequences such as compensation. In the most serious cases, it may be necessary to evacuate the area and demolish houses. In general however possibilities of limiting the problems by suitable measures are first considered. Nonetheless, measures which are possible in principle often entail such high costs that they exceed the economic value of the objects concerned. For the sake of completeness, some possibilities are given below.

For structures with spread foundations, three different measures can be taken:

- Fill already placed is excavated in full or in part and replaced by lightweight fill, for example 'flugsand'.
- A retaining wall is constructed between the fill and the structure to be protected. This wall can often be a relatively flexible since the intention is not to retain soil or to resist horizontal deformation but merely to counter vertical deformations behind the retaining wall. 'Relatively flexible' means that the sheet wall profile when at a certain length, still requires to be 'heavy' in view of the driveability required. It must be remembered that the wall should be installed in a dense sand layer, so that the negative friction can be eliminated. In this solution, it must be ensured that the wall does not transfer excessive horizontal forces to the foundations of the structure via a rigid surface layer.
- Piled foundations are installed. Here a stiff network of girders or a reinforced concrete slab is placed on piles below the spread foundations; the types of pile required would be cast-in-situ.

For structures on piled foundations, the following measures are possible:

- Full or partial excavation of fill material and replacement with lighter material;
- The installation of new foundations beneath the structure such that these can withstand the stresses expected;
- The installation in the ground of a rigid structure between the project to be carried out and the structure.

If such rigorous measures are taken, it must be certain that the deformations which have already occurred are acceptable for the structure without further measures.

8.4 References

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9 Ecological impact of civil engineering activities

To be added

9.1 References