MANUAL

GEOTECHNICAL AND FOUNDATION ENGINEERING

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DESIGN AND ENGINEERING PRACTICE

USED BY

COMPANIES OF THE ROYAL DUTCH/SHELL GROUP

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The information set forth in these publications is provided to users for their consideration and decision to implement. This is of particular importance where DEPs may not cover every requirement or diversity of condition at each locality. The system of DEPs is expected to be sufficiently flexible to allow individual operating companies to adapt the information set forth in DEPs to their own environment and requirements.

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All administrative queries should be directed to the DEP Administrator in SIOP.

NOTE: In addition to DEP publications there are Standard Specifications and Draft DEPs for Development (DDDs). DDDs generally introduce new procedures or techniques that will probably need updating as further experience develops during their use. The above requirements for distribution and use of DEPs are also applicable to Standard Specifications and DDDs. Standard Specifications and DDDs will gradually be replaced by DEPs.

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APPENDICES

1. INTRODUCTION

This manual deals with the geotechnical design and engineering problems relating to the foundation of structures and to the stability and deformation of excavations, embankments and (earth) retaining structures.

Civil works such as roads, underground piping, etc. are not discussed here specifically. However, the design and engineering of such works should be performed in corresponding fashion.

The present manual is intended for use in oil refineries, chemical plants, gas plants, in marketing installations, in onshore exploration and production facilities and new ventures.

It is to be used in conjunction with other DEP publications covering the specific requirements of various types of civil engineering structures, such as concrete and steel structures, control buildings, stacks, drainage, etc.

This manual is supplementary to the following DEPs:

DEP 34.11.00.10-Gen.	Site Investigations
DEP 34.11.00.11-Gen.	Site preparation and earthworks
DEP 34.13.20.31-Gen.	Roads, paving, surfacing, slope protection and fencing
DEP 34.19.20.31-Gen.	Reinforced concrete foundations and structures

Unless otherwise authorized by SIPM, the distribution of this manual is confined to companies forming part of or managed by the Royal Dutch/Shell Group and, in consultation with the principal, to dedicated contractors nominated by those Companies.

As a rule the requirements of this manual shall be observed. However national and/or local regulations may exist under which certain requirements are more stringent.
The contractor shall determine by careful scrutiny which of the requirements are more stringent. In all cases, the contractor shall inform the principal of any deviation from the requirements of this manual which is considered to be necessary in order to comply with the national or local regulations.

The specifications given in this manual are based on a generally accepted approach to the geotechnical problems that are encountered in the design and implementation of foundations, jetties, earthworks, retaining structures, etc.

On the basis of geotechnical expertise and local experience, the contractor may propose deviations from the requirements laid down in this manual. Any deviation will be subject to approval by the principal.

Some methods used by SIPM to evaluate designs and proposals from a contractor are set out in the appendices to this manual.

All publications referred to in this manual are listed in (10.).

Where cross references are made, the number of the section or sub-section referred to is shown in brackets.

2. DEFINITIONS

2.1 GENERAL DEFINITIONS

For the purpose of this manual, the following definitions shall hold:

SHALL and **SHOULD** - the word 'shall' is to be understood as mandatory and the word 'should' as strongly recommended to comply with the requirements of this manual.

The **PRINCIPAL** * is the party which initiates the project and ultimately pays for its design and construction.
The principal will generally specify the technical requirements.
The principal may also include an agent or consultant, authorized to act for the principal.

The **CONTRACTOR** is the party which carries out all or part of the design, engineering, procurement, construction and commissioning for the project.
The principal may sometimes undertake all or part of the duties of the contractor.

* For Group operating companies having a service agreement with SIPM or SICM, the term principal shall be taken as referring to SIPM - MFE/3.

2.2 TECHNICAL DEFINITIONS

For the purpose of this specification, the following technical definitions shall hold:

The FOUNDATION is the part of the structure in direct contact with, and transmitting loads to, the ground.

The SUPERSTRUCTURE is the total structure excluding the foundation or pile considered.

The ULTIMATE BEARING CAPACITY or ULTIMATE TENSION CAPACITY for a particular foundation or pile is the load at which the maximum resistance of the ground is mobilized.

The DESIGN LOAD is the maximum load to be transferred by the foundation or pile, according to the design of its superstructure, see (3.4).

The WORKING LOAD follows from one of the next three definitions:

1) In the case of vertical compression load, the working load equals the design load plus the (submerged) own weight of the foundation or pile.
2) In the case of vertical tension load, the working load equals the design load minus the (submerged) own weight of the foundation or pile.
3) In the case of horizontal loading, the working load equals the design load.

The ALLOWABLE LOAD for a particular foundation or pile is the maximum allowable working load taking into account the specified safety factor with regard to the ultimate bearing capacity (or tension capacity), the amount and type of deformation expected and the amount and type of deformation that can be tolerated by the superstructure.

ROCK and CEMENTED SOILS mainly consist of natural aggregate of mineral particles connected by strong and 'permanent' cohesive forces.

SOIL mainly consists of sediments or other accumulations of solid particles produced by weathering of rock, and which may or may not contain organic matter.

CPT (Cone Penetration Test) and SPT (Standard Penetration Test) are site investigation techniques (refer to DEP 34.11.00.10-Gen.)

CONSOLIDATION is the process in which the soil volume decreases (causing settlement) due to draining of pore water and increasing effective soil stresses.

LIQUEFACTION is the loss of shear strength due to increased pore water pressure in cohesionless soil.

EQUIVALENT PILE DIAMETER (Deq) is defined by the following equation:

$$D_{eq} = 1.13 \sqrt{A}$$

in which A = cross-section area of the pile base.

3. BASIC APPROACH TO GEOTECHNICAL ENGINEERING

3.1 GEOTECHNICAL TERMS AND DEFINITIONS

Appendix I defines geotechnical terms and symbols used in this manual and gives briefly some basic information about geotechnics. The design of foundations is primarily concerned with ensuring that the deformations of the superstructure remain within the limits that can be tolerated by the structure without affecting its proper functioning.
The design shall take into account the character of the load of the structure (e.g. static or dynamic).

As soil and rock are building materials formed by nature, their properties vary from one location to another. As a consequence, the behaviour of soil or rock cannot be calculated in exact figures. For these reasons the designing contractor should seek the advice of an experienced, specialized soil consultant in every case.

3.2 SITE INVESTIGATION

For each foundation or earthwork a site investigation shall be carried out to determine the character and variability of the soil strata, the groundwater pressures in the various strata and other factors interacting with the proposed work or structure. Reference is made to DEP 34.11.00.10-Gen., which contains guidelines for the scope of site investigations and related reports.
A chemical analysis of soil and ground water shall be carried out to establish whether the soil and groundwater conditions may affect, for example, the proposed foundation or drainage system.
Attention shall be given to possible changes in the soil and groundwater conditions (qualitatively and quantitatively) caused by the engineering works, both during construction and when in service.

3.3 SOIL AND ROCK CLASSIFICATION

Soils and rocks should be classified in accordance with British Standard BS 5930 Section 8 (1981 issue).

3.4 LOADS ON FOUNDATIONS

3.4.1 General

The loads on the foundations shall be calculated according the principles as outlined in DEP 34.00.01.30-Gen. 'Minimum Requirements for Structural Design and Engineering'.
For the design loads of marine and waterfront structures, reference is made to (3.6).

3.4.2 Dynamic loads

Where relevant, dynamic loads on the foundation (e.g. from compressors or reciprocal machines) shall be taken into account for the design of the foundation. The natural frequency of the foundation-ground system shall be estimated. See DEP 34.00.01.30-Gen., para. 3.9

3.5 EARTHQUAKES

It shall be investigated whether the area in which the proposed works or structures are located is prone to earthquakes. Existing national or local earthquake codes shall be adhered to. With regard to earthquake-prone areas where no codes exist, the basic design assumptions are subject to approval by the principal.

In any earthquake prone area a special site investigation shall be carried out to analyse the possible ground movements and the liquefaction potential of the soil.

3.6 MARINE AND WATERFRONT STRUCTURES

For marine and waterfront structures, the design load as referred to in (2.2) shall be defined by the designing contractor. The required minimum safety factors with respect to stability loss of foundations, piles, retaining structures, etc. shall be selected in relation to the definition of the design load and may differ from the values mentioned in this manual. Design loads and safety factors are subject to approval by the principal. The principal is recommended to consult SIPM. Reference is made to the German Code EAU 1985.

3.7 DESIGN COMPUTATIONS AND REPORT

A design report shall be prepared containing the following information as a minimum (refer to DEP 34.11.00.10-Gen. 'Site Investigations'):

- the results of site investigations used for the design
- geotechnical advice
- short outline of the design methods applied
- all calculation results

All geotechnical computations shall be made using SI units. The calculation methods are not prescribed by this manual. However, the methods used by SIPM to evaluate the design are set out in the appendices.

3.8 INTERACTION WITH ADJACENT STRUCTURES

Implementation of a new structure in the vicinity of an existing structure may have adverse effects on the existing structure and/or on the new structure. In the design and during construction all possible interactions shall be analysed.

Such checks shall include but not be limited to:

- Excavations or fills which may cause horizontal and vertical soil deformations that may affect adjacent piles or shallow foundations.
- Dewatering which may cause settlements of spread foundations or cause downdrag loads on piled foundations.
- Loading of a new foundation adjacent to an existing foundation which may cause tilting of either one or both foundations.
- Vibrations due to pile driving that may cause settlements of adjacent structures or damage to sensitive equipment in adjacent structures.

Where risk of damage to adjacent structures exists, the principal shall be consulted before the implementation is started.

3.9 MONITORING OF FOUNDATIONS

Especially foundations on soft soil and weak rock should be evaluated by monitoring of settlement at intervals of:

- 2 to 6 weeks during construction of the superstructure,
- 6 months during the first two years after construction/water testing,
- 3 to 5 years in the period thereafter.

For horizontally loaded foundations, the horizontal movements should also be monitored.

The monitoring marks etc. at the structure or foundation shall be installed by the contractor. During his construction period, the contractor shall be responsible for carrying out the monitoring.

4. SPREAD FOUNDATIONS ON SOIL

4.1 FOUNDATION DEPTH

Foundation depths shall be selected with reference to the following requirements:

- an adequate bearing stratum shall be reached under the total area of the foundation, and
- the foundation level shall be below the zone affected by erosion, frost and other influences (seasonal shrinkage and swelling, tree roots, etc.).

4.2 ULTIMATE BEARING CAPACITY

The ultimate bearing capacity shall be calculated taking into account the distribution of stresses in the subsoil. The preferred calculation method is outlined in Appendix II.

4.3 SETTLEMENT BEHAVIOUR

The settlement behaviour of the foundation shall be calculated from the stress distribution in the subsoil, and be based on compression parameters of the substrata derived from laboratory tests.
Both the total and differential settlement as well as the increase of the settlements in time shall be calculated. See (7.3).
Where relevant, effects of vibrations on long-term settlement (tilting) shall be taken into account.

4.4 ALLOWABLE LOAD

In general, the allowable load on a foundation is determined by the (differential and absolute) settlements that can be accepted. The settlements shall be calculated as a function of the loading and shall be compared with the absolute and differential settlements that can be tolerated by the superstructure.
A minimum safety factor of 2.0 (1.5 for less important structures) shall be adopted with respect to the ultimate bearing capacity.

4.5 DYNAMIC LOADS AND EARTHQUAKES

Reference is made to (3.4.2) and (3.5).

4.6 INTERACTION WITH ADJACENT STRUCTURES

Reference is made to (3.8).

4.7 IMPLEMENTATION AND SUPERVISION

Prior to the construction of spread foundations, the contractor shall check whether the expected bearing layer is present over the total area of the foundation.

In those cases where the soil of the bearing layer may be susceptible to weathering, the time between excavation and construction of the foundation shall be as short as possible. Before excavation is started, the contractor shall submit a time schedule showing the period of exposure of the bearing layer.
Backfill shall not be started before approval by the supervisor nominated by the principal.

If any fill or backfill has been applied underneath a foundation, its compaction shall be checked.
Refer to DEP 34.11.00.11-Gen., para. 4.4.

As a means of Quality Assurance the Contractor shall provide an Implementation Log Book and shall record at least:

- depth, area and volume of each excavation
- encountered soil layers
- encountered groundwater level

- data of excavation
- date and results of inspection of the bearing layer
- date of (back) filling
- number of fill layers, compaction procedure
- results of compaction tests
- date of pouring the blinding layer
- date of pouring the foundation

4.8 TANK FOUNDATIONS

Generally vertical (atmospheric) storage tanks are founded on a tank pad.

Main long-term functions of a tank pad are:

- to provide a surface with sufficient bearing capacity and to spread and transfer the tank load to the subsoil,
- to raise the tank bottom to a sufficient height above ground water.

Design calculations shall be in accordance with (7.1.5). The design and construction shall be in accordance with DEP 34.11.00.11-Gen.

Before taking into operation, each tank (placed on its foundation) shall be water-tested in accordance with DEP 34.11.00.11-Gen., para. 6.4 and Appendix 6. During the water test the tank settlements shall be monitored. If the maximum settlement of the tank shell during the water test is more than 50 mm, the tank settlements should also be monitored during operation. See (3.9).

5. PILE FOUNDATIONS IN SOIL

5.1 GENERAL

The main function of bearing piles is to transfer the load on a foundation to deeper (soil) strata which are capable of carrying the working load in such a way that:

- an adequate safety factor against loss of stability is provided, and
- the foundation settlement/deformation is less than the absolute and differential settlements/deformations that are allowable for the superstructure and the equipment supported.

The design and testing of piles shall take into account both short-term and long-term behaviour of the pile foundation.

5.2 TYPE OF SOIL INVESTIGATION

The design of pile foundations shall preferably be based on the results of static cone penetration tests (CPT) and some reference borings. Refer to DEP 34.11.00.10-Gen.

5.3 TYPES OF PILES

5.3.1 Prefabricated or cast-in-situ

Driven prefabricated piles shall be applied wherever possible.

This is because the integrity of bored, augered and also of driven cast-in-situ piles depends strongly on the soil conditions and on the workmanship and (local) experience of the piling contractor (even more than installing prefabricated piles does). Moreover the quality of cast-in-situ piles is hard to supervise. Because of these difficulties, the application of cast-in-situ piles requires:

- higher safety factors in the design calculations,
- continuous supervision of the construction work by experienced staff,
- a relatively large number of load tests.

Moreover a foundation on drilled or augered piles is generally less stiff with regard to settlement than a foundation on driven piles.

The application of cast-in-situ piles (e.g. augered piles, cast-in-situ driven piles, bored piles) shall be subject to approval by the principal, who is recommended to consult SIPM.

<u>The guidelines in this manual apply only to prefabricated driven piles.</u>

5.3.2 Selection of prefabricated piles

The piles should have a base dimension equal to the shaft dimension of the pile or at most 10 mm larger, with the exception of piles especially designed as 'piles with enlarged base', see (5.5.5).
The use of concrete piles or steel piles is preferred. Pile material shall be selected in accordance with (5.4).

5.4 PILE DESIGN AND MATERIAL

5.4.1 General

The structural design of piles shall be based on British Standard CP 2004.

The pile material shall be selected considering:

- type of loading (tension, compression, lateral), see (5.5), (5.6), (5.7) and (5.8)
- driveability, see (5.4.2), including the effects of raking if relevant
- corrosiveness of soil and ground water, see (5.4.3) and (5.4.5)
- possibility of tension stresses in the pile due to heave during driving of adjacent piles, see (5.4.4)
- possibility of bending stresses in the pile due to horizontal soil deformations, e.g. near (future) adjacent slopes

5.4.2 Driveability

Prior to the installation of piles the driveability shall be analysed, preferably via stress (wave) analysis. Risks of pile damage or misalignment due for example to boulders or sloping surfaces shall be assessed. If very heavy driving is expected, open-end or H-type steel piles should be considered. If open-end piles are selected, the stress (wave) analysis should consider possible plugging of the soil inside the pile during driving.
Preboring or jetting may be considered as well, see (5.10.6).

5.4.3 Corrosion

In some cases piles may become exposed to an agressive environment due to soil or groundwater condition, for example due to high contents of acid, sulphates or chlorides, or due to soil oxidation etc.

The following protective measures (separately or in combination) should then be considered:

- prestressing of concrete piles,
- application of special cement, such as Portland-Blast Furnace Cement (BS 146) or Sulphate Resistant Cement (BS 4027),
- application of steel piles,
- coating of (the relevant part of) the pile surface,
- sacrificial wall thickness or casing,
- cathodic protection.

5.4.4 Heave

If a large number of piles have to be driven on a relatively small area, in combination with the occurrence of thick layers of saturated stiff cohesive soil, heave of the piles already driven may occur due to driving of adjacent piles.
If relevant, the pile design shall consider the tension forces that can result from the heave.
To reduce heave forces, preboring may be considered, see (5.10.6).

5.4.5 Timber piles

No timber piles shall be applied to carry tension loads. If timber piles are applied, the head of such a pile shall be cut 0.50 m below the lowest groundwater level.
Timber piles shall not be applied in an agressive environment.

5.4.6 Precast concrete piles

5.4.6.1 General

The concrete mix shall be designed in accordance with British Standard BS 8110.
The water-cement ratio shall be at most 0.45. Minimum concrete cover shall be 50 mm.
Quality control procedures shall be in accordance with DEP 34.19.20.31-Gen. 'Reinforced concrete foundations and structures'.

The design shall take into account all loading conditions, including pile driving, handling/pitching and transport. For both latter conditions an impact factor of 2.0 (working stress design) or a partial safety factor of 2.8 (limit state design) shall be taken into account.

5.4.6.2 Precast Reinforced Concrete piles

The steel reinforcement shall be designed in such a way that the crack width of the concrete will remain less than 0.2 mm under all loading conditions. The total minimum percentage of reinforcement shall be 1.5%.

The reinforcement of tension piles shall be based on the assumption that the tensile strength of the concrete is nil.

The slenderness of the pile (defined as pile length divided by the smallest width of the pile) shall be at most 40 for grade 30 concrete, or at most 60 if the concrete quality is grade 40 or higher.

5.4.6.3 Prestressed concrete piles

The design shall be subject to approval by the principal. The design shall be such that the crack width of the concrete will remain less than 0.05 mm (so-called 'crackfree') under all loading conditions. Prestressing wire shall be of high tensile steel in accordance with BS 5896.

5.5 COMPRESSION PILES

5.5.1 End bearing and positive friction

The bearing capacity of piles under compressive loads is a combination of end bearing and so-called positive friction along the pile shaft. The ultimate end bearing and the ultimate positive friction shall be calculated. If compressible layers cover the layer in which the pile tip bears, the positive friction in the layers above this bearing layer shall be disregarded.
The calculations should be based on the results of CPT's if available. Appendix III outlines the methods used by SIPM to evaluate bearing capacity calculations.
In the design of open end or H-type piles it shall be predicted whether plugging will occur.

5.5.2 Settlements

The load-settlement behaviour of a (single) pile shall be predicted, preferably based on local experience from pile load tests. If compressible layers occur below the pile base level, consolidation of these layers due to the total weight of the structure shall also be taken into account.
If open-end piles are applied, deformation of the soil plug inside the pile shall also be considered in the settlement prediction.
Pile group effects shall be taken into account in accordance with (5.5.6).

5.5.3 Negative friction

Where a pile is installed through a stratum which may settle, for example because of consolidation of deeper layers, the maximum downdrag load (called negative friction) on the pile shall be calculated.

The calculation shall take into account:

- ultimate friction between soil and pile shaft over the full depth of the settling layers, and
- maximum weight causing the soil settlement (consolidation).

Appendix IV sets out the method used by SIPM to evaluate negative friction on piles.

5.5.4 Allowable load on a single pile

The allowable compression load under static conditions shall be calculated as the lowest value resulting from the following equations:

$$Q_a = \frac{Q_{pu} + Q_{su} - U_f * N_f}{F_c} \quad \text{and} \quad Q_a = \frac{Q_{pu} + Q_{su}}{0.8F_c} - N_f$$

In which:

Q_a = allowable (compression) load (kN)

Q_{pu} = ultimate end bearing (kN)

Q_{su} = ultimate positive shaft friction (kN)

N_f = ultimate negative shaft friction (kN)

U_f = uncertainty factor, to be taken as 1.1

F_c = safety factor, see (5.5.5)

Incidental loads (e.g. wind loads) do not have to be combined with negative shaft friction.

The maximum compression stress in the pile material shall be calculated from Q_a + N_f.

5.5.5 Safety factors

In the case where the design is based on CPT results a minimum safety factor F_c = 2.0 shall be applied for driven pre-fabricated piles without enlarged base.

If the design is based on other types of site investigation, in accordance with DEP 34.11.00.10-Gen., a minimum safety factor of F_c = 2.5 shall be adopted.

For piles with an enlarged base higher safety factors (subject to approval by the principal) shall be adopted, depending on the pile dimensions. The principal is recommended to consult SIPM.

Based on the number and results of load tests and depending on the type of structure, it might be allowable to reduce the above safety factors, but only after consultation with SIPM.

5.5.6 Pile group effects

For groups of four piles or more with pile spacings less than eight (equivalent) pile diameters, the bearing capacity of the total group shall be checked. Moreover the settlement increase by group effects shall be checked. Possible reduction of the negative friction on the inner piles of a group shall be taken into account.

5.6 TENSION PILES

5.6.1 Individual piles

The ultimate pulling capacity of an individual pile shall be calculated and the pile rise shall be predicted as a function of the load. Appendix V states the methods used by SIPM to evaluate tension capacity calculations.

The working load - i.e. design load minus the (submerged) pile weight - under static conditions shall include a safety factor of at least 3.0 with respect to the ultimate capacity.
Based on the number and results of load tests and depending on the type of structure, it might be allowable to reduce the above safety factor, but only after consultation with the principal who is recommended to consult SIPM.

5.6.2 Pile group effects

Possible reduction of the friction per pile owing to group effects shall be analysed. The (submerged) weight of the soil mass causing the friction along the piles shall be at least 1.2 times the total upward working load on the pile group. Refer to Appendix V.

5.7 PILES UNDER CYCLIC AXIAL LOADS

When pile load alternations occur frequently, for example due to wind, wave or mooring forces, the load is called cyclic. The maximum load shall never be more than the allowable static load.

As a minimum requirement it shall be analysed whether the cyclic character of the load requires reducing of the allowable maximum load when both of the following two conditions apply:

- maximum (peak) load is more than 75% of the allowable static load
- peak to peak load cycle is more than 50% of the maximum load

The analyses required shall take into account:

- (increased) pile displacements
- stiffness of the pile
- stiffness of the various soil layers
- residual friction in the soils (after degradation)
- effects of lateral pile loads, if applicable

5.8 LATERALLY LOADED PILES

The design of laterally loaded piles shall take into account:

- stiffness of the pile
- stiffness of the soil
- passive (yield) earth pressure and residual soil strength
- bending stresses in the pile
- pile deformations
- the pile head displacement and rotation that can be tolerated by the superstructure
- minimum safety factor of 2.0 with regard to stability loss (overturning). For dolphins, the stability factor may be reduced to 1.5 or less depending on the definition of the design load, see (3.6)
- effects of cyclic loading
- interaction with axial loads.

The method used by SIPM to evaluate the design of laterally loaded piles is indicated in Appendix VI. For marine structures, the pile design shall be in accordance with the API-RP-2A code.

For piles spaced less than eight times the pile width apart, group effects shall be considered.

5.9 RAKING PILES

Raking piles may be applied when large horizontal loads occur in combination with top layers of soft soil.
In the design calculations, a possible deviation of 20:1 from the theoretical rake shall be taken into account.
When raking piles penetrate through settling strata, the bending moments caused shall be taken into account in the design. Effects of the rake on the stresses during pile driving shall be considered.

5.10 PERFORMANCE OF PILING WORK

5.10.1 Storage and handling of piles

Prior to the pile driving operation, the storage and handling of piles shall be such that damage to the piles does not occur.
Concrete piles shall only be transported, handled or driven either after they have reached their characteristic strength or at least 28 days after fabrication.

5.10.2 Supervision

The piling work shall be supervised by or on behalf of the principal. The supervision shall not be subcontracted to the piling contractor. No piles shall be installed or tested without witnessing by authorized supervision.

5.10.3 Pile Installation Log Book

As a means of Quality Assurance the Contractor shall supply a Pile Installation Log Book comprising the following information per pile:

- description of equipment, type, size and maximum energy of hammer, stroke of hammer, frequency of blows and if applicable setting of fuel pump
- date of pile manufacture
- date and time of driving
- pile data: length, working load, size and number/location of pile as shown on the drawing
- the number of blows per 250 mm penetration (together with an indication of the actual hammer stroke) over at least the last four (4) metres above design pile tip level
- for each pile nearest to a CPT location, blow counts shall be recorded over the total length of the pile, and these blow counts shall be presented in a graph together with the relevant CPT
- levels with respect to Datum:
  . final elevation of the pile top
  . final elevation of pile tip
  . grade/bottom elevation at the time of pile driving
- sequence of installation of the piles
- replacement of the cushion material in the driving cap
- occurrence of plugging of open end piles
- any special event that occurred during installation

For marine works, the total number of blows should also be recorded.

5.10.4 Driving tolerances

The driving tolerances shall be specified in the design.

Driving of concrete piles shall be done with the application of fixed leaders or other means which will hold the pile firm in position and alignment. No concrete pile shall be pulled or jacked into the required position.

5.10.5 Pile Driving Sequence

In general the first pile shall be driven close to the location of a CPT (if available).
Driving of a large group of piles should start in the centre of the group.
If pile driving has to be carried out on a sloping surface, driving shall start at the higher parts.

5.10.6 Jetting and preboring

Jetting and preboring shall be applied only after approval by the principal.

Jetting and/or preboring are not allowed in those layers from which (according to the design calculations) the pile derives positive shaft friction and/or end bearing. Jetting shall not be applied for piles subjected to lateral loads. For such piles, preboring is allowed only if the applied bore diameter is sufficiently less than the pile width to prevent reduction of the stiffness of the lateral soil support. Special care should be taken with jetting and preboring for raking piles, because the driving direction may be affected.

5.11 INTERACTION WITH ADJACENT STRUCTURES

Reference is made to (3.8).

5.12 INTEGRITY TESTING

If high driving resistances have been met during the driving (of concrete piles) or there are other reasons for suspecting the integrity of the installed piles, checking the pile integrity with sonic testing or equivalent methods should be considered.
The results of the integrity tests can be used as a basis to decide on an additional pile load test scheme.

5.13 LOAD TESTING

5.13.1 Purpose

Purposes of the load tests are:

- to demonstrate that the pile stiffness and strength assumed in the design can actually be achieved, and
- to make a random check on the performance of the piling contractor's work and the piling equipment

5.13.2 Number of tests

For driven prefabricated piles the minimum number of tests that shall be performed is:

Compression piles	:	1% of the piles with a minimum of two tests.
Other types of pile load	:	2% of the piles, with a minimum of three tests

If the piling contractor submits satisfactory load test results on the relevant pile type at a geotechnically equivalent location, subject to the approval of the principal, the number of tests to be performed may be reduced. In such cases the principal is recommended to consult SIPM.
Depending on the design load levels and the type of structure, the principal may decide to restrict the test programme. The principal is recommended to consult SIPM.

In cases where the application of cast-in-situ piles has been approved as described in (5.3.1), the number of load tests and the testing procedure shall be specified by SIPM.

5.13.3 Selection of piles, time and locations of tests

The piles to be tested shall be selected by the principal, see (5.10.2). The first pile installed should be tested. At least two tests should be performed before 50% of the piles have been installed. Each test pile should be located nearest to a spot where site investigation has been carried out, preferably a CPT.

5.13.4 Loading procedure

A stepwise loading procedure shall be carried out in accordance with Table 5.13.4. The pile shall be loaded up to the Test Load Q_t, defined as:

$$Q_t = 1.5\ Q_a + N_f$$

Q_a is the allowable load according to the pile design.

Nf is the negative friction load that may act on compression piles according to the design, see (5.5.3).

For all other types of test load: $N_f = 0$.

For each load step, the load shall be held constant for not less than the period shown in the table and until the rate of pile displacement is less than 0.15 mm per 30 minutes and decreasing.

Load and pile displacements shall be recorded at 2, 5, 10, 20, 30, 45 and 60 minutes after application of each load step, and beyond that time every 30 minutes. Refer to Appendix VII.

Table 5.13.4 Test loading procedure

Load	Minimum holding time
0	10 min.
(N_f)	(1 hour)
0.25 Q_a (+ N_f)	1 hour
0.50 Q_a (+ N_f)	1 hour
0.75 Q_a (+ N_f)	1 hour
1.00 Q_a (+ N_f)	1 hour
0.75 Q_a (+ N_f)	10 min.
0.50 Q_a (+ N_f)	10 min.
0.25 Q_a (+ N_f)	10 min.
(N_f)	(10 min.)
0	1 hour
1.00 Q_a (+ N_f)	6 hours
1.25 Q_a (+ N_f)	1 hour
1.50 Q_a (+ N_f)	6 hours
1.25 Q_a (+ N_f)	10 min.
1.00 Q_a (+ N_f)	10 min.
0.75 Q_a (+ N_f)	10 min.
0.50 Q_a (+ N_f)	10 min.
0.25 Q_a (+ N_f)	10 min.
(N_f)	(10 min.)
0	1 hour

Q_a = allowable load according to the pile design

N_f = negative friction on compression piles, if applicable

5.13.5 Equipment, operators and supervision

All equipment is subject to approval by the principal, see (5.10.2) and (Appendix VII). All measurements shall be carried out by well trained and experienced operators/engineers. During and after the tests, all records shall remain available for inspection by the principal.

5.13.6 Acceptance criteria

In view of long-term pile behaviour, the pile settlement/deflection under the allowable load shall be the main criterion in evaluating the test results. The acceptance criteria for the test results shall be specified in the design, and shall include as a minimum:

- maximum total displacement, and
- maximum permanent displacement after unloading from 1.5 Q_a (+N_f)

A guideline for the test evaluation of small-diameter prefabricated piles is given in Appendix VII.

6. FOUNDATIONS ON ROCK OR CEMENTED SOILS

6.1 GENERAL

For foundations in weak, heavily weathered rock or cemented soil two design analyses shall be made:
- one assuming that the weathered rock mass/cemented soil behaves as a soil, see (5.), and
- the other assuming rock.

The latter approach is dealt with in this chapter. Typical concerns of rock engineering are described in Appendix XI.

By thorough site investigation (refer to DEP 34.11.00.10-Gen.) and geological study, it shall be assured that no unknown discontinuities occur in the rock below the foundation. No foundation shall be constructed on 'live' fractures or major fractures.
Possible effects of earthquakes shall be analysed.

In the design, the top level of the bearing rock shall be defined.
During construction the condition of the encountered rock mass shall be inspected in order to verify the assumption made in the design.
Care shall be taken to assure good contact between the foundation and the (irregular or sloping) surface of the bearing rock.

No structure should be founded partly on sound rock and partly on soil or weak rock.
When rock fill is applied underneath the foundation, the requirements stated in DEP 34.11.00.11-Gen. shall be fulfilled.
When a backfill is required underneath a part of the foundation, care shall be taken that the backfill has a similar stiffness as the rock underneath the other part of the foundation. The design of such a backfill shall be subject to approval by the principal, who is recommended to consult SIPM.

6.2 SPREAD FOUNDATIONS

6.2.1 Foundation depth

Reference is made to (4.1).

6.2.2 Bearing capacity and settlement

The foundation design shall take into account:

- the strength of the rock material
- the geometry of joints, faults and other heterogeneities in the rock mass
- all possible failure modes due to the above conditions.

In determining the allowable load, both absolute and differential settlements shall be considered, including time effects.

A minimum safety factor of 3.0 shall be adopted with respect to the ultimate bearing capacity.

6.2.3 Testing

For heavily loaded or settlement-sensitive structures to be founded on weathered rock or cemented soil, large-scale loading tests should be performed to obtain reliable settlement data. Deformation characteristics of rockmass may also be derived from in-situ tests such as pressure meter tests or plate loading tests.

6.3 PILED FOUNDATIONS

6.3.1 Pile types

Only steel (cased) piles shall be applied:

- driven into the top of the bearing rock

- socketed in the bearing rock

The piles shall be designed in accordance with (5.4). Only socketed piles shall be applied to carry tension loading. Application of other pile types is subject to approval by the principal, who is recommended to consult SIPM.

6.3.2 Driven piles

6.3.2.1 Design

Only the end bearing capacity shall be taken into account. The ultimate end bearing capacity shall be calculated. A minimum safety factor of 3.0 shall be adopted to determine the allowable pile load.
Moreover the design shall fulfil the requirements stated in (5.5.2) to (5.5.4), and (5.5.6), (5.8), (5.9) and (5.11).

Based on the number and the results of load tests and depending on the type of structure, it might be allowable to reduce the above safety factor, but only after consultation of SIPM.

6.3.2.2 Installation and supervision

A driveability study to analyse the risks of pile damage or misalignment during driving, for example due to boulders, hard seams or irregular/sloping surfaces, shall be carried out taking into account pile dimensions and piletip reinforcement. Driveability study via stress wave analyses should be considered.
The piling work shall be performed and supervised in accordance with (5.10), (5.11) and (5.12).

6.3.3 Socketed piles

6.3.3.1 Concrete quality and reinforcement

The contractor shall submit a concrete mix design, based on a minimum cube strength of 25 N/mm^2 (including all additives). A non-shrinking concrete mix shall be applied. The properties of the concrete mix shall be demonstrated by test results, for approval by the principal. Refer to DEP 34.19.20.31-Gen.

Tension piles shall be reinforced over the total length of the pile shaft and socket. Other piles shall have a reinforcement over at least 4 m below cut-off level.

6.3.3.2 Socket length

The socket length below the top of the bearing rock depends on the type of load (e.g. tension or compression) and the required bearing capacity, but should be at least two times the pile base diameter or at least 1.50 metres, whichever is the greater.

6.3.3.3 Bearing (tension) capacity and settlements

The design shall take into account:

- type of load (e.g. static or cyclic, compression or tension)
- ultimate friction at socket-rock and rock-rock interfaces
- end bearing (only for compressive loads)
- stiffness ratio between socket and rock
- brittle behaviour of rock (residual friction)
- pile settlement/pile rise
- a minimum safety factor of 3.0
- pile group effects

Moreover the design shall fulfil the requirements mentioned in (5.5.2) to (5.5.4), (5.5.6), (5.7), (5.8), (5.9) and (5.11).

Based on the number and the results of load tests and depending on the type of superstructure, it might be allowable to reduce the above safety factor, but only after consultation with SIPM.

6.3.3.4 Installation and supervision

Attention shall be paid to harmful effects of water ingress in the drilled hole.

The piling work shall be supervised by or on behalf of the principal in accordance with (5.10.2).

As a means of Quality Assurance the contractor shall provide a Pile Installation Log Book, recording:

- description of equipment, including torque capacity, vertical force, maximum rotation speed, drill bits, drilling mud
- date and time of drilling
- date, time and depth of installation of the casing (if relevant)
- date and time of concreting
- pile data: length, design load, diameter, number/location of pile as shown on the drawing
- levels with respect to Datum:
  - final elevation of pile top
  - final elevation of pile tip
  - grade level
- over at least the full length of the socket, recorded per max. 0.25 m interval:
  - rotation speed
  - torque applied
  - downward force applied
- description of recovered drilling mud
- sequence of installation of piles
- change of drill bit
- any special events occurred during installation

6.3.4 Pile load testing

At least 2% of the piles installed shall be load tested, with a minimum of three piles. See (5.13).
Acceptance criteria for the test results shall be specified in the design.

7. EARTHWORKS

7.1 GENERAL

7.1.1 General references

Reference is made to DEP 34.11.00.11-Gen. 'Site preparation and earthworks' and to British Standard BS 6031.

7.1.2 Site investigation

Prior to the design of an excavation, fill, soil improvement, dewatering, etc. a site investigation shall be carried out in accordance with DEP 34.11.00.10-Gen. Attention shall be paid to the character of the rock or soil and to the groundwater head in each stratum.

7.1.3 Excavations

If in deep layers a groundwater head occurs that is higher than the bottom elevation of an excavation, the stability of the bottom shall be checked. For permanent excavations the bottom stability shall be maintained with a safety factor of 1.2.
Stability of slopes of the excavations shall be checked, see (7.2), taking into account a possible surcharge owing to excavated material stored near the top of the slope.

7.1.4 Fills and embankments

The design criteria for a fill or embankment shall be related to the purpose of the fill. Compaction of the fill shall be specified depending on:

- design requirements, viz. strength (bearing capacity, slope stability), stiffness (settlement) and liquefaction potential (during vibrations, earthquakes or wave impacts)
- construction method
- applied fill material

Refer to DEP 34.11.00.11-Gen.

The contractor shall demonstrate the suitability of the proposed fill material before application.

The stability and settlements of a fill or embankment shall be calculated in accordance with (7.2) and (7.3).

7.1.5 Tank pads (See (4.8) and DEP 34.11.00.11-Gen.)

The design of a tank pad will include but not be limited to the calculation of:

- overall stability (general bearing capacity) of tank, tank pad and subsoil, see (7.2) and (Appendix II)
- the local stability considering the tank shell to be founded on a spread foundation, see (Appendix II), and analysing the slope stability and deformation of the tank pad shoulder, see (7.2)
- overall settlement
- three types of differential settlements:
  . differential settlements along the tank shell
  . differential settlements between tank centre and perimeter
  . overall tilting

  See (7.3).

7.2 SLOPES

7.2.1 Erosion protection

Seepage from a slope shall be prevented.

Permanent slopes shall be protected against erosion by wind or water.

7.2.2 Stability

7.2.2.1 Calculations

The long term and short term stability of a slope shall be analysed. Possible shear reduction due to (excess) pore water pressures shall be considered. Special loads, e.g. surcharges due to traffic or crane loads, shall be taken into account if relevant. Consideration shall be given to the slope deformations. Appendices VIII and XI set out the methods used by SIPM to evaluate stability analyses.

7.2.2.2 Safety factors

The required safety factors shall be based on sound engineering judgement and careful analysis of:

- consequences of a failure
- heterogeneity of the soil conditions
- extent of the site investigation
- required lifetime of the slope
- accepted slope deformation, maintenance and effects on adjacent structures
- effects of earthquakes

For guidance, the following two examples are given:

If there are no structures or pipelines adjacent to the slope, maintenance is accepted and reliable soil data have been used in the design calculations, a long-term safety factor of at least 1.3 should be applied. For temporary slopes local failure may sometimes be acceptable. The short term safety factor may then be 1.1, provided that the design is based on reliable soil data.

Note After failure the (residual) soil strength will be less than the original strength, which may impede (future) construction works near the failed area.

7.2.2.3 Earthquakes

The analyses of earthquake effects on a slope shall include but not be limited to:

- horizontal and vertical accelerations
- increase of pore water pressures
- liquefaction potential

7.2.3 Monitoring

The stability of a slope should be monitored over a minimum period of six months after completion of that slope. As a minimum a monthly visual inspection for cracks and deformations shall be carried out. For major works monitoring should also comprise measurements of pore pressure, settlements and horizontal displacements.

7.3 SETTLEMENTS

The following types of settlement should be analysed:

a) consolidation of the fill material itself

b) settlement of the subsoil:

   b.1 - elastic (undrained) compression

   b.2 - (primary) consolidation

   b.3 - secondary compression or creep

Settlement calculations shall be based on the stress distribution that is appropriate for the

type of surcharge (e.g. flexible tank or rigid footing) and the type of subsoil.
The calculations shall be based on soil compressibility parameters determined by laboratory tests on samples from each compressible layer.

Appendix IX outlines the methods used by SIPM to evaluate settlement predictions.

7.4 SOIL IMPROVEMENT

7.4.1 General

The following methods may be applied separately or in combination:

1. Preloading
2. Drainage
3. Shallow compaction
4. Soil replacement

See (7.4.2).
Other methods of soil improvement may be allowed after field trials and approval by the principal. Such methods include:

- deep compaction by vibroflotation
- dynamic consolidation using a falling weight
- stone, lime or grout columns

The principal is recommended to consult SIPM.

7.4.2 Review of methods

7.4.2.1 Preloading

The design of the preloading shall include:

- area to be preloaded
- required weight of the preloading
- required duration of the preloading
- stability of the preloading
- prediction of the settlements versus time
- effects of vertical drainage, if relevant

During the preloading the actual settlements of the original subgrade shall be monitored. For major works, the pore water pressure in the compressible soil layers shall also be monitored.

7.4.2.2 Drainage

Horizontal drainage may be applied to lower the groundwater table. See (7.5) and to DEP 34.13.20.31-Gen., para. 2.4.10.

Vertical drains may be installed to accelerate the consolidation of compressible soil layers. Either sand drains or plastic drains may be applied.
Sand drains shall have a minimum diameter of 0.25 m. The sand shall contain at most 10% (mass) of particles smaller than 0.063 mm and less than 3% organic matter. The drains shall be installed by jetting or drilling, not by driving.
The manufacture of plastic drains is subject to approval by the principal.

To discharge the excess water the top of the drains shall be placed in a free draining (sand) layer of at least 0.6 m thickness. The drains shall penetrate the compressible soil layers to be drained. However, the drains shall be installed to a maximum depth of 1.0 m above the top of the water-bearing layer below those compressible layers. Care should be taken with regard to possible long term effects on the groundwater quality. See (7.5.2).

7.4.2.3 Compaction

Compaction results shall be checked by the contractor, using CPT's or equivalent test methods.

See (7.1.4) and DEP 34.11.00.11-Gen.

7.4.2.4 Soil replacement

The bottom of the excavation shall be inspected for soft spots before backfilling is started. The backfill shall be compacted. See (7.4.2.3).

7.5 DEWATERING AND DRAINAGE

7.5.1 Design and construction

For substantial dewatering systems, the design shall be based on the results of a site investigation summarized in a geo-hydrological schematization, including the following data:

- number and type of aquifers (water-bearing layers)
- hydraulic data per aquifer:
  - piezometric head
  - transmissivity
  - hydraulic resistance of confining layers
  - specific yield/storage coefficients
- existing groundwater and surface flows
- effects of rainfall, snow etc.
- chemical contents of the groundwater per water-bearing layer.

The design shall consider time effects (non-stationary flow) and stationary flow if relevant.
For major projects the geo-hydrological model and its parameters should be verified by a pumping test. Refer to British Standard BS 5930.

Attention shall be paid to the possibility of rotting and clogging of drains and filters. Washing out of fines from the surrounding soil shall be prevented. Porous drains shall be installed at such a depth that the system will be continuously submerged.

After installation of wells the output of each individual well should be tested.
It shall be analysed whether emergency provisions are required in case of sudden failure of the dewatering system.

7.5.2 Effects on environment

The design shall analyse the possible environmental effects of:

- discharging the drainage water (considering both quantity and biological/chemical contents)
- lowering the groundwater table (settlements, aeration of organic soils)
- groundwater flows directed inward or outward plot or downward to deep water bearing layers (e.g. via vertical drains)

8. EARTH RETAINING STRUCTURES

8.1 STRUCTURE TYPES

The following types of retaining structures may be applied:

Type Ia - sheet piling

Type Ib - diaphragm walls and pile walls

Type II - gravity type or L-shaped reinforced concrete walls

8.2 LOADS

The following loads shall be considered in the design if relevant:

- hydrostatic pressures due to open water or groundwater
- increased water pressures due to (ground) water flow
- (excess) pore pressures
- effective earth pressures
- external vertical or horizontal loads on or behind the retaining structures (e.g. mooring forces, traffic loads)
- earthquakes

See also (3.4) and (3.5).

8.3 SHEET PILING

8.3.1 Calculations and safety factors

The following failure mechanisms shall be considered in the design.

- Occurrence of the yielding stress (or of cracking) in any cross section of the wall
- Failure of the passive soil wedge at the toe of the wall
- Yielding of the anchor steel or failure of any coupling or girder
- Failure of the anchoring structure (e.g. ground anchor, anchor wall, batter piles)
- Loss of overall stability of the soil mass, including wall and anchoring structure, via a deep sliding surface, see (7.2)
- Loss of vertical stability due to vertical loads

The safety factors to be adopted per failure mechanism depend among other things on the design requirements and the type of anchoring structure, but shall be at least 1.5. See (3.6). Moreover the deformations of each part of the retaining structure shall be analysed.

The methods used by SIPM to evaluate the design of sheet piling are stated in Appendix X.

8.3.2 Corrosion

The corrosiveness of surface water, ground water, soil and backfill material shall be investigated. See (5.4.3).

8.4 DIAPHRAGM WALLS AND PILE WALLS

The design of diaphragm walls (reinforced concrete slurry walls) and pile walls is subject to approval by the principal, who is recommended to consult SIPM.

8.5 GRAVITY TYPE OR L-SHAPED RETAINING WALLS

In the structural design of this type of rigid retaining structures it shall be assumed that the acting pressure is the effective earth pressure at rest (being at least 50% of the effective vertical pressure) plus the (ground) water pressures.
The overall stability (overturning moment) as well as the maximum bearing pressure shall be calculated assuming active earth pressure plus (ground)water pressure as a minimum load. The safety factor against overturning shall be at least 1.5. The allowable bearing pressure shall be calculated according to (4.).

Moreover an overall stability analysis according to (7.2.2) shall be carried out.

9. GROUND ANCHORS

9.1 DESIGN

The design (including the safety factor adopted) shall take into account:

- type and manufacture of anchor
- site investigation results
- consequences of failure of an anchor
- design life of the anchor: permanent or temporary use
- all possible failure modes, including as a minimum:
  . failure within the soil or rock mass
  . failure of rock/grout or soil/grout bond . failure of grout/tendon bond
  . failure of grout body
  . failure of steel tendon or its connection(s)
- deformations, including time effects (creep) and the possibility of progressive failure
- effects of relaxation
- weight of overburden
- possible group effects

The Contractor shall submit a grout mix design including all additives for approval by the principal.
Each anchor (including tendon) shall be adequately protected against corrosion considering the required design life of the anchor and the aggressiveness of the environment, see (5.4.3). Reference is made to the FIP/2/7 Recommendations. The design of permanent anchors is subject to approval by the principal, who is recommended to consult SIPM.

9.2 INSTALLATION, SUPERVISION AND TESTING

9.2.1 Installation

When anchors are installed by drilling, a steel casing shall be applied in layers consisting of cohesionless material, in order to prevent collapse of the hole. The borehole and casing shall be thoroughly cleaned by flushing with clean water or bentonite slurry.
Boreholes in rock shall be tested for water-leakage rate, in order to determine whether pregrouting is required. Grouting shall be performed immediately after the borehole/casing is cleaned (standing overnight of the bored hole shall not be allowed). Refer to the FIP/2/7 Recommendations.

An anchor shall not be stressed until seven days after grouting and the cement grout has reached a cube strength of at least 28 N/mm^2.

9.2.2 Anchor Installation Log Book

As a means of Quality Assurance the contractor shall provide an Installation Log Book, recording the following data per anchor:

- description of drilling and/or driving equipment, drilling mud and procedures,
- date and time of drilling/driving,
- description of encountered strata and penetration rate,
- any loss of flushing medium,
- time of installation of the tendon,
- inspection findings on the tendon, including corrosion protection, spacers, etc.,
- time of grouting,
- results of viscosity measurements of the grout,
- number of test cubes made of the grout,
- date and time of stressing/testing.

9.2.3 Supervision

The installation and stressing/testing of the anchors shall be supervised by or on behalf of the principal. The supervision shall not be subcontracted to the contractor installing the anchors.

9.2.4 Load testing

9.2.4.1 Destructive testing of trial anchors

The first three anchors installed shall be trial anchors. They shall be test loaded up to the ultimate anchor capacity according to the design. For this purpose it may be necessary to apply a higher tendon capacity for these test anchors, in order to maintain a safety factor of at least 1.5 with regard to the tendon strength during the load test.

9.2.4.2 Acceptance testing of working anchors

Before stressing an anchor to its working load it shall be load tested. Each anchor with a design life of more than two years (permanent anchor) shall be tested up to 1.5 times its design load. Each temporary anchor shall be tested upto 1.2 times its design load. However, at least 5% of the temporary anchors shall be up to 1.5 times its design load.

The load tests shall be performed in accordance with the German Code DIN 4125, Parts 1 and 2.

9.2.4.3 Acceptance criteria

The design shall specify the acceptance criteria for the results of the load tests. The criteria shall refer to:

- total deformation
- elastic rebound
- creep
- permanent deformation after unloading

9.2.5 Monitoring

For anchors with a design life of more than two years and installed in soil susceptible to creep, monitoring should be considered. The monitoring should be performed by measuring the actual force in the tendon one year after prestressing. Approximately 5% of the anchors (to be selected by the principal) should be monitored.

10. REFERENCES

In this manual reference is made to the following publications:

Note: The latest issue of each publication shall be used together with any amendments/supplements/revisions to such publications, unless otherwise stated

DEPs

The use of SI units	DEP 00.00.20.10-Gen.
Minimum requirements for structural design and engineering	DEP 34.00.01.30-Gen.
Site investigations	DEP 34.11.00.10-Gen.
Site preparation and earthworks	DEP 34.11.00.11-Gen.
Roads, paving, surfacing, slope protection and fencing	DEP 34.13.20.31-Gen.
Reinforced concrete foundations and structures	DEP 34.19.20.31-Gen.

US Standards

Natural building stones - soil and rock	ASTM - Volume 04.08

Issued by:
American Society for Testing and Materials
1916 Race Street, Philadelphia
PA 19103 USA

Planning, designing and constructing fixed offshore platforms	API-RP 2A

Issued by:
American Petroleum Institute
211 N. Ervay, Suite 1700
Dallas
TX 75201
USA

British Standards

Site investigations	BS 5930
Earthworks	BS 6031
The structural use of concrete	BS 8110
Foundations	CP 2004

Issued by:
British Standards Institution
2 Park Street, London W1A 2BS
England

German Standards

Verpreβanker im Lockergestein (Ground Anchors)	DIN 4125, part 1 and 2

Issued by:
Beuth Verlag GmbH
Burggrafenstrasse 4-10
D-1000 Berlin 30, Germany (BRD)

Recommendations of the Committee for Waterfront Structures (English version)	EAU 1985

Issued by:
Verlag W. Ernst und Sohn

Hohenzollerndamm 170
1000 Berlin 31, Germany (BRD)

International Recommendations

FIP Recommendations for the design and construction of prestressed concrete ground anchors (1982)	FIP/2/7

Issued by:
Federation Internationale de la Preconstrainte (FIP)
Wexham Springs, Slough SL3 GPL
England

11. APPENDICES

I	General principles and terms in geotechnical engineering
II	Ultimate bearing capacity of spread foundations
III	Pile bearing capacity
IV	Calculation principles for negative friction on piles
V	Tension capacity of driven piles
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APPENDIX I GENERAL PRINCIPLES AND TERMS IN GEOTECHNICAL ENGINEERING

I-1 TERMS AND SYMBOLS

I-1.1 PHYSICAL PROPERTIES

Symbol	Term	Dimension
ρ	density of solid particles	kg/m^3
γ_s	unit weight of solid particles	kN/m^3
γ_w	unit weight of water	kN/m^3
γ	unit weight of soil (bulk)	kN/m^3
γ_d	unit weight of soil (dry)	kN/m^3
e	void ratio	%
e_{max}	void ratio in loosest state	%
e_{min}	void ratio in densest state	%
D_r (I_D)	relative density (density index)	%
n	porosity	%
W	water content	%
W_L	liquid limit (LL)	%
W_p	plastic limit (PL)	%
I_p	plasticity index (PI) = W_L-W_p	%
I_L	$\text{liquidity index} = \frac{W - W_p}{I_p}$	%
i	hydraulic gradient	m/m
k	coefficient of permeability	m/s

I-1.2 IN-SITU TESTS

Symbol	Term	Dimension
q_c	cone resistance measured with CPT	MPa (= MN/m^2)
f_s	local side friction measured with CPT	MPa
N	SPT blow count	-

I-1.3 CONSOLIDATION

Symbol	Term	Dimension
C	compressibility coefficient in: $s = \frac{1}{C} \cdot h \cdot \ln \frac{\sigma' + \Delta\sigma'}{\sigma'}$	-
C_c	compression index in:	-

$$s = \frac{C_c}{1+e_0} \cdot h \cdot \ln \frac{\sigma' + \Delta\sigma'}{\sigma'}$$

C_p	compressibility coefficient for direct (primary) compression	-
C_s	compressibility coefficient for secondary compression	-
c_v	coefficient of consolidation	m^2/s
m_v	coefficient of volume change	m^2/kN

I-1.4 SHEAR STRENGTH

τ_f	shear strength	kP_a
c'	effective cohesion	kP_a
ø	effective angle of internal friction	(degrees)
c_u	apparent cohesion (undrained)	kPa
$ø_u$	apparent angle of internal friction	(degrees)

I-1.5 STRESS AND STRAIN

u	pore pressure	kPa (= kN/m^2)
c'	total normal stress	kPa
ø	effective stress = σ- u	kPa
τ	shear stress	kPa
v	Poisson's ratio	-
E	modulus of linear deformation	kPa
G	modulus of shear deformation	kPa
K	modulus of compressibility	kPa

I-1.6 EARTH PRESSURE

δ	angle of wall friction	(degrees)
c_a	pile shaft or wall adhesion	kPa
K_a,K_p	active and passive earth pressure coefficients	-
K_o	coefficient of earth pressure at rest	-

I-1.7 FOUNDATIONS

A	area of foundation	m^2
B	width of foundation	m

L	length of foundation	m
D	depth of foundation beneath ground level	m
Q	applied axial load on piles	kN
Q_p	total base resistance of piles	kN
Q_s	total shaft friction of piles	kN
s	settlement	m or mm
N_c, N_q, N_γ	bearing capacity factors	-

I-2 EFFECTIVE SOIL STRESSES

The total vertical stress σ_v (z) in the soil at a certain depth z is the cumulated weight of the soil layers above level z, including the weight of any surcharge if appropriate.

At the level z the pore water (groundwater) pressure is called u(z).

The effective vertical stress σ_v' (z) is the pressure between the soil particles proper. The definition is:

effective stress = total stress - pore water pressure

or $\sigma_v' \sigma_v$ (z) $= \sigma_v(z) - u(z)$

In geotechnical calculations, effective stress analysis will give the most realistic results, particularly in (partially) drained situations. For undrained clay or peat, total stress analyses also have to be made, assuming the undrained cohesion is c_u and the angle of internal friction is ø = 0.

I-3 BASIC SOIL PARAMETERS

I-3.1 DENSITY AND VOLUME WEIGHT

The density or unit mass (ρ) is defined as the mass per unit of volume (kg/m^3). The unit weight (γ) is the weight per unit of volume (kN/m^3). Typical average values for the unit weight are:

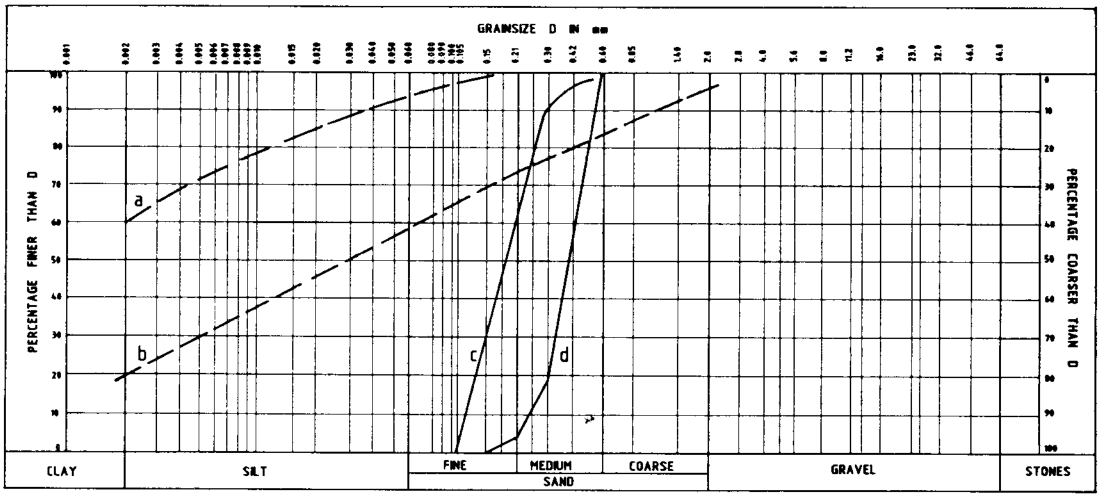
saturated sand	20	kN/m^3
dry sand	17	kN/m^3
silt	18	kN/m^3
saturated sandy clay	16-18	kN/m^3
saturated clay	14-16	kN/m^3
satureated peat	10-11	kN/m^3
water	10	kN/m^3

The unit weight of soil particles depends on the grain material. Some average values are:

siliceous sands	26.5	kN/m^3
clay minerals	27.5	kN/m^3
non porous chalk particles	26.5	kN/m^3

I-3.2 GRAIN SIZE DISTRIBUTION

The particle or grain size distribution of a soil is determined by sieving for particles larger than 0.06 mm and by sedimentation for particles smaller than 0.06 mm. The test results are presented in curves. Fig. I-1 shows some examples.



a. clay c. dune sand

b. boulder clay d. river sand

Fig. I-1: Examples of grain size distribution

I-3.3 WATER CONTENT, POROSITY AND VOID RATIO

The water content is defined as the ratio between the weight of the pore water and the weight of the solid particles, expressed as a percentage. Porosity is the ratio between the volume of voids and the total volume of the soil.
The void ratio is the ratio between the volume of the voids and the volume of the solid particles.
The degree of saturation is the ratio between the volume of pore water and the volume of voids.
Typical average values for fully saturated soils are (indicatively):

	unit weight	e void ratio	n porosity	w water content
sand	20 kN/m^3	0.67	40%	25%
clay	16-14 kN/m^3	1.78-3.17	64-76%	67%
peat*)	10 kN/m^3	2-8	65-90%	200-800%

*) because γ_s is approximately equal to γ_w the water content of peat cannot be derived from the unit weight.

In sand, the density and porosity can vary substantially, depending on grain size distribution, particle shape and water content. The maximum and minimum density can be determined by laboratory tests. These values depend also on the methods by which the maximum and minimum densities are measured.

The relative density (D_r) is defined as:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\% = \frac{\gamma_{dmax}}{\gamma_d} \times \frac{\gamma_d - \gamma_{dmin}}{\gamma_{dmax} - \gamma_{dmin}} \times 100\%$$

D_r varies between zero (very loose) and 100% (very dense), while e may vary between 26% and 50%.

I-3.4 ATTERBERG LIMITS

Depending on its moisture content, a fine-grained soil can be in a liquid, plastic, semi-solid or solid state (Fig. I-2). The change from one state to another is defined by limits which are expressed in terms of water content.

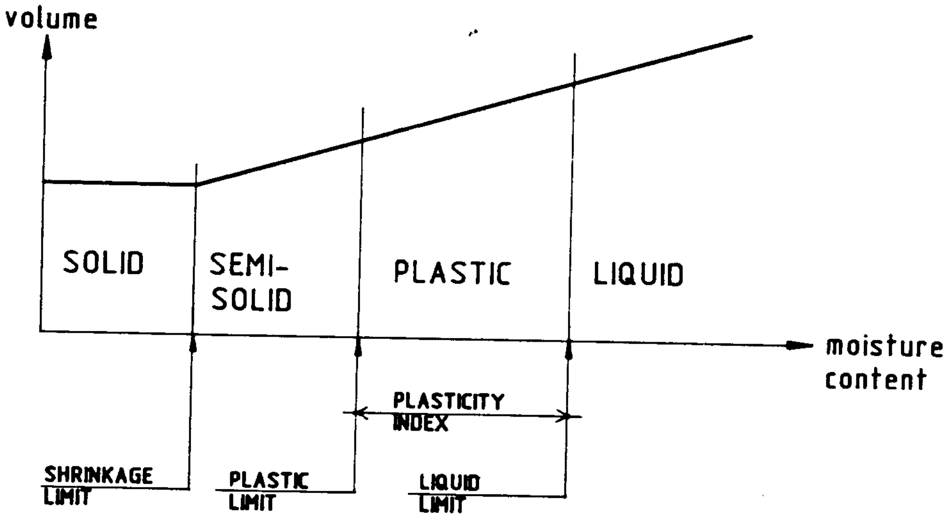


Fig. I-2: Relationship between soil volume and moisture content

The two most important limits are the liquid (W_L) and the plastic limit (W_p) which define the upper and lower bounds of the plastic state. The difference between these limits is the plasticity index (I_p) of a soil. Other terms used are:

- liquidity index (I_L), which relates the natural moisture content of the soil to both the plastic and liquid limits by the relationship:

$$I_L = \text{liquidity index} = \frac{W - W_p}{i_p} \times 100\%$$

- shrinkage limit, which is the moisture content at which the soil remains at a constant volume when drying out.

Determination of the liquid and plastic limits is performed on material with a maximum particle size of 0.425 mm.

I-3.5 COMPRESSIBILITY AND CONSOLIDATION

I-3.5.1 General

Compressibility of soil depends on the water content and the grain size distribution. For a saturated soil compression can only occur if water is dissipated. Depending on the permeability of the soil, the dissipation of the pore water will take more or less time. This process is called consolidation.
The stress history is an important factor. If a soil is preloaded and consolidated under the preload, the compressibility is much lower than under normal load conditions. If a soil is unloaded, swelling will occur. The swelling of a soil is generally a factor of 4 to 10 smaller than the compression. However, some types of clay may show a much larger swell.

I-3.5.2 Laboratory tests

The compressibility is determined in the laboratory by means of compression or oedometer tests on undisturbed samples.
The compression s of a soil sample with thickness h, original effective stress σ_o' and a stress increase of σ_1' is given by the equation:

$$s = \frac{1}{C} \cdot h \cdot \ln \frac{\sigma_0' + \sigma_1'}{\sigma_0'}$$ in which:

C = compressibility coefficient (dimensionless)

ln = natural logarithm = 2.3 log10

The C-value is determined by measuring the settlements during the compression test.

Typical average values are:

peat	5 to 10
clay	10 to 20
loam, silt or stiff clay	20 to 50
sand	50 to 500

For clay and peat the compression is time dependent, due to the dissipation of the pore water. The following equation gives the time T_e in which 92% of the consolidation is complete. For engineering purposes this is often taken as the end of the consolidation.

$$T_e = \frac{h^2}{4 \cdot c_v} (\text{sec})$$

in which:

h = thickness (m) of layer, free draining in two directions

$$c_v = \frac{k}{m_v \cdot \gamma_w}$$ = coefficient of consolidation (m^2/sec)

k = permeability (m/sec)

m_v = compressibility (m^2/kN)

γ_w = unit weight of water = 10 kN/m^3

Typical values of c_v for clays are between 10^{-8} and 10^{-6} m^2/s

I-3.5.3 Secondary compression

This is the compression that occurs after the excess pore pressure has substantially dissipated. After completion of the theoretical compression/consolidation, in practice further compression may be observed, which is called secondary compression.

The secondary compression is included in the Terzaghi-Buisman formula:

$$s = h\left(\frac{1}{C_p} + \frac{1}{C_s}\log\ t\right)\ \ln\frac{\sigma'_0 + \sigma'_1}{\sigma'_0}$$ in which:

C_p and C_s are compressibility coefficients (dimensionless)

t = time (days)

σ'_o = original effective vertical stress (kN/m^2)

σ'_1 = effective vertical stress increase (kN/m^2)

h = layer thickness (m)

I-3.6 SHEAR STRENGTH

The drained shear strength of a soil determines the long-term stability of foundations and embankments. Furthermore, knowledge of the shear strength is needed for, among other things the determination of earth pressures against retaining structures such as sheet piling.

In general the shear strength of a soil is expressed by the Mohr-Coloumb formula:

τ = $c' + \sigma' \tan(ø)$ (kN/m^2)

in which:

c' = cohesion (kN/m^2)

ø = friction angle (degrees)

σ' = effective stress (kN/m^2)

For permeable cohesionless soils such as sand and gravel, the load applied on the soil instantaneously increases the effective stress. For cohesive soils such as clay, peat and silt, the load applied initially increases the pore pressure, while the effective stress remains constant or is only partly increased.

For sand the formula becomes:

τ = $\sigma'.\tanø$

For undrained cohesive soils the formula becomes:

τ = c_u

APPENDIX II ULTIMATE BEARING CAPACITY OF SPREAD FOUNDATIONS

II-1 BASIC FORMULA

The ultimate bearing capacity of a long strip with a vertical line load (Fig. II-1) can be determined with the formula:

$$q = c \cdot N_c + q_0 \cdot N_q + \gamma \cdot \frac{B}{2} N_\gamma$$

in which:

q	=	ultimate bearing capacity	kPa
c	=	cohesion	kPa
q_o	=	unit charge outside the foundation at depth D	kPa
B	=	width of the foundation	m
γ	=	effective unit weight of the soil 0-2B below foundation level	kN/m^3

N_c, N_q, N_γ bearing capacity factors (from Fig. II-2), being a function of ø

The ultimate bearing capacity of the strip is Q = q.B (kN/m')

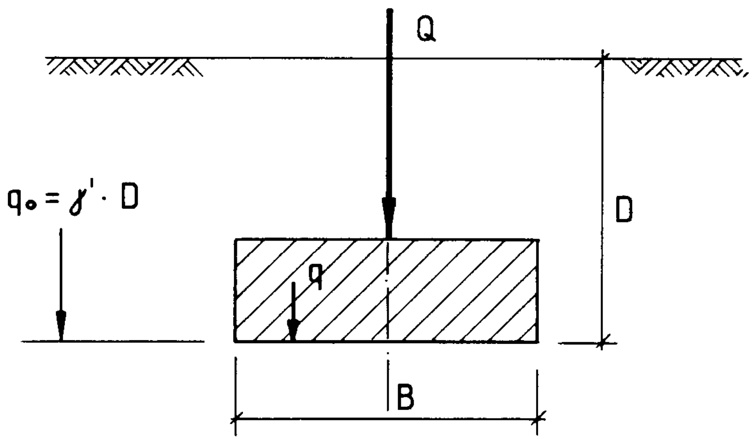
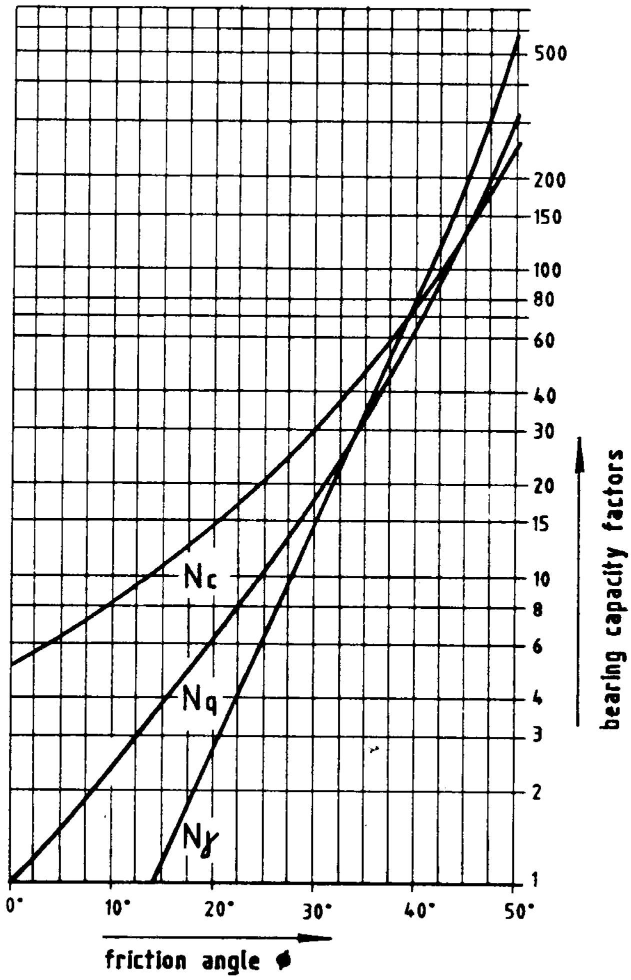


Fig. II-1: Basic case: vertical centric line load on a strip foundation



Typical values of Ø are:

very loose sand	Ø = 27.5°	loose sand	Ø = 30°
medium dense sand	Ø = 32.5°	dense sand	Ø = 35°
very dense sand	Ø = 37.5°		

Fig. II-2: **Bearing capacity factors**

II-2 EXTENDED FORMULAE, ACCORDING TO BRINCH HANSEN:

For other shapes of the foundation and for inclined loads on the foundation, the formula of (II-1) has to be extended with a shape factor and/or a load inclination factor. For a horizontal ground surface, the extended formula is:

$$q = (q_0 + c \cot \phi) \cdot N_q \cdot s_q \cdot i_q + \gamma \cdot \frac{B}{2} \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma$$

s = shape factor

i = load inclination factor

B = effective width (m)

(kPa)

$$q = \frac{Q}{A}$$

A = effective foundation area (m^2)

For the special case of ø = 0 (undrained cohesive soils) the following formula should be applied:

$$q = (\pi + 2) c_u (l + s_c^a - i_c^a)$$

For foundations on or near sloping surfaces the bearing pressures have to be reduced.

II-2.1 EFFECTIVE FOUNDATION AREA IN THE CASE OF EXCENTRIC LOADING

The resultant load Q on the base of the foundation has a vertical component V and a horizontal component H. The effective foundation area of rectangular shape is determined in such a way that its geometric centre coincides with the load centre and that it follows the nearest contours of the actual base area as closely as possible. Some examples are given in Fig. II-3.

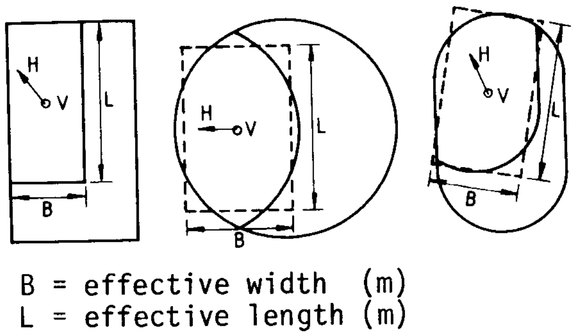


Fig. II-3: Equivalent and effective foundation areas

The bearing capacity of the centricaly loaded effective foundation area is a good approximation of the actual bearing capacity of the eccentrically loaded foundation.

II-2.2 SHAPE FACTOR s

The following shape factors should be used for drained soils:

$$s_\gamma = 1 - 0.35\frac{B}{L}$$

$$s_q = 1 + 0.35\frac{B}{L}$$

For ø = 0 (undrained clay) applies:

$$s_c^a = 0.2\frac{B}{L}$$

(for circular footings: B = L = diameter)

II-2.3 INCLINATION FACTOR i

An inclination of the load implies a reduction of the bearing capacity.

For drained soils the inclination factors are:

i_q = $(1 - 0.5\, H/(V + A.c.\cot ø))^5$

and

i_γ = $(1 - 0.7\, H/(V + A.c.\cot ø))^5$

For ø = 0 (undrained clay) applies:

$$i_c^a = 0.5 - 0.5\sqrt{1 - H/A \cdot c_u}$$

II-3 REMARKS

After the excavation of the soil to the foundation depth, the foundation base has to be inspected carefully in order to check whether the visual soil conditions comply with the data and the recommendations from the soils report. Silty soils, with a low plasticity index, are in general very sensitive to disturbance during excavation under wet conditions. However, reduction of the allowable bearing capacity is not required if the foundation base is carefully handled and disturbance is minimized.
Groundwater conditions can be of great influence on the bearing capacity. The extreme possible conditions should be considered. Excavations close to shallow foundations may affect the bearing capacity. For safety reasons the foundation may be considered as a surface foundation.

APPENDIX III PILE BEARING CAPACITY (DRIVEN COMPRESSION PILES)

III-1 GENERAL

Preferably the bearing capacity of piles should be calculated from the results of CPT's, see (III-2).
Alternatively the bearing capacity of piles can be calculated from the angle of internal friction ø, see (III-3).
The presented methods apply to single piles (prefabricated driven type) bearing in siliceous sands. For piles bearing in other types of sand (e.g. calcareous sands) or in clay, the advice of a specialist consultant is required.

III-2 BEARING CAPACITY CALCULATED FROM CPT-CONE RESISTANCE

III-2.1 END BEARING

The ultimate end bearing pressure q_p of the pile tip can be determined from the cone resistances q_c measured with a CPT, using the following empirical calculation method consisting of seven steps (see Fig. III-1):

Step 1 - Determine the (equivalent) solid pile tip diameter D. For a square pile tip with a width B,

$$D = B \cdot \sqrt{4 / \pi} = 1.13\,B\ (m)$$

Step 2 - Assume a (preliminary) pile tip level.

Step 3 - Determine the average (I) of the cone resistances over a depth b.D below the tip, taking depth b.D as determined in step 4 (b is a ratio between 0.7 and 4).

Step 4 - Determine the average (II) of minimum cone resistances below the pile tip going upward over b.D, starting at such a depth b.D between 0.7 D and 4 D below the tip, which leads to the lowest average (II) of minima. If, going in upward direction, the cone resistance increases, the previous lower minimum cone resistance should be taken instead of the recorded value.

Step 5 - Calculate the average value of I and II: $q_b = {}^1/_2\,(I + II)$.

Step 6 - Determine the average q_a (III) of minimum cone resistances going upwards from the pile tip level over a.D (a = 8) distance. As a starting value the minimum of the cone resistances found in step 4 is taken. If, going in upward direction, the cone resistance increases, the previous lower minimum cone resistance should be taken instead of the recorded value.

Step 7 - The ultimate end bearing capacity is calculated from the equation:

$$q_p = \frac{1/2\,(I + II) + III}{2} = 1/2 \quad (q_b + q_a)(kN/m^2)$$

The resulting q_p value should be limited to 15,000 kPa. In gravel and overconsolidated sand an even lower limit may have to be adopted.

Particularly via step 4, the above method emphasizes the risk of punching that may exist when layers with low cone resistance occur at 0-4 D depth below the pile tip. If such weaker layers consist of loose sand, some compaction during pile driving and consequently some increase of cone resistance may be expected. However, low cone resistances due to clay or peat layers will certainly not increase due to pile driving.

Note: An alternative CPT interpretation method exists in which only the cone resistances between 2 D below and 5 D above the pile tip level are considered. This method gives less conservative results, but its application requires more expertise.

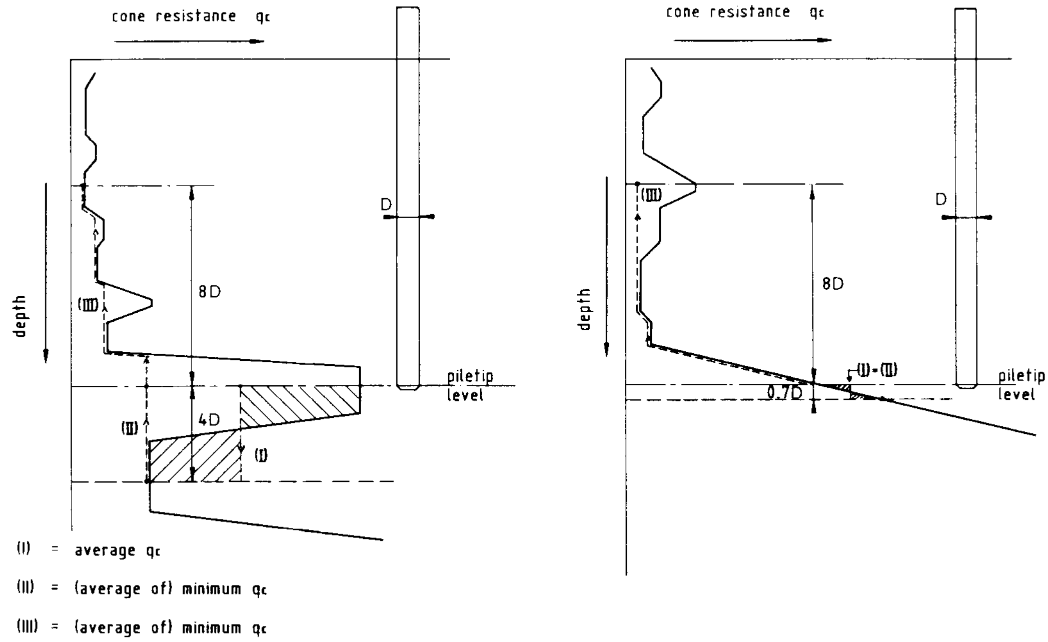


Fig. III-1: Examples of pile bearing capacity calculation based on CPT cone resistances

III-2.2 POSITIVE FRICTION

The ultimate upward (= positive) friction f_p between soil and pile shaft can be derived from the CPT cone resistance q_c,

according the following equation $f_p = C_f \cdot q_c$ (kPa).

C_f is a friction factor depending on soil type and pile type. Of course, the reliability of this method depends strongly on the reliability of the CPT data. A cross check according to the method described under III-3.2 is required.

In granular soil, the following C_f values may be applied:

C_f = 0.003 - 0.005 for open end and H-type piles

C_f = 0.005 - 0.010 for closed end steel, concrete or timber piles

In any case the positive unit friction should be limited to 120 kPa.

III-3 BEARING CAPACITY CALCULATED FROM FRICTION ANGLE ø

III-3.1 END BEARING

For piles bearing in granular soil (e.g. sand) the ultimate bearing pressure qp should be calculated according to the method published by G.G. Meyerhof (Geotechnique No. 2, 1951, pp. 301-332).

For estimating purposes, the ultimate end bearing pressure q_p of driven prefabricated piles (without enlarged base) may be approximated by the equation:

$q_p = \sigma_v' \cdot N_q'$ (kPa)

in which:

σ_v' = effective vertical stress at pile tip level (kPa)

N_q' = bearing capacity factor (-) from Fig. III-2 as a function of ø, D (equivalent pile diameter) and H (penetration depth into the granular bearing layer).

The resulting q_p value should be limited to 1500 kPa

For piles with shallow penetration in the bearing layer the method presented in Appendix II may also be used to estimate the pile bearing capacity.

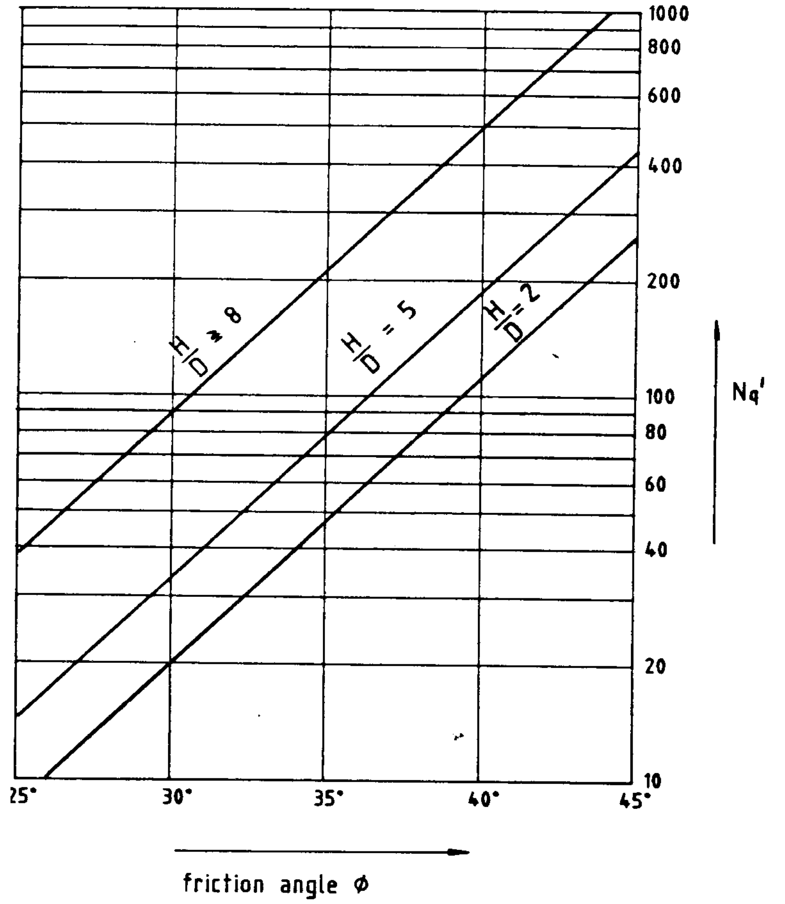


Fig. III-2: **Approximate values for xbearing capacity factor N_q', based on Berezantsev, 1961, Terzaghi, 1943 and Meyerhof, 1953**

III-3.2 POSITIVE FRICTION

The ultimate positive skin friction between the pile and the granular soil of the bearing layer can be calculated from:

f_p = $\sigma_v{}'$. K . 3/4 . tan (ø) (kPa)

in which:

$\sigma_v{}'$ = effective vertical stress at the relevant depth (kPa)

ø = angle of internal friction of the bearing soil

K = earth pressure coefficient, depending on soil displacement /compaction during pile driving:

K = 0.5 - 1.0 for open end and H-type piles

K = 1.0 - 2.0 for closed-end steel, concrete or timber piles

For (thin) clay layers in the bearing sand layer, the K value should be limited to 0.7.

In any case the ultimate positive friction should be limited to maximum 120 kPa.

APPENDIX IV CALCULATION PRINCIPLES FOR NEGATIVE FRICTION ON PILES

Downdrag load (negative friction) on piles occurs when the surrounding soil settlement exceeds the pile settlement. The stress increase Δp causing the soil settlement may be due to a fill or to dewatering.

The ultimate negative shaft friction Q_{nu} is limited by the following two factors:

1. Maximum effective weight of the fill or soil causing the settlement and the downdrag forces on the pile:

   (see Fig. IV-1)

   $$Q_{nu_1} = A_i \cdot \Delta p$$

2. Sum of maximum local friction forces that can be developed between pile and soil. This value is determined by the strength of the soil:

   (see Fig. IV-1)

   $$Q_{nu_2} = C \cdot \int_0^H \beta \cdot \sigma_V' \cdot dz$$

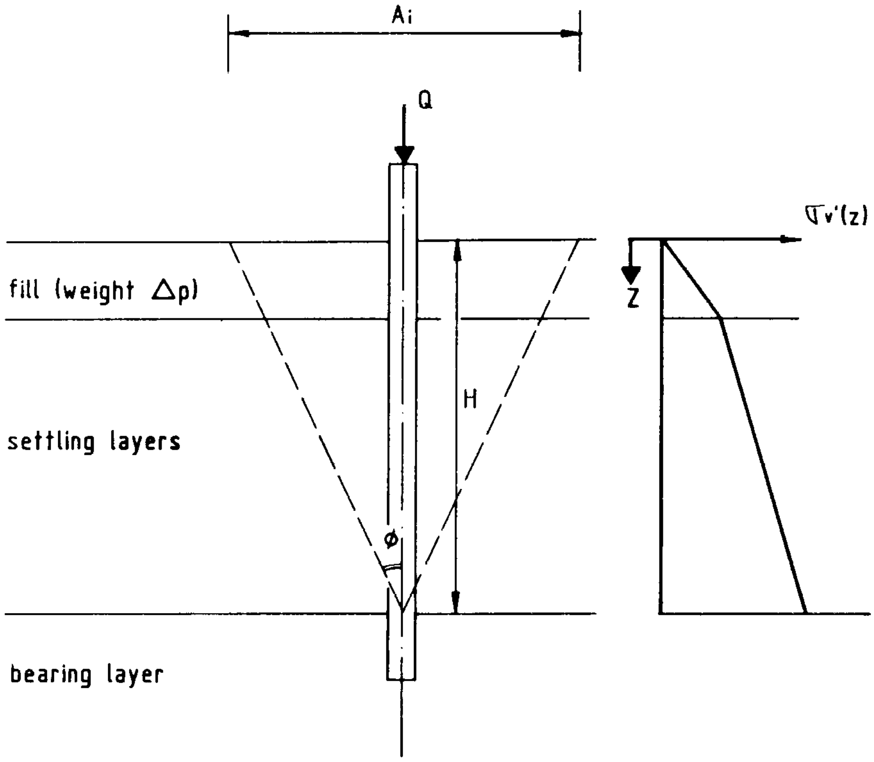
The slip factor β depends on the soil type and the pile type. Indications of β values for prefabricated (driven) piles are given below:

soft clay	β = 0.20 to 0.25
silt and sandy clay	β = 0.25 to 0.35
sand	β = 0.35 to 0.50

If a group of piles is driven in medium to dense sand with pile spacings of less than eight pile diameters, "-values between 0.6 and 1.2 have to be adopted.

On the other hand, the negative friction load Q_{nu} per pile may be reduced by placing the piles in a group with a narrow pile grid of 1 pile per A_g (m^2) area. For the inner piles of the group Q_{nu} will be reduced if A_g is smaller than A_i (defined in Fig. IV-1) and

$Q_{nu_1} = A_g \cdot \Delta p$ is less than Q_{nu_2} defined above.



Δp	=	stress increase causing soil settlement	(kPa)
Q	=	pile load causing pile settlement	(kN)
H	=	depth of settling layers	
A_i	=	influence area for negative friction	
	=	$\pi(H.tg\ ø)^2$ (m^2)	
σ_v'	=	effective vertical stress (including Δp)	(kPa)
ø	=	friction angle of settling soil	

The ultimate negative shaft friction Q_{nu} (kN) is the smaller of the following values:

$$Q_{nu_1} = A_i \cdot \Delta p$$

$$'Q_{nu_2} = C \cdot \int_0^H (\beta \cdot \sigma_v')dz$$

in which

β	=	slip factor, depending on soil type	(-)
C	=	pile circumference (m)	

Fig. IV-1: Downdrag on a single pile (negative friction)

APPENDIX V TENSION CAPACITY OF DRIVEN PILES

In general the ultimate tension capacity of piles should be calculated from the pile skin friction in granular soil, disregarding the friction in soft soil (clay) layers. Piles with enlarged base should not be used to carry tension loads.

The ultimate soil friction f_t on a prefabricated driven pile can be calculated using either of the equations:

$f_t \quad = \quad 0.7 \,.\, C_f \,.\, q_c$

$f_t \quad = \quad 0.7 \,.\, \sigma_v' \,.\, K \,.\, 3/4 \,.\, \tan(\varnothing)$ (preferred approach)

The parameters in these equations are defined in Appendix III, (III-2.2) and (III-3.2) respectively.

The ultimate friction f_t should be limited to maximum 120 kPa.

For a group of tension piles an additional requirement applies (See Fig. V-1): The (submerged) weight of the soil in zone ABCD should be at least 120% of the total (uplift) working load on the foundation. Working load being the design load minus the (submerged) weight of the foundation (piles).

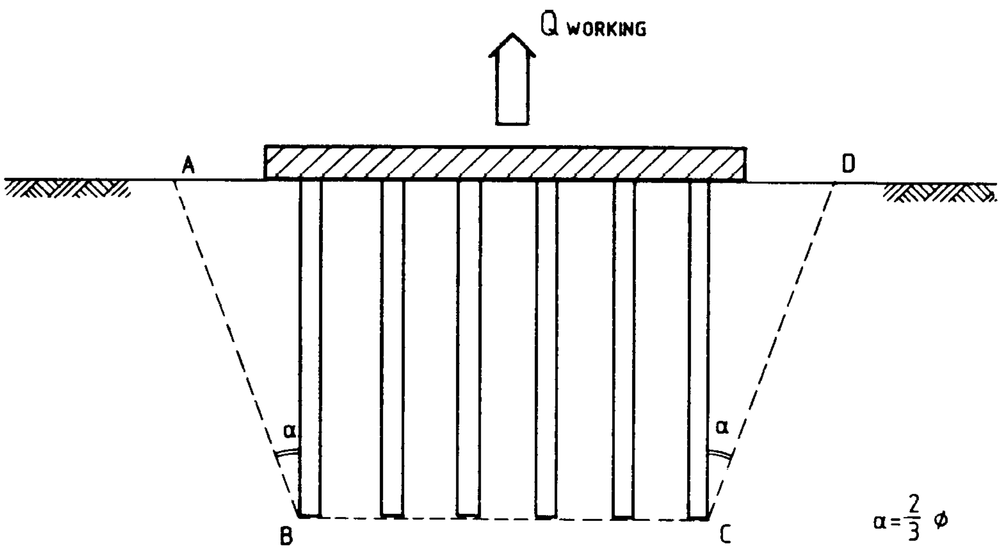


Fig. V-1: Group effects under tension load.

APPENDIX VI CALCULATION PRINCIPLES FOR LATERALLY LOADED PILES

VI-1 OVERTURNING STABILITY

The maximum lateral soil resistances (passive pressures) should be calculated from the equations:

$q(x) = B.p(x)$

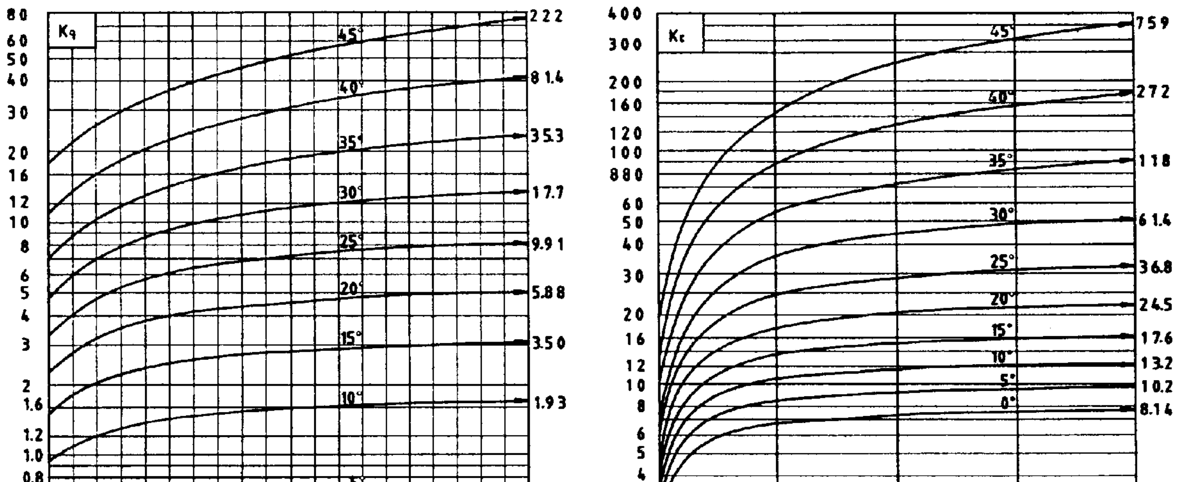
$p(x) = \sigma_v'(x) \,.\, K_q(x) + c.K_c(x)$

in which:

x	=	depth (m)
q	=	lateral line load (kN/m')
p	=	lateral passive earth pressure (kN/m^2)
B	=	pile width (m)
σ_v'	=	effective vertical stress (kPa)
c	=	cohesion (kPa)
K_q and K_c	=	earth pressure coefficients according to Fig. VI-1

The active earth pressure at the back of the pile is disregarded.

Assuming a (preliminary) pile length and knowing the distribution of the lateral earth pressures along the pile length, the depth of the centre of rotation of the pile is calculated from the equilibrium of rotating moments. Finally the ultimate lateral bearing capacity of the pile is calculated from the equilibrium of horizontal forces.



Earth pressure coefficient for overburden pressure. Earth pressure coefficient for cohesion

Fig. VI-1: Lateral (passive) earth pressure coefficients for piles (Brinch Hansen, 1961)

VI-2 PILE DEFLECTION AND BENDING MOMENTS

In general the main criteria in the design of horizontally loaded piles are the allowable pile head displacement y and the allowable bending moments M.
In order to limit the pile deflections it is necessary to keep the soil deformation within the 'elastic' range.

The analysis of the pile deflection should be based on the calculation methods for 'beams on elastic foundations'. These methods start from the basic differential equation:

$$(EI)_p \cdot \frac{d^4y}{dx^4} + K(x) \cdot y = 0$$

in which:

$(EI)_p$	=	bending stiffness of the pile (kN/m^2)
x	=	depth (m)
y = y(x)	=	lateral pile displacement (m)
K(x)	=	k(x) . B (kN/m^2)
k(x)	=	lateral subgrade (stiffness) modulus of the soil (kN/m^3)
B	=	pile width (m)

The type of solution of the above equation mainly depends on the relative pile flexiblity L/T (L = pile length).

T is stiffness ratio: $T = 4\frac{EI}{K}$

If a ratio L / T > 2 applies, the actual pile length does not influence the type of solution.

L / T > 2 indicates long flexible piles

L / T < 2 indicates short rigid piles

Various simplified solutions have been published for the pile deflections and bending moments following from the above approach. However, such simplified solutions generally do not take into account:

- the actual variation of the subgrade modulus k with depth (as determined from the site investigation).
- effects of local yielding (in-elastic performance) of the soil, particularly in top layers.
- soil degradation (remoulding) due to large deformations and/or cyclic loads

For piles with a major lateral load (e.g. dolphins) requiring an accurate design sufficiently accurate assessment of the subgrade modulus k at various depths is necessary. Moreover computer calculations are required. A typical computation model is illustrated in Appendix X, Fig. 3.

APPENDIX VII AXIAL PILE LOAD TESTS

VII-1 TESTING INSTALLATION

For a compression test either kentledge or tension piles or ground anchors may be used as reaction facility. The application of kentledge is preferred because piles or anchors may affect the test result. For testing a tension pile, the reaction force may be transmitted via a spread foundation (or via compression piles). Depending on the soil conditions and the geometry of both test pile and reaction installation, the test results are influenced by the distance between the test pile and the foundation of the reaction frame.
Reaction piles should be founded at least 2 m deeper than the test pile. The centre-to-centre distance between the test pile and any reaction pile should be at least four pile diameters (of the largest pile) or 3 m, whichever is the greater. The minimum spacing between the test pile and the nearest edge of the (kentledge) support should be 2.5 m, or three times the pile diameter.

As a force input element, an hydraulic jack should be used.

VII-2 MEASURING EQUIPMENT

The hydraulic jack should be equipped with a suitable, calibrated pressure gauge. The load on the pile head should be measured directly, using a calibrated electric force transducer (also called load cell).

The pile head displacement can be measured discontinuously via gauges or by optical levelling, or continuously by means of electric displacement transducers. The transducers/gauges should be mounted on a reference beam that is founded at least 3 m from any loaded pile or (kentledge) support. All instruments, beams and supports should be protected from direct sunshine, temperature changes and wind.

The foundation(s) of the levelling instrument and/or the reference beam should be checked for movements by frequent levelling (once per 30 minutes), relative to a datum placed 10 to 30 m from the load test area.

Because of possible tilting, the pile head displacement should be measured simultaneously at three points, uniformly spaced around the perimeter of the pile head.

The accuracy of the load measurements should be 1% of the maximum load that is applied. The displacements should be recorded in units of 0.01 mm. The overall accuracy of the measurement should be 0.1 mm.

VII-3 TEST RESULTS EVALUATION

The acceptance criteria for the load test results shall be specified in the design.

As an example acceptance criteria for a small diameter (maximum 600 mm) prefabricated driven compression piles in sandy soil are given below:

- the total pile head settlement under working load should be maximum 1 mm per 100 kN.
- the permanent pile settlement after unloading should be maximum 10 mm and maximum 50% of the total pile settlement.
- 2 hours after application of the load step $Q_t = 1.00\ Q_a$ (+ N_f) (refer to table 5.12.4) the pile settlement rate should be less than 0.15 mm per 30 minutes.
- 6 hours after application of the load step $Q_t = 1.50\ Q_a$ (+ N_f) the pile settlement rate should be less than 0.15 mm per 30 minutes.

APPENDIX VIII PRINCIPLES OF SLOPE STABILITY CALCULATIONS

Various modes of failure should be considered (see Fig. VIII-1):

- circular rotational slide
- non-circular rotational slide
- planar slide
- wedge slide

Planar and non-circular rotational slides are especially important if the sliding surface can follow a well defined weak layer, fault or crack.

For simplicity, two-dimensional stability analyses are carried out, which is generally a safe approach.

Clear distinction has to be made between short-term and long-term stability, as illustrated in Fig. VIII-2. Total stress analysis (i.e. based on undrained shear strength) is only allowed for the determination of the short-term stability of embankments or excavations which are constructed rapidly. For the calculation of intermediate and long-term stability, effective stress analysis shall be used taking into account the pore pressures along the slip surface. If applicable, the pore pressures shall include excess pore pressures due to recent fills and other surcharges or due to seismic loading.

The main principles of a stability calculation are illustrated in Fig. VIII-3 for an effective stress analysis of a circular rotational slide:

- A possible slip surface, defined by a rotation centre and a radius, is assumed
- The slice of soil above the slip surface is divided into a number of vertical strips with width = b.
- Depending on the calculation method applied, the forces interacting between the strips are defined, e.g. using the Bishop Simplified Method the interacting forces are assumed to be zero.
- The weight W of the soil (and water) mass inside a strip acts as a vertical force in the centre of gravity.
- The shear stress acting along the base of a strip is defined as:

  $\tau = c' + (p - u).\tan ø$,

  in which

  $p = W(\cos \alpha)^2/b$ and

  $u =$ pore pressure at the base of the strip.
- According to the Bishop Simplified Method the safety factor F (also called stability factor) of the slope follows from the equation given in Figure VIII-3.

The equation requires an iterative solution. Moreover, the calculation has to be repeated for all likely rotation centres and radii of the critical slip circle. For these reasons computers are generally used to perform stability calculations.

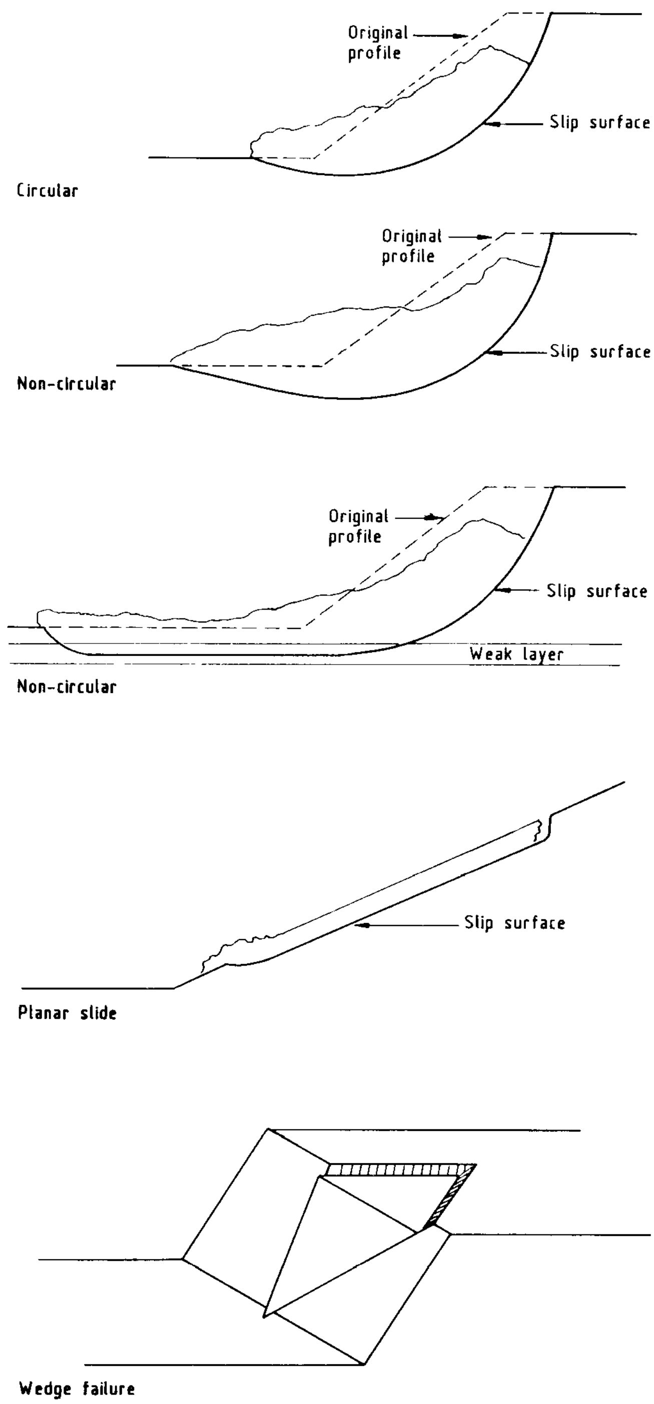
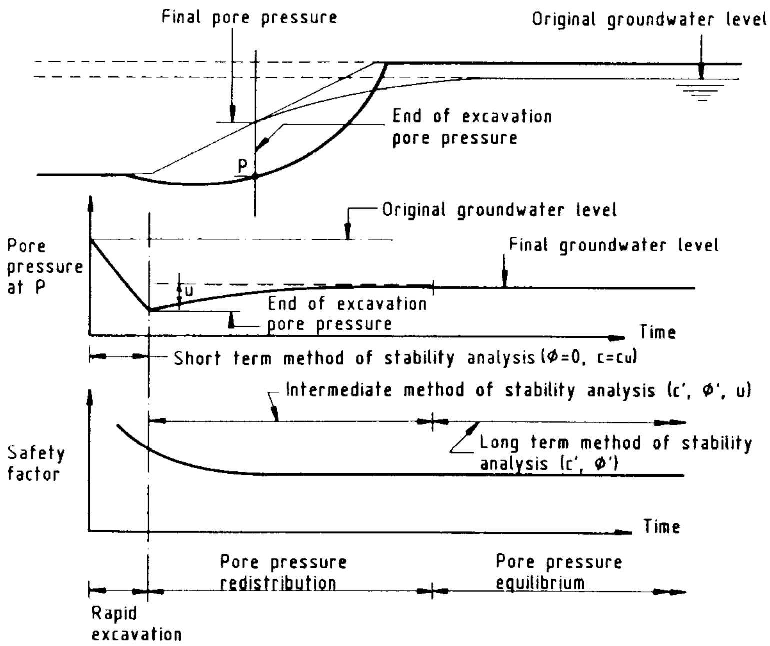
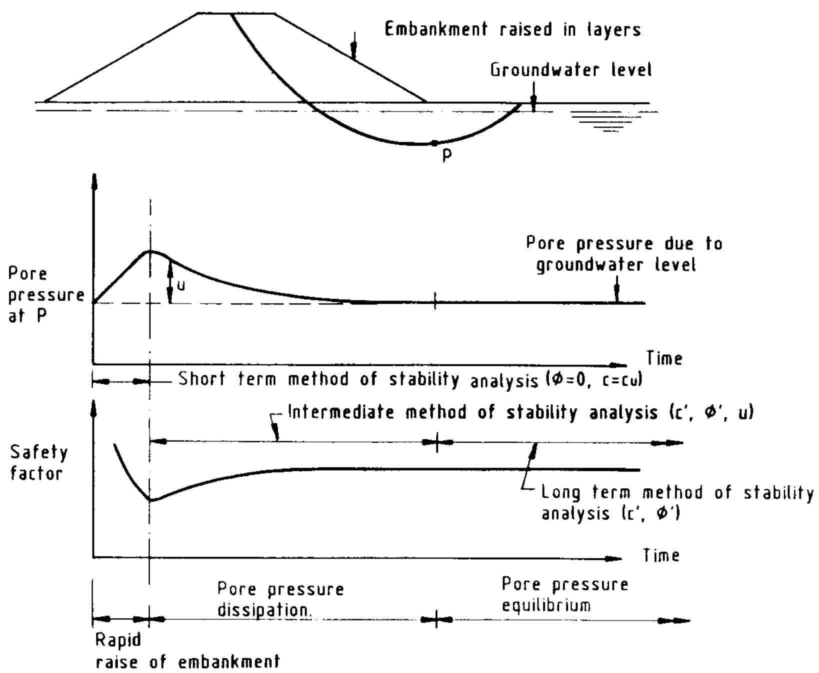


Fig. VIII-1 : Slope failure modes

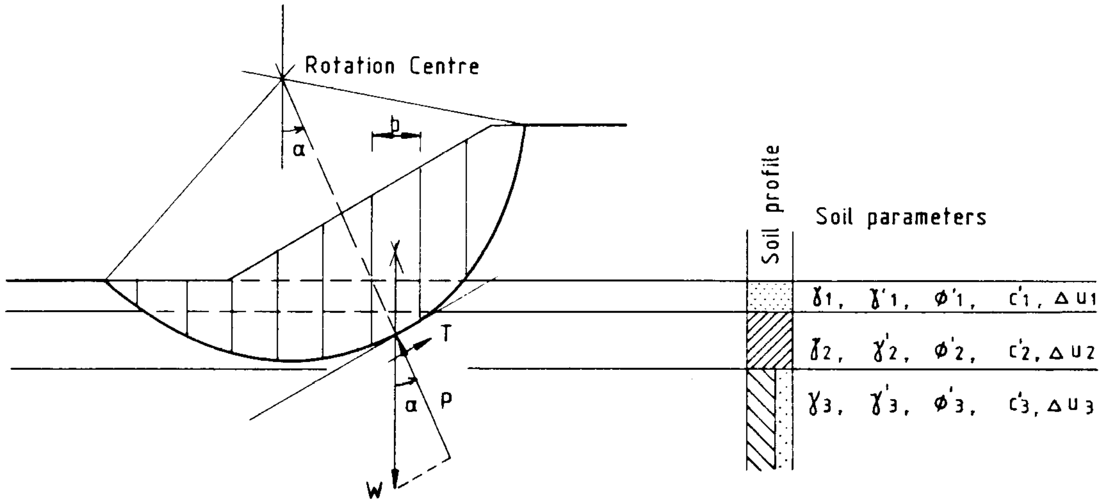


(a) Time effects in stability of cuttings



(b) Time effects in stability of embankment

Fig. VIII-2 : Short and long term stability of cutting and embankment slopes



W = weight of vertical soil strip (width = b)

$$P = W.\cos\alpha = p\ .\ \frac{b}{\cos\alpha}$$

$$T = \tau\ .\ \frac{b}{\cos\alpha}.\ (c' + (p - u)\ .\ \tan\phi')$$

<u>According to Bishop Simplified Method</u>:

$$\text{Stability factor } F = \frac{\sum\left(\dfrac{c' + (p - u)\ .\ \tan\phi'}{M}\right)}{\sum p\ .\ \sin\alpha}$$

in which: $M = \dfrac{\cos\alpha(1 + \tan\alpha\ .\ \tan\phi')}{F}$

u = pore pressure at the base of a strip including excess pore pressure Δu

Fig. VIII-3: Example of effective stress analysis of the safety factor F of a slope

APPENDIX IX PRINCIPLES OF SETTLEMENT CALCULATIONS

IX-1 STRESS DISTRIBUTION

Each settlement analysis should be based on a consideration of the distribution of stresses in the ground. Attention has to be paid to increased stresses due to adjacent structures (Fig. IX-1) or (pile) group effects (Fig. IX-2).

IX-2 IMMEDIATE SETTLEMENTS (Elastic settlements)

Settlements may be considered as elastic deformations in the case of:

- immediate or undrained settlement of clay layers
- total settlement of clean sand layers or unweathered to moderately weathered rock

For homogeneous subsoil conditions the elastic settlement S_e can be calculated from:

$$S_e = \frac{q \cdot B}{E} \cdot I$$

in which:

q = surcharge (foundation pressure) (kN/m^2)
B = width of foundation or embankment (m)
E = modulus of elasticity of the soil (undrained modulus E_u for clays) (kN/m^2)
I = influence factor (dimensionless) given by elastic theory as a function of the dimensions of the loaded area and the Poisson Ratio v of the soil. For undrained clays v = 0.5

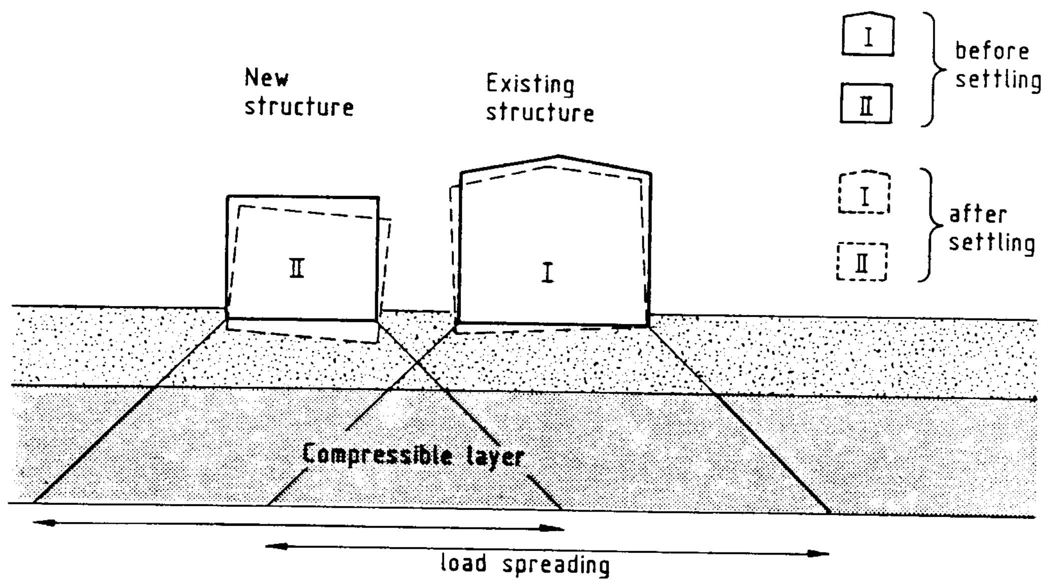


Fig. IX-1: Tilting of old and new structure due to surcharge by new structure

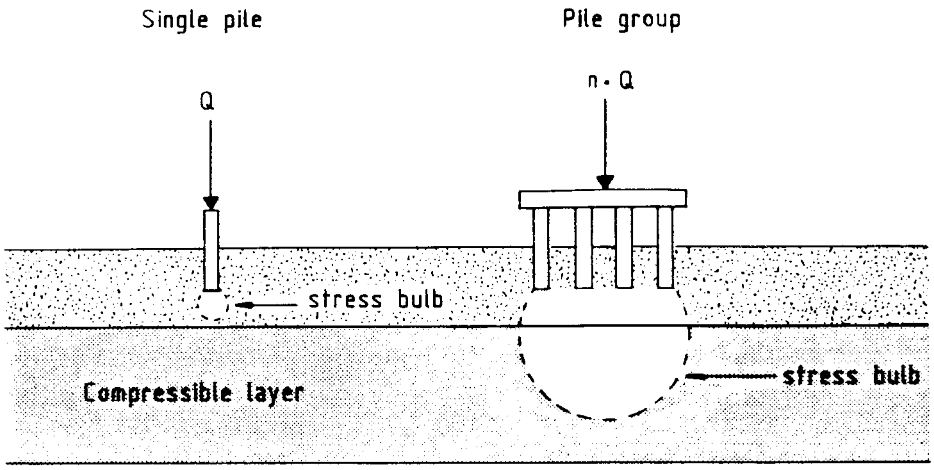


Fig. IX-2: Pile group effect: The group of n piles settles more due to higher stress increase in deep layer

IX-3 LONG TERM SETTLEMENTS

Fig. IX-3 shows the settlement versus time behaviour of low permeable saturated soils. The settlements occurring after the Consolidation Period T_e, see (IX-3.2), can be calculated from the equations presented in (IX-3.1).

Even after complete consolidation some soils, like soft clay or peat, may show additional time dependent settlement, which is called Secondary Compression.

IX-3.1 PRIMARY AND SECONDARY SETTLEMENT

Assuming 100% consolidation and referring to Fig. IX-3, the time-dependent settlement S can be calculated from:

$$S = S_p + S_c + h\left(\frac{1}{C_p} + \frac{1}{C_s} \cdot \log_{10}\ t\right) \cdot \ln \frac{\sigma_0' + \sigma_1'}{\sigma_0'}$$

in which:

- S = settlement (m)
- h = thickness of the soil layer (m)
- C_p = primary compression coefficient (-)
- C_s = secondary compression coefficient (-)
- t = time (days)
- σ_o' = original effective vertical stress at the relevant depth (kPa)
- σ_1' = increase of effective vertical stress at the relevant depth, taking into account the appropriate stress distribution (kPa)

Selecting C_p and C_s values, consideration has to be given to possible precompression (overconsolidation) effects due to geological history or preloading.

For soils showing a negligible secondary settlement, the above equation is modified to Terzaghi's equation:

$$S = h \cdot \frac{1}{C} \cdot \ln \frac{\sigma_0' + \sigma_1'}{\sigma_0'}$$

Refer to Fig. IX-3.

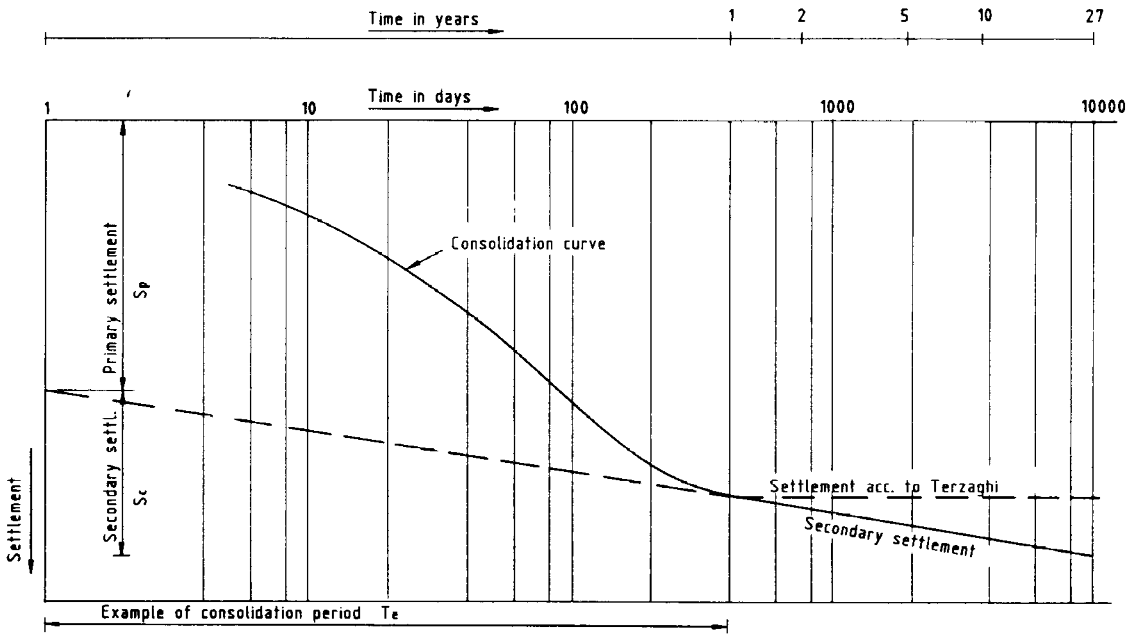


Fig. IX-3: Settlement versus time due to consolidation and creep (secondary settlement)

IX-3.2 CONSOLIDATION PERIOD

The calculation should be based on the equation:

$$T_e = \frac{1}{4} h^2 \cdot \frac{1}{C_v} \cdot A_t$$

in which:

T_e = period of time after which the consolidation process has practically ended (year),called Consolidation Period

h = thickness of the soil layer (m)

C_v = coefficient of consolidation (m^2/year)

A_t = dimensionless time factor to be selected:

A_t = 2 implies consolidation 99% completed

A_t = 1 implies consolidation 92% completed

The above equation applies to one-dimensional consolidation of a layer free-draining both upwards and downwards. If there is an impermeable boundary at either the top or the bottom of the consolidating stratum, the consolidation period will be four times as long.
In the case of three-dimensional consolidation, for example if a vertical drainage system is applied, alternative equations have to be used.

APPENDIX X DESIGN PRINCIPLES FOR SHEET PILING

X-1 GENERAL

The design shall take into account all possible failure modes (Fig. X-1). Overall stability analyses are dealt with in Appendix VIII.

The calculation of:

- stability of the toe of the wall
- bending moments in the wall
- anchor wall stability
- anchor force

should be based on the calculation of water pressure and earth pressure distribution at both sides of the wall.

The limit state earth pressures are calculated from

$$P_{active} = K_a \cdot \sigma_v' - c' \cdot 2\sqrt{K_a}$$

$$P_{passive} = K_p \cdot \sigma_v' + c' \cdot 2\sqrt{K_p}$$

An example of values of the horizontal component of the earth pressure coefficients K_a and K_p is given in Table X-A, for a case assuming:

- horizontal ground/bottom levels
- a friction between soil and wall described by a friction angle 2/3 ø.
- vertical stability (equilibrium) of the wall.

Table X-A: Earth pressure coefficients for walls

δ = 2/3 ø	ø (degrees)	K_{ph}	K_{ah}
	10.0	1.57	0.66
	12.5	1.76	0.59
	15.0	2.00	0.53
	17.5	2.26	0.48
	20.0	2.58	0.43
	22.5	2.96	0.39
	25.0	3.41	0.35
	27.5	3.96	0.32
	30.0	4.63	0.29
	32.5	5.46	0.26
	35.0	6.51	0.23
	37.5	7.84	0.20
	40.0	9.57	0.18
	42.5	11.86	0.16
	45.0	14.95	0.14

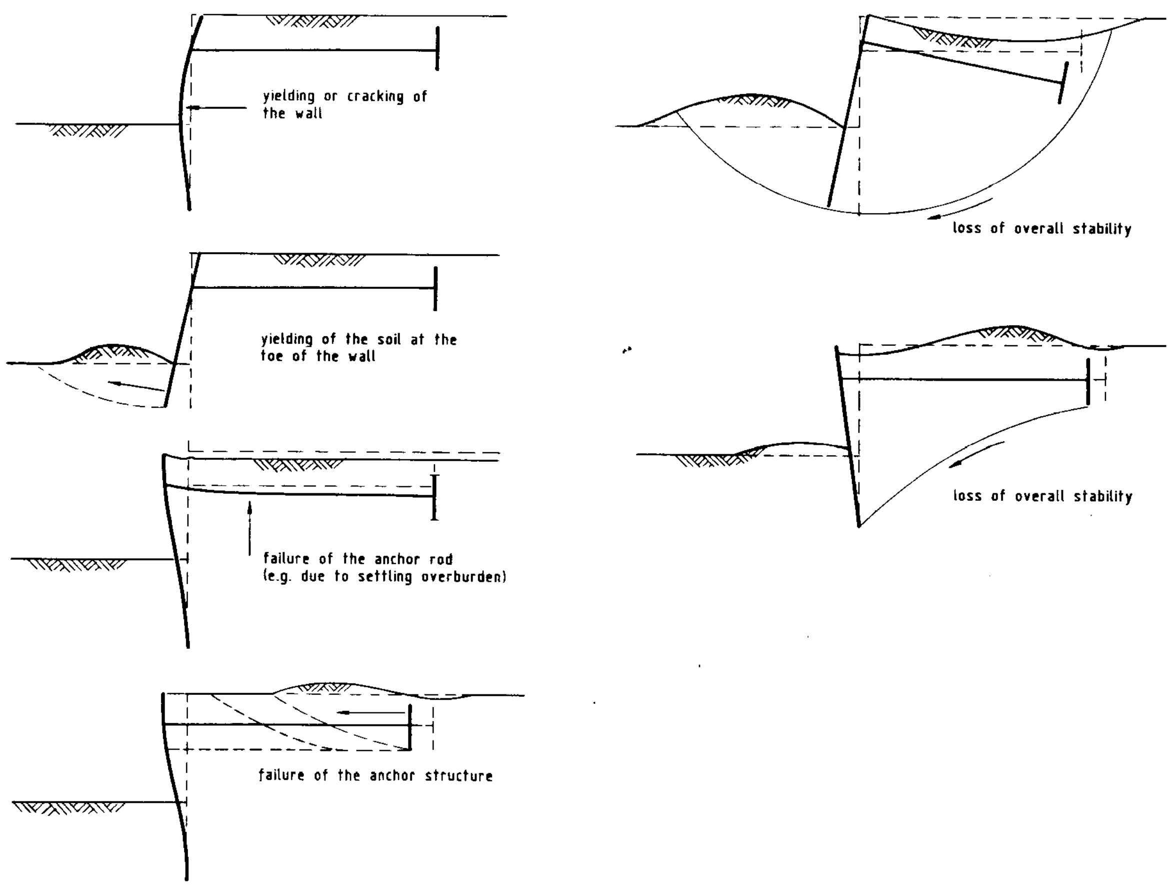


Fig. X-1: Possible failure modes of a sheet pile wall

X-2 UNANCHORED WALLS

The resulting horizontal earth and water pressure at the wall is determined as a function of depth. Subsequently the theoretical penetration depth t_o of the wall is calculated from the equilibrium of rotating moments around the toe of the wall. See Fig. X-2. From the equilibrium of horizontal forces it follows which concentrated load C is required at depth t_o. For actually creating this lateral support C the wall penetration has to be extended over a minimum depth Δt calculated from

(see Fig. X-2)

$$\Delta t = \frac{C}{2 \cdot \sigma_v'(T_0) \cdot K_{ph}}$$

As a lower limit, $\Delta t = 0.2\ t_o$ should be taken.

Application of unanchored walls generally implies large displacements of the top of the wall and large bending moments. Unanchored walls will only be an economic design in the case of a small retaining height.

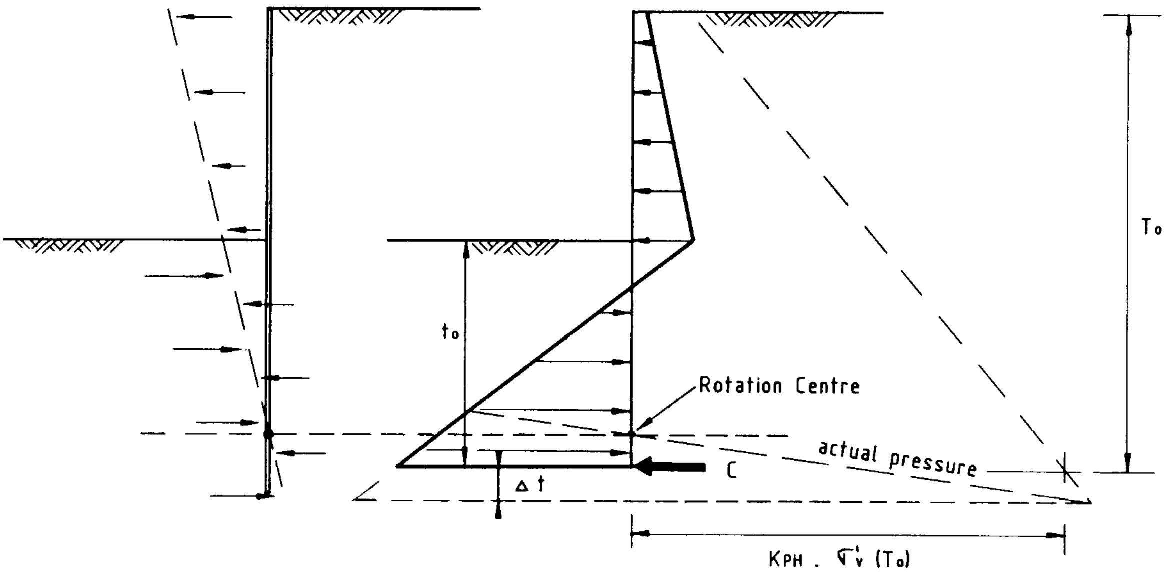


Fig. X-2: Schematized resulting lateral pressure and concentrated support force C

X-3 ANCHORED WALLS (single anchor)

Based on the resulting distributions of lateral earth and water pressures, the theoretical penetration depth t_o is calculated from the equilibrium of rotating moments around the anchor point.

Subsequently, the minimum anchor load follows from the equilibrium of horizontal forces. This minimum anchor load corresponds with active state earth pressures near the anchor, implying large anchor displacements.
The design anchor load may have to be increased, assuming passive state pressures above the anchor, in order to limit the anchor displacements.

If the designing contractor accepts a free toe support of the wall, generally an actual penetration depth of t = 1.05 - 1.15 t_o will be sufficient to create an adequate safety factor (minimum 1.5) against toe instability. However, a rather high anchor load may result from this approach.

Reduction of the anchor load and the maximum bending moments in the wall, possibly leading to a more economic design, can be achieved by creating a fixed soil support at the toe of the wall by increasing the penetration depth.
By assuming various values of a concentrated lateral force C (Fig. X-2) at a depth t_o, the distribution of bending moments and the anchor load are recalculated/optimized.
Subsequently, from the force C resulting from this optimization, the required additional penetration depth Δt is calculated from

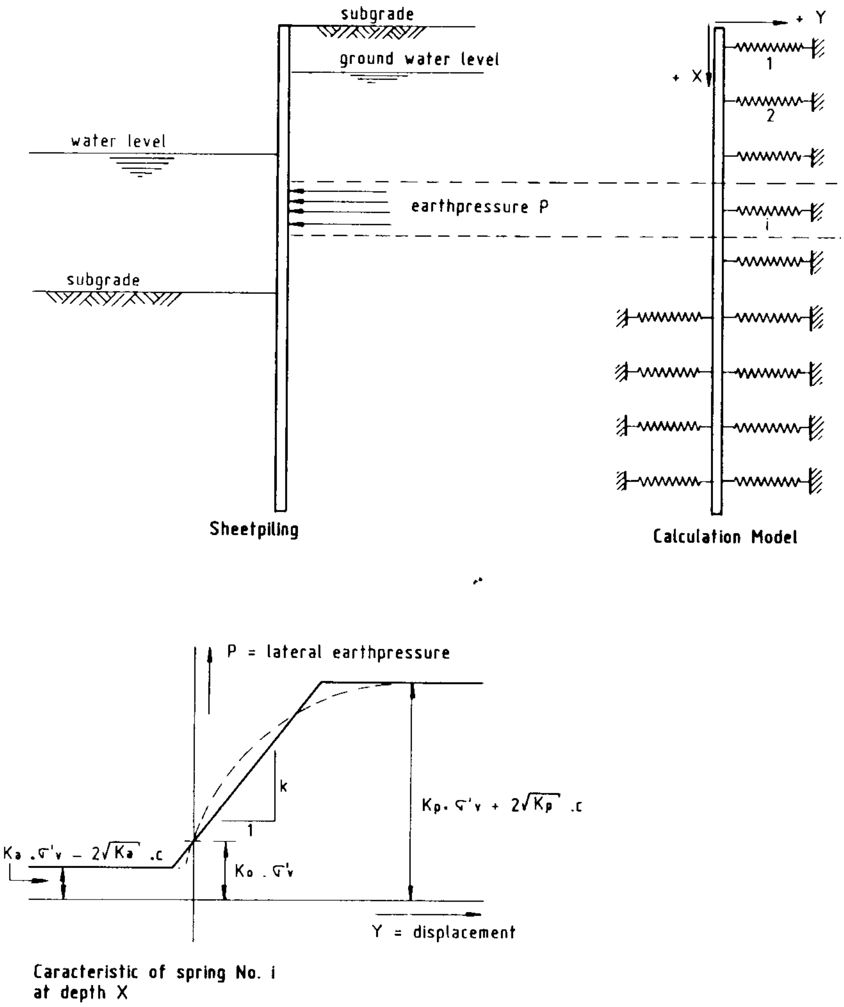
(see Fig. X-2)

$$\Delta t = \frac{C}{2 \cdot \sigma_v{}'(T_0) \cdot K_{ph}}$$

Special attention has to be paid to effects of (differential) deformations of the wall and the anchoring structure.

X-4 DEFORMATIONS

For an accurate design, particularly of multi-anchored walls, deformation/stiffness effects must be considered in the calculation.
This can be realized by schematizing the sheet pile wall to a 'beam on elastic foundation' (Fig. X-3). In this case the model is a vertical beam supported by horizontal springs. By defining springs with a bi-linear or non-linear characteristic, both the stiffness of the soil and the limit state earth pressure, see (X-1), are taken into account. The calculations require specialized computer software.



___	linear calculation model
-----	more accurate calculation model
σ_v'	= effective vertical soil stress at depth X
k	= spring stiffness (deformation modulus)
K_a (K_p)	= active (passive) earth pressure coefficient
K_o	= coefficient of earth pressure at rest

k and K values depending on soil condition

Fig. X-3: Calculation principle for flexible walls and piles

APPENDIX XI SURFACE ROCK ENGINEERING

XI-1 GENERAL

XI-1.1 INTRODUCTION

Clear distinction should be made between rock mass and rock (material). Rock is defined as the solid material from which the earth crust is built, composed of mineral grains and with an unconfined compression strength over 1 MN/m^2. Rock is considered to be intact. Rock as it occurs in-situ, i.e. including discontinuities such as joints, bedding planes, weathering features, etc., is called rock mass.

Note 1: In practice, different reports use different meanings for the terms intact rock, solid rock, bed rock, fresh rock, etc. In each report it should be clarified what the definition is of the applied terms.

Note 2: The in-situ behaviour of rock mass is dominated by the discontinuities. Together with the brittleness of rock, the influence of discontinuities forms the main difference between the behaviour of soil and rock.

XI-1.2 ROCK MASS PROPERTIES AND CLASSIFICATION

Main features affecting the engineering behaviour of rock masses are:

- frequency, orientation, nature and condition of discontinuities and their possible infill with soil-like material
- weathering characteristics
- climatologic conditions
- hydrological situation, e.g. hydraulic pressures of water accumulated in faults or joints
- creep settlements of weak rocks
- solution of rock (e.g. gypsum and limestone), which may cause cavities
- heave by chemical reactions, e.g. oxidation of pyrite
- soft or hard seams, dips, shelves or other hazards causing foundation contact problems
- sink holes
- seismic hazards
- reaction to possible future stress relief (e.g. due to excavation)

To prepare the design of structures on or in rock, all features affecting the behaviour of the rock have to be described. In the literature, many different classification systems for rock are presented. It is recommended to adhere to the classification system presented in BS 5930, para. 44.

XI-1.3 WEATHERING

All rocks near the ground surface are weathered to some degree. In general, weathering processes weaken the rock. Ultimately weathering may lead to the development of residual soil near the surface, grading through a (deep) transition zone into unweathered rock.

Two types of weathering can be distinguished:

- chemical weathering (e.g. solution of limestone, transformation of feldspars in granite into clay),
- mechanical weathering (e.g. erosion or frost action).

Chemical weathering tends to dominate in warm, humid climates, in colder climates the speed of chemical reactions is generally low.

Both the weathering state, the weatherability and the environmental situation of the rock mass shall be investigated, in view of the short term and long term engineering behaviour of the rock mass.

XI-2 FOUNDATIONS

XI-2.1 GENERAL

In most cases the ultimate bearing capacity of all except the weakest rocks will exceed the strength of the concrete of the foundation. Therefore, settlement (and not bearing capacity) is generally the critical factor.

In any case attention shall be paid to:

Good contact between the foundation (or pile) and the (irregular and/or sloping) surface of the bearing rock. Refer to Fig. XI-1.

XI-2.2 FOOTINGS AND SLAB FOUNDATIONS

XI-2.2.1 Bearing capacity

For estimating purposes, the allowable foundation pressure p on unweathered rock can be calculated as the smaller value of:

$p = \sigma_{uc}$ = unconfined compressive strength

p = function of the number of joints and faults, expressed in the RQD-value (Rock Quality Description), according to the table below (for chemically unweathered rock):

RQD (%)	p (kN/m^2)
100	30,000
90	20,000
75	12,000
50	6,500
25	3,000
0	1,000

This approximation includes a safety factor of 3. The Rock Quality Description is defined in BS 5930.

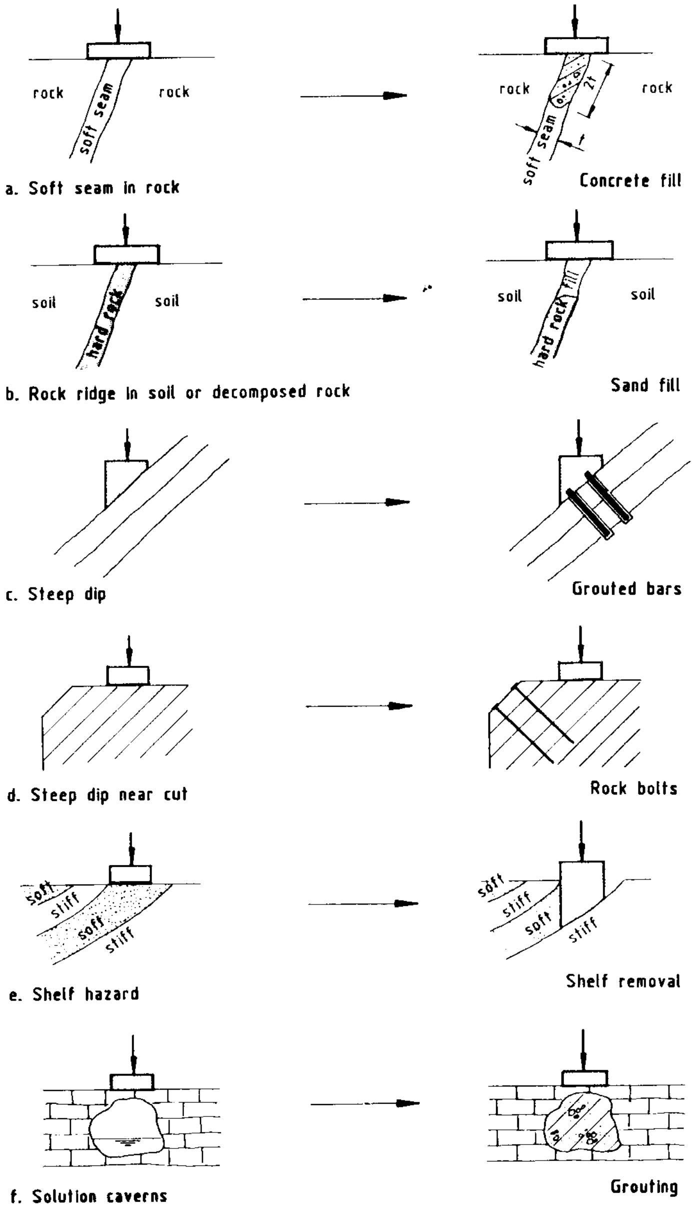


Fig. XI-1: **Rock foundations contact problems and possible solutions (Sowers, 1979)**

For weathered rock which is homogeneous in its weakness the ultimate bearing capacity may be calculated in the same way as that of soil, with the method given in Appendix II. Because it is often difficult to define the friction angle ø, the allowable bearing pressure p is often calculated from:

$$p = \frac{1}{F} \cdot C_u \cdot N_c$$

in which:

F = safety factor, minimum 3

C_u = undrained cohesion, defined as 0.5 x unconfined compressive strength

N_c = bearing factor (refer to Appendix II)

However, in many cases the strength of the foundation layers is not uniformly distributed. Depending on the type of heterogeneities or structural defects, various calculations have to be made, taking into account each possible failure mode. Refer to Fig. XI-2.

XI-2.2.2 Settlements

For more or less homogeneous conditions settlement analyses may be performed using elastic theory. Critical in this analysis is the determination of a representative elasticity modulus of the rock mass (E_m). This mass modulus may be correlated to the elasticity modulus of intact rock samples (E_{50}), according to the table below:

RQD (%)	E_m/E_{50}
90- 100	0.8- 1.0
75- 90	0.5- 0.8
50- 75	0.2- 0.5
25- 50	< 0.2
0- 25	< 0.2

Note: E_{50} = elasticity modulus of an intact rock sample at 50% of its unconfined compressive strength

E_m could be measured in-situ by pressure meter tests or plate loading tests.

Generally, settlement analysis can be performed using elastic theory. However, for clayey and very soft rocks, extra settlement due to creep should also be considered.

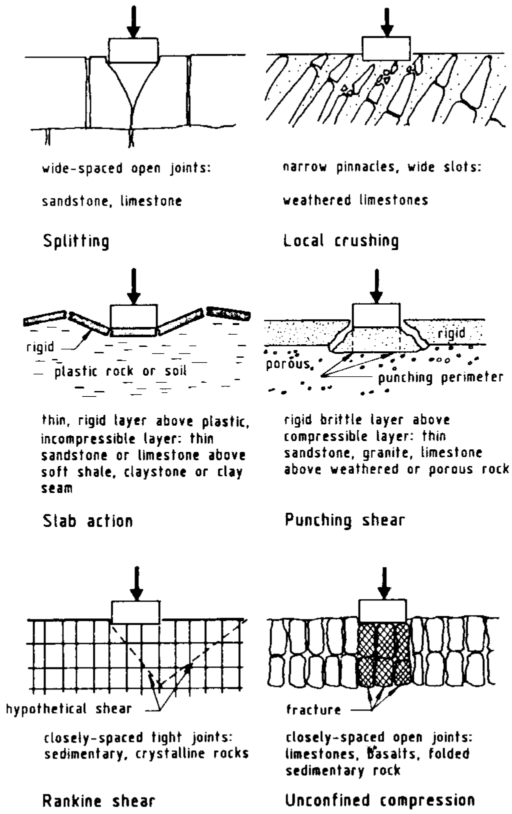


Fig. XI-2: Various failure modes of shallow foundations

XI-2.3 PILED FOUNDATIONS

XI-2.3.1 Socketed piles

For preliminary design purposes the ultimate bearing capacity of a socketed pile may be calculated from:

$P_{ult} = 4.5 \cdot \sigma_{uc}$ (B)

$f_{ult} = 0.15 \cdot \sigma_{uc}$ (A)

in which:

P_{ult} = ultimate bearing pressure at the pile base (kPa)

f_{ult} = ultimate friction along the rock socket (kPa)

σ_{uc}(B) = unconfined compressive strength of the rock below the pile base (kPa)

σ_{uc}(A) = average unconfined compressive strength of the rock over the height of the socket (kPa)

To determine the allowable pile load, minimum safety factors F_s = 6 for the base resistance and F_s = 3 for the friction should be used.

Pile settlements may be analysed using elastic theory. For preliminary design purposes, single pile base settlement may be estimated from:

$$S = \frac{Q}{D \cdot E_m}$$

in which:

S = pile settlement (m)

Q = pile load (kN)

D = pile base diameter (m)

E_m = elastic modulus of the rockmass (kPa)

The actual pile settlement depends, amongst other things, on the distribution of the load between pile base resistance and friction.

XI-2.3.2 Driven piles

Particularly in areas with hard bearing rock, the rock head topography should be surveyed carefully prior to construction as severe risk of pile damage may exist (see Fig. IX-3).

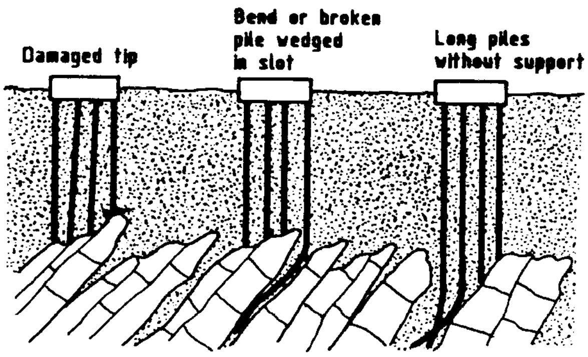


Fig. XI-3: Pinnacles and slots influencing pile capacity

The bearing capacity of the piles is only derived from end bearing. For preliminary design purposes the ultimate bearing capacity may be calculated from:

P_{ult} = 3 σ_{uc}

in which:

P_{ult} = ultimate bearing pressure (kPa)

σ_{uc} = unconfined compressive strength of the bearing rock (kPa)

When applying the above equation, a safety factor of at least 6.0 should be adopted to determine the allowable load.

When driven into weak rock the pile bearing capacity may be calculated using the calculation method for piles in soil (refer to Appendix III).

For pile settlement estimation, reference is made to (XI-2.3.1).

XI-3 SLOPE ENGINEERING

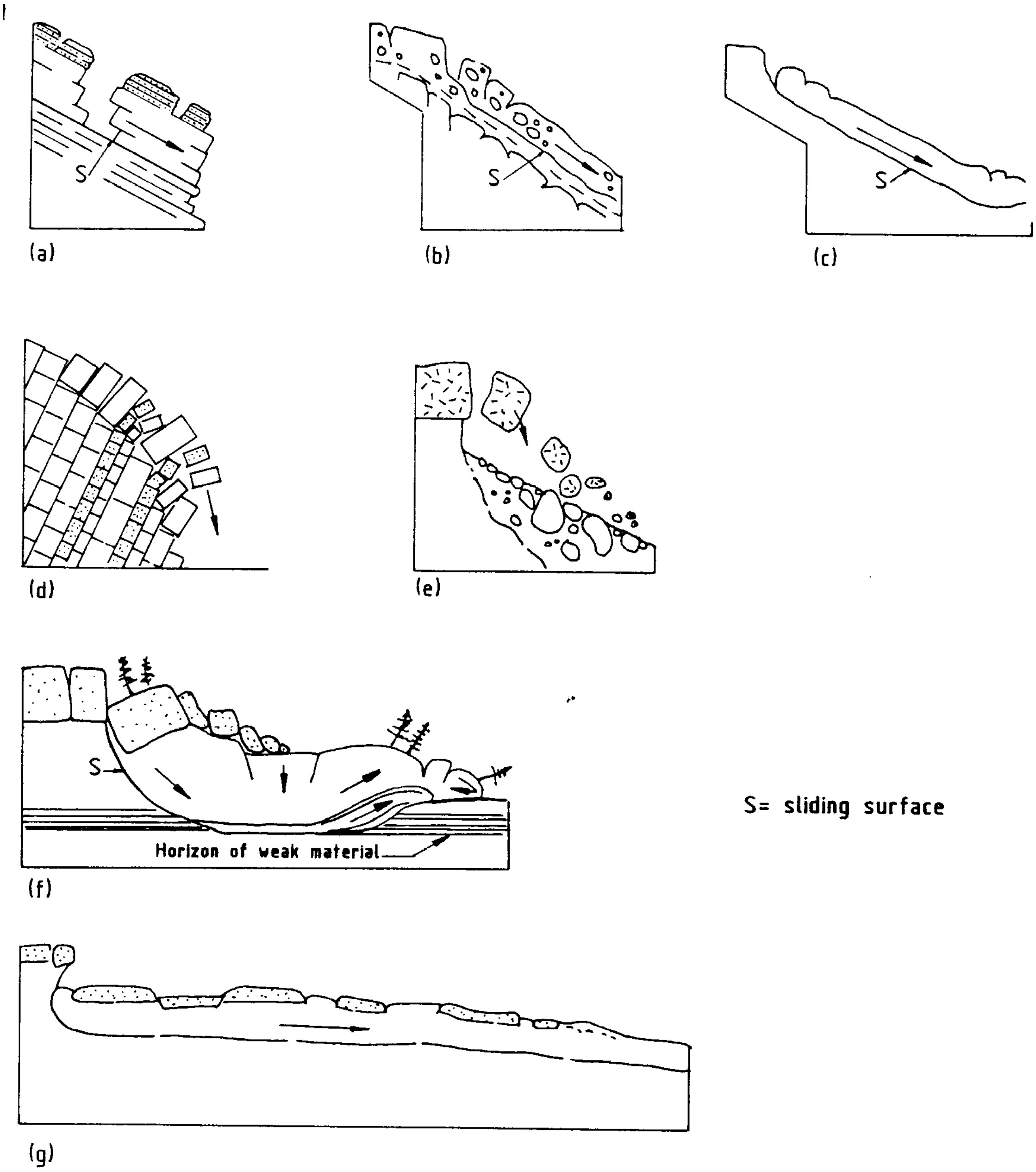
The stability of rock slopes depends primarily on the orientation and shear resistance in discontinuities such as fissures, joints and fault zones.
The shear resistance depends on:

- roughness and waviness of the two rock surfaces at the discontinuity
- shear strength (cohesion) of any infilling material
- possible water pressures in the fault

Additionally, possible frost action, weathering effects, effects of blasting and seismic forces shall be taken into account.

Depending on, amongst other things, the orientation of discontinuities, various failure modes shall be considered in the slope stability analysis. Refer to Fig. XI-4.

The principles of the calculations to be made for the planar-slide case are indicated below (Fig. XI-5). For rotational failure calculations, refer to Appendix VIII.



(a) = slide on planar surfaces in rock
(b) = slide on scree or weathered mantle
(c) = slide on clays or soft sediment
(d) = toppling failure
(e) = rock falls
(f) = rotational failure
(g) = flow side

Fig. XI-4: Various modes of rock slope instability

Figure XI-5 shows a pre-existing geological discontinuity oriented at an angle of dip α to the horizontal.

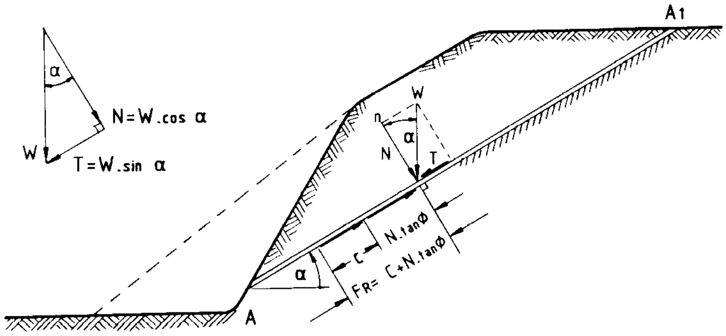


Fig. XI-5 Slope stability problem involving sliding over a geological discontinuity.

Driving force is $F_D = T = W \cdot \sin \alpha$. This driving force causes shearing in the rock discontinuity A-A_1. The resisting force F_R to shear is:

F_R = (N-U) . tan ø + C

= (W . cos α - U) tan ø + C

where:

T = tangential (shear) force in the plane of discontinuity

W = weight of the sliding rock mass over the shear plane

α = angle of dip of slope

ø = angle of friction

N = W . cos α

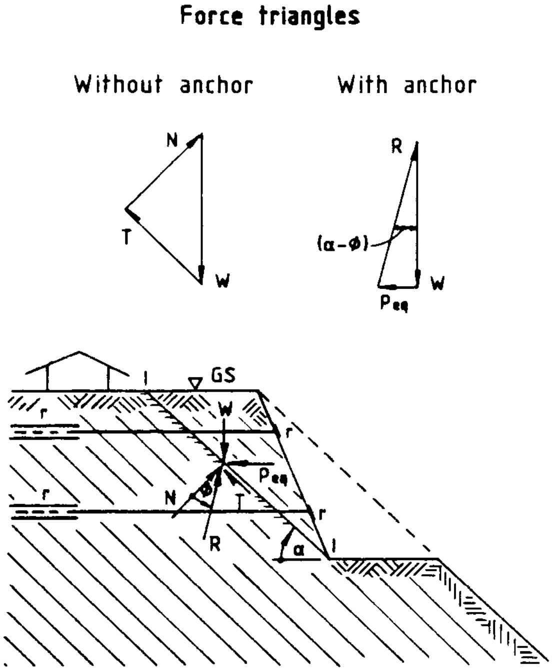
C = total cohesive (resisting) force along the shearing surface (A - A_1)

U = possible water pressure in faults etc.

The safety or stability factor of the rock slope is expressed by:

$$F.S. = \frac{F_R}{F_D} = \frac{(W \cdot \cos\alpha - u)\tan\phi + C}{W \cdot \sin\alpha}$$

If the factor of safety proves to be unacceptable, the slope must be re-designed adopting a different angle of slope, or reinforced by means of rock anchors (Fig. XI-6), or buttressed by means of a retaining wall. Provision for drainage facilities may decrease pore-water pressure in rock as well as uplift forces.



1-1 = potential failure line
r-r = rock anchor
W = weight of ruptured rock wedge above plane 1-1
ø = angle of friction along plane 1-1
N = normal force
T = tangential shear resistance
R = reaction
P_{eq} = anchor force

Fig. XI-6 : Anchoring of a rock slope