DRAFT FOR DEVELOPMENT

DD ENV 1997-3:2000

Incorporating Corrigendum No.1

Eurocode 7: Geotechnical design —

Part 3: Design assisted by fieldtesting

ICS 91.010.30; 93.020

National foreword

This Draft for Development is the official English language version of ENV 1997-3:1999.

This publication is not to be regarded as a British Standard.

It is being issued in the Draft for Development series of publications and is of a provisional nature. It should be applied on this provisional basis so that information and experience of its practical application may be obtained. This document does not have a parallel British Standard and, therefore, it has been published for use in the United Kingdom (UK) without any National Application Document.

ENV 1997-3:1999 results from a programme of work sponsored by the European Commission to make available a common set of rules for the structural and geotechnical design of buildings and civil engineering works. The full range of codes covers the basis of design and actions, the design of structures in concrete, steel, composite construction, timber, masonry and aluminium alloy, and geotechnical and seismic design.

Comments arising from the use of this Draft for Development are requested so that the UK experience can be reported to the European organization responsible for its conversion into a European Standard. A review of this publication will be initiated 2 years after its publication by the European organization so that a decision can be taken on its status at the end of its three-year life. The commencement of the review period will be notified by an announcement in *Update Standards*.

According to the replies received by the end of the review period, the responsible BSI Committee will decide whether to support the conversion into a European Standard, to extend the life of the prestandard or to withdraw it. Comments should be sent in writing to the Secretary of BSI Subcommittee B/526/3, Soil tests, at 389 Chiswick High Road, London W4 4AL, giving the document reference and clause number and proposing, where possible, an appropriate revision of the text.

A list of organizations represented on this subcommittee can be obtained on request to its secretary.

Cross-references

The British Standards which implement international or European publications referred to in this document may be found in the BSI Standards Catalogue under the section entitled “International Standards Correspondence Index”, or by using the “Find” facility of the BSI Standards Electronic Catalogue.

Summary of pages

This document comprises a front cover, an inside front cover, the ENV title page, pages 2 to 146, an inside back cover and a back cover.

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This Draft for Development, having been prepared under the direction of the Sector Committee for Building and Civil Engineering, was published under the authority of the Standards Committee and comes into effect on 15 April 2000

Amendments issued since publication

Amd. No.	Date	Comments
12051 Corrigendum No.1	October 2000	Corrections to page numbers throughout contents

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EUROPEAN PRESTANDARD

PRÉNORME EUROPÉENNE

EUROPÄISCHE VORNORM

ENV 1997-3

July 1999

ICS 91.080.01; 93.020

English version

Eurocode 7: Geotechnical design - Part 3: Design assisted by fieldtesting

Eurocode 7: Calcul géotechnique - Partie 3: Calcul sur la base d'essais en place

Eurocode 7: Entwurf, Berechnung und Bemessung in der Geotechnik - Teil 3: Felduntersuchungen für die geotechnische Bemessung

This European Prestandard (ENV) was approved by CEN on 30 July 1997 as a prospective standard for provisional application.

The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into a European Standard.

CEN members are required to announce the existence of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

CEN members are the national standards bodies of Austria, Belgium, Czech Republic, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

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Ref. No. ENV 1997-3:1999 E

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FOREWORD

Objectives of the Eurocodes

(1) The structural Eurocodes comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

(2) They are intended to serve as reference documents for the following purposes:

a) As a means to prove compliance of building and civil engineering works with the essential requirements of the Construction Products Directive (CPD)
b) As a framework for drawing up harmonised technical specifications for construction products.

(3) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship, needed to comply with the assumptions of the design rules.

(4) Until the necessary set of harmonised technical specifications for products and for methods of testing their performance is available, some of the Structural Eurocodes cover some of these aspects in informative annexes.

Background to the Eurocode programme

(5) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonised technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the "Structural Eurocodes".

(6) In 1990, after consulting their respective Member States, the CEC transferred work of further development, issue and updates of the Structural Eurocodes to CEN and the EFTA Secretariat agreed to support the CEN work.

(7) CEN Technical Committee CEN/TC 250 is responsible for all Structural Eurocodes.

Eurocode programme

(8) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991 Eurocode 1 Basis of design and actions on structures
EN 1992 Eurocode 2 Design of concrete structures
EN 1993 Eurocode 3 Design of steel structures
EN 1994 Eurocode 4 Design of composite steel and concrete structures
EN 1995 Eurocode 5 Design of timber structures
EN 1996 Eurocode 6 Design of masonry structures
EN 1997 Eurocode 7 Geotechnical design
EN 1998 Eurocode 8 Design of structures for earthquake resistance
EN 1999 Eurocode 9 Design of aluminium alloy structures.

(9) Separate sub-committees have been formed by CEN/TC 250 for the various Eurocodes listed above.

(10) This part of the Structural Eurocode for Geotechnical design, is being issued by CEN as a European Prestandard (ENV) with an initial life of three years.

(11) This Prestandard is intended for experimental practical application in the design of the building and civil engineering works covered by the scope as given in 1.1.2 and for the submission of comments.

(12) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future action.

(13) Meanwhile, feedback and comments on this Prestandard should be sent to the Secretariat of sub-committee CEN/TC250/SC7 at the following address:

NNI
P.O.Box 5059
NL-2600 GB Delft
The Netherlands

or to a national standards organisation.

National application documents

(14) In view of the responsibilities of authorities in member countries for the safety, health and other matters covered by the essential requirements of the CPD, certain safety elements in this ENV have been assigned indicative values which are identified by [..]. The authorities in each member country are expected to assign definitive values to these safety elements.

(15) Many of the supporting standards, including those giving values for actions to be taken into account and measures required for fire protection, will not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document giving definitive values for safety elements, referencing compatible supporting standards and giving national guidance on the application of this Prestandard will be issued by each Member State or its Standard Organisation. This Prestandard should be used in conjunction with the National Application Document valid in the country where the building and civil engineering works is to be constructed.
It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works are located.

Matters specific to this prestandard

(16) This prestandard is intended to serve as a reference document for the use of field tests for geotechnical design. It covers the execution and interpretation of the most commonly used field tests. This prestandard aims at ensuring that adequate quality is reached in the execution of field tests and their interpretation.

(17) In the framework of European Standardization, Eurocode 7 Part 1 on design of geotechnical structures has been established. The link between the general requirements for the design such as stated in ENV 1997-1 and the existing standards, codes and other types of generally accepted documents for operating field investigations is covered by Eurocode 7 Part 3: "Geotechnical design assisted by field tests". Eurocode 7 Part 3 in particular addresses some of the requirements of ENV 1997 - Part 1, especially section 3: "Geotechnical data".

(18) ENV 1997-3 does not replace standards for equipment and performance of different test methods, but provides basic requirements for such standards.

(19) Section 2 of ENV 1997-3 gives general requirements with respect to planning of field and laboratory investigations. This section serves as a common section of both Eurocode 7 Parts 2 and 3.

1 GENERAL

1.1 Scope

1.1.1 Scope of Eurocode 7

(1)P Eurocode 7 applies to the geotechnical aspects of the design of buildings and civil engineering works. It is subdivided into various separate parts. (see 1.1.2)

(2)P Eurocode 7 is concerned with the requirements for strength, stability, serviceability and durability of the structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(3)P Eurocode 7 shall be used in conjunction with ENV 1991-1 Eurocode 1 "Basis of Design and Actions on Structures". Part 1: "Basis of Design", which establishes the principles and requirements for safety and serviceability, describes the basis for design and verification and gives guidelines for related aspects of structural reliability.

(4)P Eurocode 7 gives the rules to calculate actions originating from the ground such as earth pressures. Numerical values of actions on buildings and civil engineering works to be taken into account in the design are provided in ENV 1991 Eurocode 1 "Basis of Design and Actions on Structures" applicable to the various types of construction.

(5)P In Eurocode 7 execution is covered to the extent that is necessary to indicate the quality of the construction materials and products which should be used and the standard of workmanship on site needed to comply with the assumptions of the design rules. Generally, the rules related to execution and workmanship are to be considered as minimum requirements which may have to be further developed for particular types of buildings or civil engineering works and methods of construction.

(6)P Eurocode 7 does not cover the special requirements of seismic design. Eurocode 8, "Design provisions for earthquake resistance of structures" provides additional rules for seismic design which complete or adapt the rules of this prestandard.

1.1.2 Scope of ENV 1997-3

(1)P In addition to ENV 1997-1 the scope of ENV 1997-3 is to provide for a number of commonly used field tests:

a) requirements for the equipment and test procedures;
b) requirements for the reporting and the presentation of test results;
c) interpretation of test results.

(2)P Part 3 shall serve as a link between the design requirements of Part 1 and the results of a number of field tests. Therefore part of the scope is to give:

d) examples on how to derive values of geotechnical parameters from the test results.

(3)P ENV 1997-3 shall be used in conjunction with ENV 1997-1.

1.1.3 Limitations

(1)P The derivation of parameter values is dedicated primarily to the design of pile and spread foundations as elaborated in the annexes B, C, D, and E of ENV 1997-1.

(2)P The scope of ENV 1997-3 does not cover the following:

– the assessment of characteristic values;

– environmental geotechnics, chemical investigations or the environmental impact of structures;

– geohydrological tests e.g. pumping tests.

1.2 References

(1)P This European Prestandard incorporates by dated or undated reference, provisions from other standards. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this Prestandard only when incorporated in it by amendment or revision.

ENV 1991-1:1994	Eurocode 1 Basis of design and actions constructures Part 1 Basis of design
ENV 1997-1:1994	Eurocode 7 Geotechnical design Part 1 General rules ENV 1997-2:1998 Eurocode 7 Geotechnical design Part 2 Design assisted by laboratory testing
ISO 3898:1997	Basis for design of structures. Notations. General symbols

1.3 Distinction between Principles and Application Rules

(1)P Depending on the character of the individual paragraphs, distinction is made in this prestandard between Principles and Application Rules.

(2)P The Principles comprise:

– general statements and definitions for which there is no alternative, as well as;

– requirements and analytical models for which no alternative is permitted unless specifically stated.

(3)P The Principles are preceded by the letter P.

(4)P The Application Rules are examples of generally recognised rules which follow the Principles and satisfy their requirements.

(5)P It is permissible to use alternative rules different from the Application Rules given in this Eurocode, provided it is shown that the alternative rules accord with the relevant Principles.

1.4 Definitions

1.4.1 Defenitions common to all Eurocodes

(1)P The terms used in common for all Eurocodes are defined in ENV 1991-1.

1.4.2 Definitions used in Eurocode 7

(1)P For terms which are specific to Eurocode 7 reference is made to 1.5.2 of ENV 1997-1.

1.4.3 Definitions used in ENV 1997-3

(1) In sections 3 to 14 specific definitions relating to that section are given.

(2) For the purpose of this prestandard the following terms apply:

1.4.3.1 derived value: value of a geotechnical parameter obtained by theory, correlation or empiricism from test results. Derived values form the basis for the selection of characteristic values of ground properties to be used for the design of geotechnical structures, in accordance with 2.4.3 of ENV 1997-1.

1.4.3.1.1 Concept of derived values

(1) The concept of 'derived values' is elaborated as a way to link test results to geotechnical parameters. From the test results the values for geotechnical parameters for the use in analytical methods and coefficients for the use in semi-empirical or direct methods may be arrived at through:

— results of field tests	-----> through correlations -->	to geotechnical parameter values
	-->	to coefficients in direct methods
— results of lab. tests	-->	to geotechnical parameter values
	-----> through correlations with other tests -------->	to geotechnical parameter values

These values of the geotechnical parameters and/or coefficients, arrived at through, for example correlations, are called 'derived values'.

(2) ENV 1997-3 provides a set of examples of derived values for geotechnical parameters. From this the characteristic and the design values according to the requirements of ENV 1997-1 have to be established.

(3) The concept of 'derived values' is as follows: assume a homogeneous zone of ground governing the behaviour of a geotechnical structure. Assume that two types of field tests are carried out (see Fig. 1.1): five Cone Penetration Tests (CPT) and five pressuremeter tests (gives P_{LM}), and assume five laboratory tests to establish the undrained shear strength. From the five (over the depth of the layer averaged) CPT values and P_{LM} values from pressuremeter tests, the following sets of derived values are established through certain correlations with the undrained shear strength:

C_u scale in kPa

CPT:	--------*---*-----*-*----*--	c_u values through a correlation with q_c
PMT:	-------*--*---**---*--	c_u values through a correlation with P_{lm}
LAB:	----------*-**--*--*--	c_u from laboratory tests, theoretically evaluated

Figure 1.1 Concept of derived values

(4) From these three sets of derived values for the undrained shear strength of a homogeneous soil mass, the characteristic value to be used in the design has to be assessed.

1.4.3.1.2 Correlations

(1) The examples in the annexes,to subclauses #.7 of this prestandard, are based on various correlations obtained from the literature. These correlations may correlate a geotechnical parameter derived value either with a measured value, for example the q_c-value of a CPT, or with a corrected value, for example the q_t-value of a CPTU being the q_c-value corrected for the measured excess pore pressure.

(2) Apart from this, the correlation may connect a geotechnical parameter derived value either with the mean value of the measured/corrected value or a conservative estimate of the measured/corrected value (see Fig 1.2).

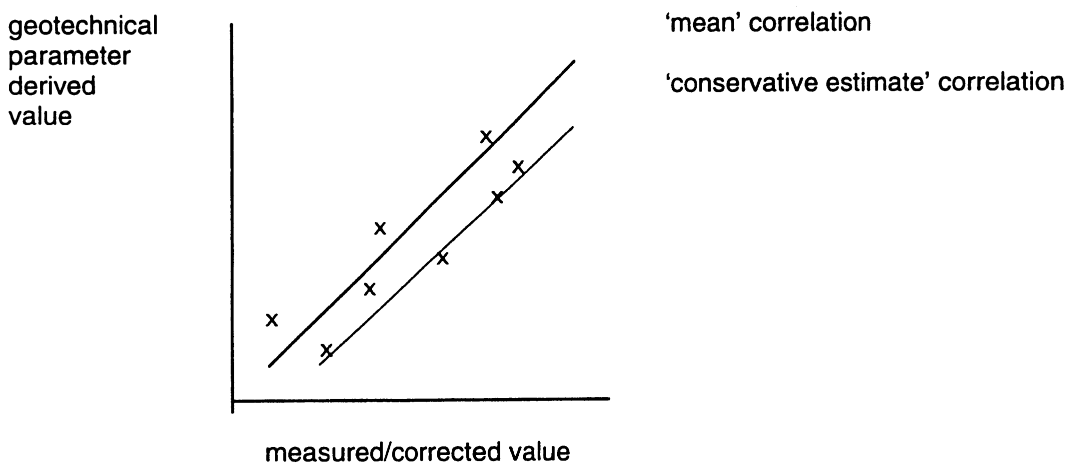


Figure 1.2 Types of correlation

(3) Each subclause #.7 of the tests covered by ENV 1997-3 gives examples on how to derive values of geotechnical parameters or coefficients in direct methods. In the examples in the annexes, the subclauses #.7 of ENV 1997-3 refer to, either 'mean' or 'conservative' correlations are used. Sometimes the correlation can even be meant as a correlation of a geotechnical parameter and the characteristic value of the measured/corrected value. Normally the type of correlation is unknown. Also the theory used to determine a soil parameter value may differ between references and are not always presented in the references. In evaluating the examples in the annexes this should be kept in mind. If the type of correlation is known, this is indicated in the annexes.

1.4.3.2 excess pore pressure: the pore water pressure over and above the equilibrium pore pressure at the end of the consolidation

1.5 Symbols and units

1.5.1 Symbols common to all Eurocodes

(1)P The symbols used in common for all Eurocodes are defined in ENV 1991-1 "Basis of design".

1.5.2 Symbols and units used in Eurocode 7

(1)P The symbols commonly used in ENV 1997-3 are defined in each section. Other symbols are defined where they are used locally in the text.

(2)P The units recommended for geotechnical calculations are defined in 1.6 of ENV 1997-1.

(3) The symbols follow the rules given in ISO 3898.

1.6 The link between ENV 1997-1 and ENV 1997-3

(1) The flow chart shown below demonstrates the link between design and field and laboratory tests. Design is covered by ENV 1997-1; the parameter values part is covered by ENV 1997-2 and ENV 1997-3.

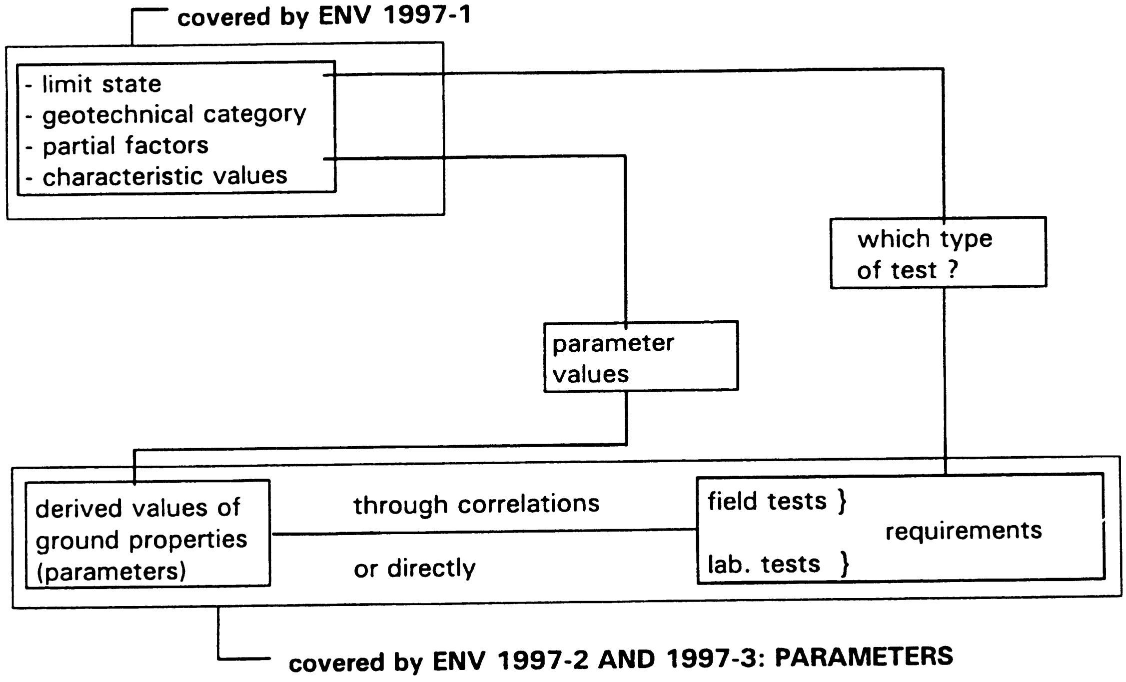


Figure 1.3 Flowchart geotechnical design

2 PLANNING OF SITE INVESTIGATIONS

2.1 General

(1)P Site investigations consist of desk studies, field and laboratory investigations and all other investigations performed in order to establish knowledge of the ground; soil, rock and groundwater conditions, and to determine the properties of the soil and rock. Site investigations may also include appraisal of existing constructions, tunnels, embankments and slopes and the environmental impact of the project.

(2) Ground investigations are normally performed in phases depending on the questions raised during planning, design and construction of the actual project, see 3.2.1 of ENV 1997-1. The following phases are treated separately in this section:

- preliminary investigations for positioning and preliminary design of the structure, see 2.3;
- investigations for design and construction, see 2.4;
- control investigations, see 2.5.

(3) The provisions in this document are planned primarily for geotechnical category 2 projects (see 2.1 of ENV 1997-1) and based on the fact that the results from investigations recommended in one phase are available before the next phase is started. In cases where all investigations are performed at the same time, both 2.3 and 2.4 should be considered simultaneously.

(4) The ground investigation requirements for category 1 projects are normally limited as the verifications often will be based on local experience. For category 3 projects the amount of investigations required will normally be at least the same as indicated for category 2 projects in the following sections. In addition complementary investigations related to the circumstances that place the project in category 3 may apply.

(5)P The composition and the extent of each phase of the investigations shall be based on the following:

- the topographical, geological and hydrogeological conditions on the site and pertinent available information about them;
- the type and design of the construction, e.g. type of foundation, improvement method or retaining structure, location and depth of the construction;
- the local experience and climate conditions.

(6)P When composing the investigation programme both for field and laboratory tests, the knowledge from desk studies of the following documents, if available, and the results of a site examination shall be accounted for (see 3.2.3 of ENV 1997-1):

- topographical maps;
- old city maps describing the previous use of the site;
- geological maps and descriptions;
- engineering geological maps;
- geohydrological maps and descriptions;
- geotechnical maps;
- aerial photos and previous photo interpretations;
- aerogeophysical investigations;
- previous investigations at the site and in the surroundings;
- previous experiences from the area;

(7)P the use of preliminary investigations shall be considered.

(8) The extent of direct field investigations and sampling with subsequent classification and laboratory testing depends on the geology, type of structure and the local experience.

(9)P The preliminary investigations shall be planned in such a way that adequate data are obtained, when applicable, concerning:

- the overall stability of the area;
- suitable positioning of the structure;
- possible foundation methods and ground improvements;
- possible effects to neighbouring buildings;
- preliminary costs for foundations and ground improvements;
- preliminary costs for provisional constructions;
- proposals for design investigation.

For planning and execution of preliminary investigations reference is made to 2.3.

(10)P Investigations for the design and construction phase shall provide sufficient information to the following questions, when applicable:

- location of construction;
- foundation methods and levels;
- erosion protection;
- protection against swelling and shrinkage;
- ground improvements or other stabilizing measures;
- foundation design;
- design of temporary constructions;
- excavatability;
- driveability for piles;
- drainages and filters;
- frost susceptibility;
- methods and order of construction/operations;
- requirements for the selection of filling material;
- slope inclination;
- existing hindrance such as old constructions and service pipes and cables;
- durability of construction material in the ground.

For planning and execution of investigations for design and construction reference is made to 2.4.

(11) Annex A provides a flow chart which can be used for the selection of a proper fieldinvestigation method for the various soil investigation stages.

(12)P Where construction elements are applied in the geotechnical structures of which the durability may be affected by the surrounding soil and goundwater, the aggressiveness of the soil and groundwater shall be determined, to enable provisions when applicable, (see 2.3 of ENV 1997-1).

(13) To determine the aggressiveness of the ground and groundwater a selection of chemical analyses often should be undertaken and the results compared to available experience on the aggressiveness to various construction materials and protection techniques. Chemical analyses often require groundwater sampling and thus special provisions in the borehole.

(14)P The results of the desk studies and the site inspection shall be considered, when locating the various investigation points. Investigations shall be targeted at points representing the variation in ground conditions for soil, rock and groundwater.

(15)P In cases where more than one type of investigation shall be made at a certain location (e.g. CPT-test and piston sampling), the boreholes shall be placed at least 2 m apart.

(16)P The depth of investigation shall include all relevant layers for the actual geotechnical design.

(17)P The selection of the location of boreholes and test pits shall take account of possible adverse effects on the integrity of the construction to be built and neighbouring structures and shall minimize the probability of harmfull environmental effects.

(18)P The class and number of samples shall be based on the aim and status of the soil investigations, on the geology of the site and on the complexity of the geotechnical structure and of the construction to be designed.

(19)P Sampling shall be done in accordance with sections 12 and 13.

(20) Where lamination or fine stratification of the subsoil can influence the geotechnical design, consideration should be given to the use of continuous boring with open tube sampler or core drilling. Additional samples may be taken with e.g. a short open tube sampler, a fixed piston sampler or a block sampler.

(21) In case there is no stratification or no influence from stratification, in coarse soils, samples from auger or bailer may be sufficient. In clays and organic soils high quality samples should be taken with special equipment.

(22) Laboratory investigations for classification purposes and determination of the properties of the soil or rock should be performed as relevant for the project as a part of the total ground investigation programme. Guidance on the choice of investigation methods in order to obtain various properties of soil and rock may be found in paragraphs (23) to (26).

(23) Suitable routine classification for soil samples with various degrees of disturbance is presented in table 2.1. The routine tests are generally performed in both the preparatory and the design investigation phases. However, in the preparatory phase a limited number of samples may often be investigated.

Table 2.1: Routine soil classification tests

Soil type	Clayey soils			Silty soils			Sandy, gravely soils	
Sample quality	Undis-turbed	Dis-turbed	Re-moul-ded	Undis-turbed	Dis-turbed	Re-moul-ded	Dis-turbed	Re-moul-ded
Parameters								
Soil class	X	X	X	X	X	X	X	X
Water content,(w)	X	(X)	(X)	X	(X)	(X)	(X)	(X)
Density, (D)	X	(X)	-	X	(X)	-	-	-
(D_{max}), (D_{min})	-	-	-	(X)	(X)	(X)	X	X
Atterberg limits	X	X	X	X	X	X	-	-
Grain size distribution	(X)	(X)	(X)	X	X	X	X	X
Undrained shear strength, (c_u)		-		(X)	-	-	-	-
Sensitivity, (S_t)	X	-	-	-	-	-	-	-
	X		-					

X = normal to determine;
(X) = possible to determine;
\- = not applicable.

(24) In addition to these routine tests for soils other classification tests for certain cases may be performed e.g. activity, density of grain material and organic content.

(25) Laboratory tests to determine various geotechnical parameters for design purposes are indicated in table 2.2.

Table 2.2: Laboratory tests for the determination of certain geotechnical parameters

Geotechnical parameter	Type of soil					
	Gravel	Sand	Silt	NC clay	OC clay	Peat organic clay
Oedometer modulus (E_{oed})	OED**)	OED**)	OED	OED*)	OED	OED*)
Effective shear strength (c'), (ϕ')	TX, DSS, (DST)	TX, DSS, (DST)	TX, DSS, (DST)	TX, DSS, (DST)	TX, DSS, (DST)	TX, DSS, (DST)
Undrained shear strength (c_u)	-	-	TX, SIT, DSS, (DST)	TX, SIT*), DSS, (DST)	TX, SIT*), DSS, (DST)	TX, SIT*), DSS, (DST)
Permeability (k)	PT(C), SV	PT(C), SV	PT(C), PT(F)	PT(F)	PT(F)	PT(F)

*) Indicate investigations normally performed in the preliminary phase, at least to some extent.
**) Requires special device.

Abbreviations of laboratory tests:

OED	Oedometer test
TX(	Triaxial test
PT(F)	Permeability test (falling head)
PT(C)	Permeability test (constant head)
DST	Direct shear test
SIT	Strength index tests
SV	Sieving
DSS	Direct simple shear test

(26) Suitable routine laboratory tests for rock samples are presented in table 2.3. These tests normally give the necessary basis for the description of the rock material.

Table 2.3: Routine rock tests

Type of test	**Type of rock** ***)			
	1	**2**	**3**	**4**
Geological classification	X	X	X	X
Unit weight, (γ)	X	X	X	X
Water content, (w)	X	X	X	X
Porosity, (n)	X	X	X	X
Uniaxial compressive strength test	X	X	X	X
Point load test	x	X	X	X
Compressibility, (E)	X	X	X	X

***) Group according to ENV 1997-1, Annex E, Table E.1.

(27) The classification of core samples will normally comprise a geological description, the core recovery, the Rock Quality Designation (RQD), the degrees of induration, weathering and fissuring. For weathering classification see annex L. In addition to the routine tests in table 2.3 for rocks other tests may be selected for different purposes, e.g. density of grains, wave velocity, Brazilian tests, shear strength of rock and joints, slake durability tests, swelling tests and abrasion tests.

(28) The properties of the rock mass including the layering and fissuring normally may indirectly be investigated by compression and shear strength tests along joints. In weak rocks complementary tests in the field or with large scale laboratory tests on block samples may be made.

2.2 Definitions

(1)P dynamic sounding or probing: driving a penetrometer into the ground with a hammer or a percussion drilling machine while measuring the resistance in e.g. blows or seconds for a certain amount of penetration.

(2)P field investigations for geotechnical purposes: geophysical investigations, penetration tests, in situ testing, soil sampling, rock sampling, groundwater measurements, large scale tests and monitoring.

(3) Examples of the various types of field investigations are:

- geophysical investigations (e.g. seismic profiling, ground penetrating radar, resistivity measurements and borehole logging);
- penetration testing (e.g. CPT, SPT, static soundings and dynamic probings);
- in situ testing (e.g. pressuremeter tests, dilatometer tests, plate load tests, field vane tests and permeability tests);
- soil and rock sampling for description of the soil or rock and laboratory tests;
- groundwater measurements to determine the groundwater table or the pore pressure profile and their fluctuations;

- large scale tests in order to determine e.g. the bearing capacity and behaviour of elements directly;
- monitoring of the behaviour of different constructions e.g. movement of anchored retaining structures, settlement of foundations, pore water dissipation beneath embankments etc.

(4)P investigation point: the co-ordinates (*x*,*y*,*z*) where one or a number of different tests or a sampling is performed.

(5)P large scale tests: tests to determine directly the bearing capacity and behaviour of plates, piles, ground anchors or the transmissivity of the ground by pumping tests.

(6)P measuring While Drilling (MWD): technique of measuring a number of boring parameters during soil-rock sounding. Both manual and automatic recording exists.

(7) During drilling for example the penetration rate, rate of rotation, bit force, torsional moment, flush pressure and volume of flushing agent can be measured.

(8)P penetration test: all different tests where a rod provided with a tip is pushed or driven into the ground while the resistance to penetration is recorded.

(9) Penetration tests thus include both static and dynamic penetration tests and sounding methods such as CPT, WST, SPT, DP and soil-rock sounding (SR) in accordance with definitions in the following sections.

(10)P preliminary investigation: investigations performed before designing and specifying an investigation programme, including desk studies and site inspection.

(11)P soil or rock layer: layer of normally uniform material with similar properties, extending over a certain area in the ground.

(12)P soil-Rock Sounding: dynamic probing through possible soil into the rock with a percussion drilling machine and normal rock drilling tools and possibly MWD-technique.

(13)P static sounding: pushing or rotating a statically loaded penetrometer through the soil while the resistance is measured in e.g. kN or for the weight sounding test in halfturns per 0,2 m of penetration.

2.3 Preliminary investigations for positioning of a structure and preliminary design

(1)P A preliminary soils investigation for positioning of the structure and preliminary design shall supply estimates of soil data concerning:

- type of soil or rock and their stratification;
- thickness and relative density or strength of the soil;
- groundwater table or pore pressure profile;
- preliminary strength and deformation properties for soils and rock;
- the occurrence of contaminated ground or groundwater that might be hazardous to health or the durability of construction material.

(2)P In cases where the overall stability of the site may be of concern, a preliminary investigation shall be made.

(3) The slope stability investigation should be performed in the assumed most unfavourable section and the surrounding area where previous failure may have occurred. The volume to be investigated should cover at least a depth down to 5 m below potential slip surfaces.

(4) A slope stability investigation in soil should comprise: surveying, penetration tests, sampling, in situ tests, groundwater measurements and laboratory testing.

(5) Rock slope investigations in addition should comprise a site inspection with mapping of discontinuities and determination of the strength of joints.

(6)P In the preliminary investigation phase the investigation points shall be distributed over the area, in such a way that an adequate picture is derived of the variability of the site with regard to the stratification and quality of the ground and of the groundwater situation.

(7)P When parts of the soil and/or the structure are particularly complex the preliminary investigation shall be focussed on the concerned part of the building site.

(8) The influence on or effect of surrounding constructions shall be considered when defining the preliminary investigation programme.

(9) The extend of the preliminary soils investigation should as a minimum contain three investigation verticals equally distributed in a respective cross-section of the building site. This investigation may comprise penetration tests, in situ testing, boring for sampling (in the weakest soil) and ground water measurements where the most unfavourable groundwater level may be expected.

2.4 Investigations for design and construction

(1)P When planning investigations for design and construction account shall be taken of 3.2.3 of ENV 1997-1. The investigations for design and construction shall provide the necessary information for the construction works, the ground and groundwater conditions. The investigations shall be established for the ground below and around the construction site as far as the behaviour of the ground may adversily affect the construction works.

(2)P In cases where the preliminary investigations have not clearly shown that the overall stability is satisfactory, complementary investigations shall be performed during the design investigation phase.

(3)P The design investigations for the overall stability shall have such a composition and amount of investigation points that necessary stability analyses may be made and stabilization measures may be designed.

(4)P The composition of the ground investigation programme and location of the investigation points shall consider the size of the area, the topography, the geology, the groundwater conditions, the type and design of the construction and the type and location of neighbouring structures.

(5) The investigated area should extend into the neighbouring area for a distance of at least 1,5 times the expected excavation depth or depth of the soil layer that can generate settlements in the neighbouring area.

(6) When applicable, investigations in the design phase should comprise penetration tests, borings and/or test pits for sampling, in situ tests and groundwater measurements.

(7) The number of investigation points should be extended if it is deemed necesarry to obtain an accurate insight in the complexity and the variability of the ground at the building site. Where ground conditions are relatively uniform, a wider spacing or less investigation points may be sufficient. In either case this choice should be justified.

(8)P For identification and classification of soils, at least one boring with sampling shall be available. Samples shall be obtained from every separate soil layer influencing the behaviour of the actual structure.

(9) In cases of inhomogeneous soil layers, organic soils or when high quality samples are required, samples should be taken at least every 1 m in one boring.

(10) The depth of investigations should be extended to all strata that will be affected by the project. For dams, weirs, excavations below groundwater level, and where dewatering work is involved, the depth of exploration should also be selected as a function of the hydrogeological conditions. Slopes and steps in the terrain should be explored to depths below any potential slip surface.

(11) The groundwater pressure should be measured to a depth of at least 3 m below anticipated foundation level both in high and low points of the area or equal to the expected height of the phreatic surface above the foundation level.

2.5 Control Investigations

(1)P In order to check that the ground conditions, the delivered construction materials and the construction works correspond to those presumed or ordered a number of checks and additional tests shall be made during construction and execution of the project, when relevant.

(2) The following general control measures may apply:

- inspection of excavations;
- complementary site investigations;
- check of ground profile and properties;
- measurements of groundwater level or pore pressure and their fluctuations;
- measurements of the behaviour of neighbouring constructions, services or civil engineering works;
- measurements of the behaviour of the actual construction.

(3)P The results of the control measures shall be compiled, reported and checked against the design requirements. Decisions shall be taken based on the checks.

2.6 Reporting of site investigations

(1)P The results of a geotechnical investigation shall be reported and evaluated according to 3.4 of ENV 1997-1.

(2)P All site investigations shall be reported in such a way that the results may be checked and re-evaluated.

(3)P For identification and quality assurance purposes all investigation records, logs, samples and presentations shall contain the following information:

- name of company executing the test;
- identification of the actual site or area;
- identification number of the job or commission;
- borehole number;
- type of test.

(4)P In addition the following information shall be given on the field and laboratory investigation records in combination with the information required for each specific investigation method as indicated in sections 3 to 14:

- date of investigation;
- ground level;
- method of boring and borehole diameter;

- type of test methods used with reference to actual standards;
- calibration data for the actual measuring equipment;
- any deviations from the requirements of this prestandard, other relevant standards or recommendations;
- the signature of the operator in charge, or the field manager.

(5)P The ground investigation report shall include data indicating the basis for the survey of the site.

(6)P The factual ground investigation results shall be compiled in a report, with plans, in such a way that the total amount of basic information can be used in the evaluation of the results. The location of all investigation points shall be clearly indicated.

(7)P The evaluation of geotechnical information according to 3.4.2 of ENV 1997-1 shall be based on all relevant basic information available at the actual phase of the investigation.

(8)P Proposals for complementary investigations shall aim at clarifying the ground conditions in all the areas involved for the actual project and surrounding structures which are influenced by the project.

(9) Such proposals should account for the variations in the soil layer sequence, the variations in groundwater level or pore pressure and variations in the ground properties obtained from the investigations performed.

2.7 Evaluation of site investigations

2.7.1 Evaluation of the preliminary investigations

(1)P The evaluation of the ground conditions in the preliminary investigation phase for positioning of the structure and the preliminary design shall make use of all relevant information from the desk study and the investigations performed at the site, related to the actual project.

(2) Geological maps and descriptions of the site, penetration test results, in situ test results, classification of soil and rock samples, laboratory test results and groundwater measurements should be used to identify and describe the soil layers, the rock mass and the groundwater conditions of the total area involved in the project.

(3) The evaluation should be summarized in the report, presenting the factual investigation data according to paragraph 2.6(6)P.

(4) A preliminary derivation of ground parameter values for a preliminary design can be made for identified layers and can, for example, be expressed in terms of density, water content, penetration resistance, strength and deformation properties and pore pressures.

2.7.2 Evaluation of investigations for design and construction

(1)P The evaluation of the ground conditions in a design and construction phase shall

make use of all relevant information from the actual and earlier phases to describe the situation in relation to the actual project.

(2)P The influence from or on surrounding structures, services or civil engineering works shall be considered in the evaluation so that all necessary data are obtained.

(3) When deriving the boundary between different soil layers, soil and rock and the groundwater level a rectilinear interpolation between the investigation points may normally be sufficiently accurate, unless there is substantial heterogeneity.

(4) Evaluation of the unit weight may be based on density measurements on undisturbed soil samples or core samples in rock. In cases where only disturbed samples have been obtained the unit weight may be estimated from in situ density tests or from compiled local experiences especially considering the degree of saturation, the content of coarse material and the natural scatter in density.

(5) The unit weight may also be estimated from local classification charts on the basis of results from CPT or dilatometer tests. In rock, and soils within a casing the density may be estimated from calibrated geophysical loggings as an alternative.

(6)P When deriving ground parameter values for design purposes from site investigations the type of ground, the type of construction, the total and local experiences from similar design cases as well as the scatter of the test results shall be considered.

(7) When deriving ground parameter values, the profile of the ground should be schematised into a representative sequence of layers of soil and rock on the basis of a geotechnical identification and classification and fieldtests. Within the distiguised layers the strenght and deformation properties should not vary considerably.

(8) For the assesment of the strenght and deformation properties of the distinguished layers in the schematised profile of the soil or rock, the following should be taken into consideration:

- direct measurement of the properties from field vane tests, plate load tests or pile load tests;
- laboratory measurements of certain parameters at certain stress conditions, e.g. triaxial tests, direct shear tests or oedometer tests;
- transformation of measured data to soil properties by using compiled correlations with different parameters, e.g. angle of shearing resistance, modulus of elasticity, relative density from e.g. cone penetration tests, dynamic probing tests, SPT-tests, dilatometer tests or pressuremeter tests. Since correlations of this kind normally are limited to only one investigation method and one type of experiment it is not recommended to use them for cross correlations e.g. to transform N_{60} to q_c and then to N' or E_m;
- direct use of measured parameter values in a design method based on compiled experiences, e.g. calculating the bearing capacity and settlements from pressuremeter tests, cone penetration tests or standard penetration tests.

3 CONE PENETRATION AND PIEZECONE TESTS - CPT(U)

3.1 General

(1)P The cone penetration test (CPT) consists of pushing vertically into the soil, at a relatively slow and constant rate, a penetrometer which comprises a series of rods terminated by a penetrometer tip comprising a cone and a cylindrical shaft and measuring the penetration resistance of the cone as well as, possibly, the local friction on a sleeve located in the cylindrical shaft.

(2)P The piezocone test (CPTU) is a CPT which includes the measurement of the pore water pressure during penetration at the level of the base of the cone.

(3)P This section covers mainly electric penetrometers.

(4)P The tests shall be carried out in accordance with a method that complies with the essential requirements given in this section.

(5)P The test method used shall be reported in detail with the test results.

(6) The test method may be reported by reference to a standard.

(7)P Any deviation from the requirements given below shall be justified and in particular its influence on the results shall be commented upon.

(8) Experience of deviations exists with respect to:

- size of the cone;
- use of mechanical or hydraulic penetrometers.

3.2 Definitions

For the purpose of this prestandard the following definitions related to CPT and CPTU apply:

(1)P penetrometer tip: The main parts of the penetrometer tip are defined in figure 3.1.

(2)P cone resistance(q_c): the measured axial force Q_c acting on the cone divided by the total area of the base of the cone A_c;

(3)P local unit side friction (f_s): the measured frictional force Q_s acting on the sleeve divided by its area

(4)P friction ratio (R_f): f_s/q_c where q_c and f_s are determined at the same depth, expressed as a percentage

(5)P friction index (I_f): q_c/f_s where q_c and f_s are determined at the same depth.

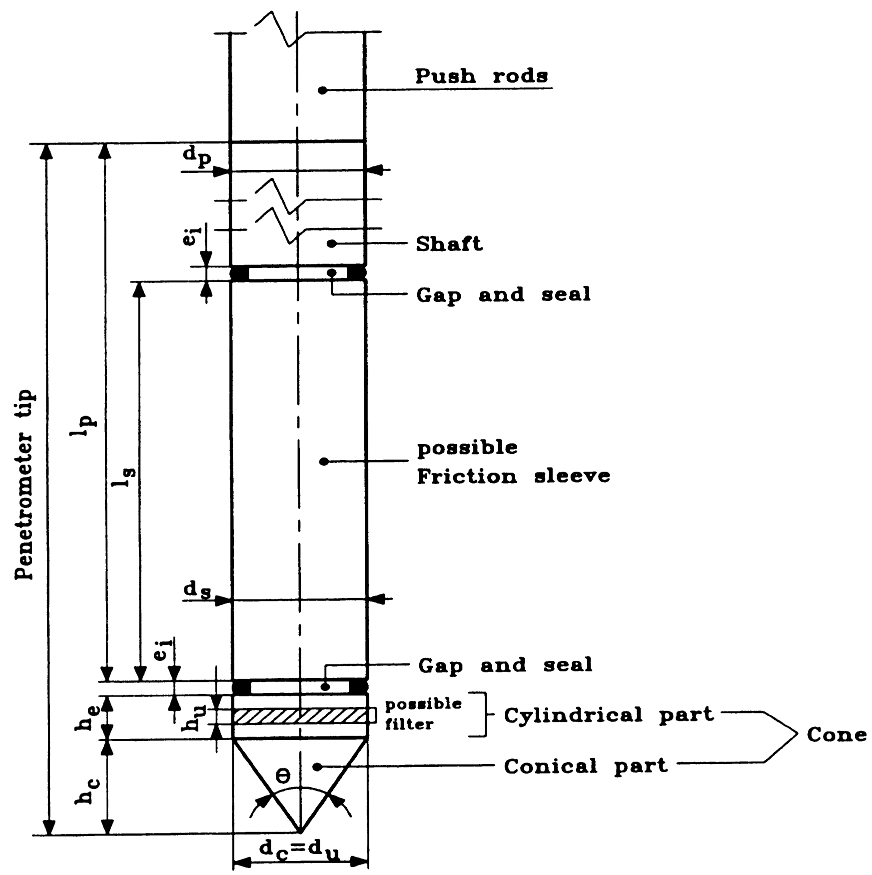


Figure 3.1: Scheme of an electric penetrometer tip

(6)P For CPTU the following additional parameters are defined.

(7)P cone area factor (*a*): A_N/A_c, where A_N is the net area of the cone (see figure 3.2)

(8)P penetration pore pressure (*u*): the pore pressure measured during penetration on the cylindrical part of the cone, just above the conical part of the cone (see figure 3.1)

(9)P generated pore pressure (Δu): $u - u_0$, where u_0 is the pore pressure existing in the ground before the penetration test at the level of the cone;

(10)P total (corrected) cone resistance (q_t): $q_c + u\,(1-a)$

(11)P pore pressure ratio B_q: $\Delta u\,/\,(q_t - \sigma_{v0})$, where σ_{v0} is the total vertical stress existing in the ground before the penetration test at the level of the cone.

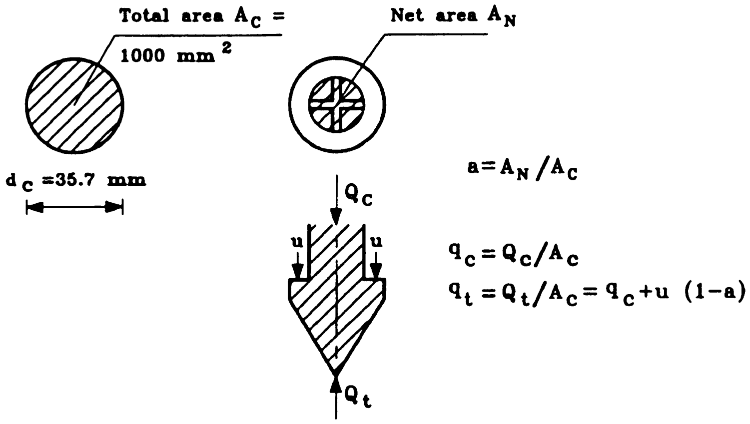


Figure 3.2: Definitions for a piezocone test (CPTU)

3.3 Equipment

3.3.1 Apparatus

(1)P The apparatus shall comprise:

- thrust machine;
- push rods;
- penetrometer tip;
- measuring/recording equipment.

(2)P The thrust machine shall push the rods into the soil at a constant rate of penetration. It shall be stabilised in such a way that it does not move significantly during the pushing action.

(3)P The push rods shall be assembled tightly in order to form a rigid-jointed series with a continuous straight axis.

(4) In order to prevent buckling, guides and/or casings should be provided for the free part of the push rods above the soil, in very soft layers situated above a highly resistant layer and for the part of the push rods in water.

(5) Before loading, push rods should be straight so that the maximum deflection of a point on a 1 m push rod, from a straight line through the ends, does not exceed 1 mm for the 5 lowest push rods and 2 mm for the remainder.

(6)P The penetrometer tip terminates the series of rods. It shall have a cone and shaft with the same axis as the series of push rods.

(7)P When the penetrometer tip includes a friction sleeve, it shall have the same axis and shall be located immediately above the cone.

(8)P The apex angle of the cone θ shall be 60°.

(9) The assessment of the derived values in 3.7 is based on the nominal total area of the base A_c = 1000 mm^2.

(10) When cones having a base A_c different from 1000 mm^2 are used, careful attention should be given to size effects.

(11) In Annex M documents have been listed that give examples of the description of the geometry, tolerances and surface roughness of the cone, friction sleeve and shaft, as well as the geometry of gaps, geometry and deformability of seals. It is recommended that these should be used.

(12)P For CPTU a filter element for pore water pressure measurement is incorporated into the cylindrical part of the cone. Its diameter shall not be less than the diameter of the cylindrical part of the cone and not more than the diameter of the friction sleeve, if applied.

(13) The use of additional filter elements placed at other locations is not in contradiction with the minimum requirements given in this section.

(14) For CPTU the geometry of the cone should be such that:

- the height h_e of the cylindrical part including the height h_u of the filter element is: 7,0 mm $\leq h_e \leq$ 10,0 mm,
- the diameter d_u of the filter element is: $d_c \leq d_u \leq d_c$ + 0,2 mm

where:
d_c is the diameter of the cylindrical part of the cone

and the seal in the gap should be such that the remaining area A_i (see figure 3.3) is as small as possible.

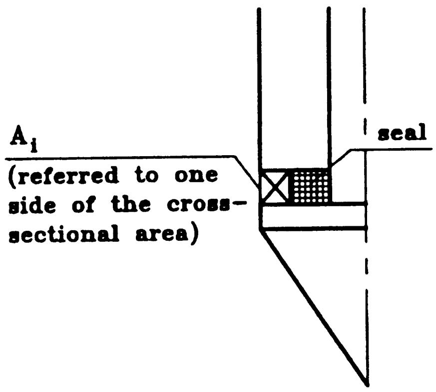


Figure 3.3: Remaining area A_i for a piezocone test CPTU

3.3.2 Measuring equipment

(1)P The apparatus for CPT or CPTU shall include suitable measuring devices with suitable data transmission and data recording systems. The recorded data shall always be accessible on the site.

(2) Devices for measuring the following quantities should be used, when relevant:

- total penetration force Q_t;
- axial force Q_c, acting on the cone, linked to the cone;
- frictional force Q_s, acting on the friction sleeve, linked to the friction sleeve (alternatively, sum of the forces $Q_c + Q_s$, acting on the cone and on the friction sleeve, linked to the cone and friction sleeve);
- inclination and penetration rate of the cone;
- in a CPTU penetration pore pressure u relative to a known pressure, linked to the filter element in the cone.

(3) If electric measuring devices are built into the tip, they should be temperature compensated.

(4) The sensing devices for measuring cone resistance and local side friction should be designed in such a way that eccentricity of the load does not affect the readings.

(5)P The inaccuracy of the measurements shall not exceed 5 % of the measured value.

3.4 Test procedure

3.4.1 Calibration and checks

(1)P All measuring devices shall provide reliable and accurate measurements.

(2) All measuring devices should be calibrated every 3000 m of CPT sounding or at least every 6 months or after repair.

(3)P For a CPTU the filter, its sensing device and all fluid spaces shall be filled with a deaired fluid before each test and precautions shall be taken to ensure full saturation of the whole system.

(4) The wear of the cone and of the friction sleeve should be checked regularly to ensure that they satisfy the tolerances. Examples of tolerances are given in the documents listed in Annex M.

(5) The seals should be checked for the presence of soil particles and be cleaned.

(6) If a friction reducer is used, it should be located at a sufficient distance above the penetrometer tip so as not to influence the measurements.

3.4.2 Performing the test

(1)P The penetration shall be performed with no rotation, no vibrations, nor blows, and the direction of the thrust shall be vertical.

(2) The axis of the push rods should at the surface not deviate, more than 2 % from vertical.

(3)P The rate of penetration shall be (20 ± 5) mm/s.

(4)P The penetration depth of the cone shall be measured with an inaccuracy not exceeding 0,2 m and the depth interval between the readings shall not exceed
0,1 m.

3.5 Interpretation of the results

(1) Results of CPT and CPTU may be used for determining stratification, classifying soils and determining properties of a wide range of soils and soft rocks, provided penetration is possible.

(2) In a CPTU the total (corrected) cone resistance q_t and pore pressure ratio B_q should be determined with q_c and u measured at the same level.

(3) For interpretation of the results of a CPTU the pore pressure u_0 and the total vertical stress σ_{v0} existing in the ground before the test should be used. u_0 is be the equilibrium pore pressure; σ_{v0} may be determined from the unit weights of the ground layers.

(4) When CPT results are used for classifying soils, the classification should be based at least on cone resistance, local unit side friction and friction ratio (or friction index). A better classification is obtained by performing CPTU and using total (corrected) cone resistance q_t, generated pore pressure)u and pore pressure ratio B_q (and/or local unit side friction and friction ratio or index, if relevant).

3.6 Reporting of the results

(1)P In addition to the requirements given in 2.6 the test report shall include the following information:

- apparatus used, type of tip used;
- date of calibrations;
- zero readings of measuring devices;
- system of measurement (electric, mechanical or hydraulic);
- the depth over which a friction reducer or push rods with a reduced diameter have been used;
- the depth of the start of the test;
- the depth of water in the hole or of the water table when available;
- graphs of the following results, with depth, on an arithmetic scale:
  - cone resistance: q_c;
  - local unit side friction: f_s, when relevant;
  - total side friction resistance Q_{st}, when relevant;
  - friction ratio: R_f or friction index: I_f, when relevant;
  - observations of the operator such as incidents, procedure details not included in the test method which might have an influence on the tests results.

(2)P For a CPTU the report shall also include:

- cone area factor: a;
- graphs of the following results, with depth, on an arithmetic scale:
  - penetration pore water pressure: u;
  - total (corrected) cone resistance: q_t (replacing q_c);
  - pore pressure ratio: B_q.

3.7 Derived values of geotechnical parameters

3.7.1 Derived values for calculations of bearing capacity and settlement of spread foundations

(1)P When the bearing capacity or the settlement of a spread foundation is evaluated from CPT results, either a semi-empirical method or an analytical method shall be used.

(2)P When a semi-empirical method is used, all the features of the method shall be taken into account.

(3) When the sample analytical method for bearing capacity of annex B in ENV 1997-1 is used, the undrained shear strength of cohesive soils c_u may be determined from the following relation:

$$c_u = \frac{q_c - \sigma_{v,0}}{N_k}$$

or, in the case of CPTU, preferably by:

$$c_u = \frac{q_t - \sigma_{v,0}}{N_{k,t}}$$

with N_k or N_{kt} estimated from local experience.

(4) When the sample analytical method for bearing capacity of annex B of ENV 1997-1 is used, the angle of shearing resistance ϕ'may be determined from the cone resistance q_c, on the basis of local experience, taking into account depth effects, when relevant.
An example of sample values for quartz and feldspar sands is given in annex B.1 to estimate a value of ϕ' from q_c, for the bearing capacity of spread foundations, when depth effects do not need to be taken into account.

(5) More elaborated methods may also be used for determining $\phi'$from q_c, taking into account the effective vertical stress, the compressibility, and the overconsolidation ratio.

(6) When the adjusted elasticity method of annex D of ENV 1997-1 is used for calculating settlements of spread foundations from CPT results, the correlation between cone resistance q_c and the drained (long term) Young's modulus E_m depends on the nature of the method; the semi-empirical elasticity method, or the theoretical elastic method.

(7) An example of a semi-empirical method for calculating settlements in cohesionless soils is given in annex B.2.

(8) When a theoretical elastic method is used, the drained (long term) Young's modulus E_m may be determined from cone resistance q_c, on the basis of local experience. An example of sample values for quartz and feldspar sands is given in annex B.1 to estimate a value of E_m from q_c.

(9) Correlations between the oedometer modulus E_{oed} and the cone resistance q_c may also be used when calculating settlements of spread foundations.
The following relation between the oedometer modulus E_{oed} and q_c is then often adopted:

$$E_{oed} = \alpha \times q_c$$

where:

α is a correlationfactor estimated on the basis of local experience

An example of a correlation is given in annex B.3.

3.7.2 Pile foundations

(1)P When the ultimate bearing resistance of piles is evaluated from CPT results according to 7.6.3.3(4)P of ENV 1997-1, calculation rules based on established correlations between the results of static load tests and CPT results shall be used.

(2) In annex B.4 an example is given for the assessment of the bearing resistance of a single pile on the basis of q_c-values from a CPT.

4 PRESSUREMETER TEST (PMT)

4.1 General

(1)P The pressuremeter test covers the measurement in situ of the deformation of soils and weak rocks to the expansion of a cylindrical flexible membrane under pressure.

(2)P Four different types of apparatus are specified:

- Ménard pressuremeter (MPM),
- prebored pressuremeters (PBP),
- self-boring pressuremeter (SBP)
- full displacement pressuremeter (FDP).

(3) The PBP and the MPM are lowered into a test hole created specifically for the pressuremeter test.

(4) The SBP is drilled into the ground using an integral cutting head at its lower end such that the probe replaces the material it removes thereby creating its own test hole.

(5) The FDP is usually jacked into the ground with an integral cone at its lower end, thereby creating its own test hole. The MPM may in some instances be jacked or driven into the ground.

(6) PBP, SBP and FDP probes may take a number of forms, therefore descriptions are given in accordance with the type of installation and measuring systems.

(7) Two different basic test procedures are specified: a procedure to obtain a pressuremeter modulus, E_M, and limit pressure, p_{LM}, that may be used in design procedures formulated for the Ménard pressuremeter; and a procedure to obtain other stiffness and strength parameters.

(8)P The tests shall be carried out in accordance with a test method that complies with the requirements given in this section.

(9)P The test method used shall be reported in detail together with the test results.

(10) The test method may be reported by reference to a standard.

(11)P Any deviations from the requirements given below shall be justified and in particular their influence on the results shall be commented upon.

4.2 Definitions

(1)P pressuremeter: cylindrical device, designed to apply a uniform pressure to the walls of a cavity by means of a flexible membrane. The main components of a pressuremeter are shown in fig. 4.1.

(2)P probe: cilindrical part of the pressuremeter that is inflated and thereby transmits the pressure to the cavity walls. The probe may be lowered into a prebored hole, be bored, jacked or driven into the ground (see fig. 4.1)

(3)P expanding section: part of the probe fitted with the flexible membrane. The expanding section may be subdivided in one or three cells (see fig. 4.1).

(4)P test hole: length of borehole, specifically created for pressuremeter testing. It can be of the same diameter or smaller than the borehole and is created below the base of the borehole.

(5)P cavity: portion of a test hole that is subjected to pressure by the expanding membrane. At the start of a test the diameter of the cavity is the same diameter as the test hole.

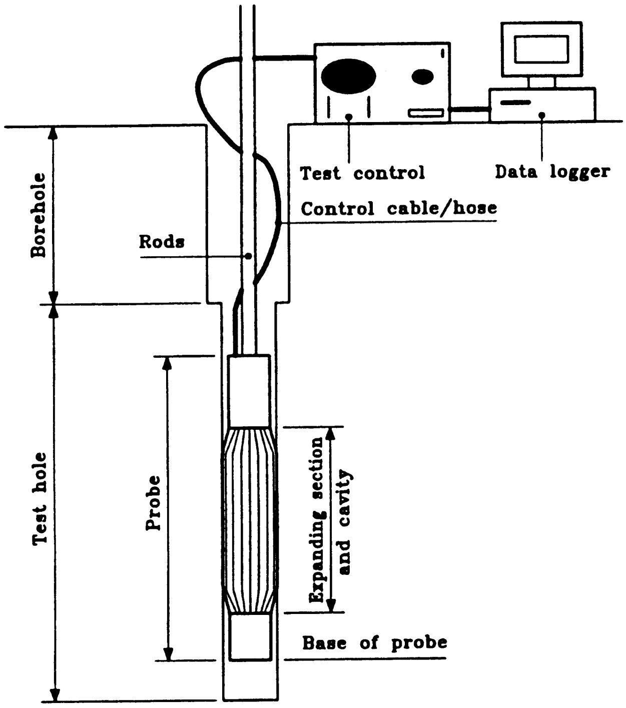


Figure 4.1: Key features of a pressuremeter

(6)P monocell probe: pressuremeter with a single expanding cell.

(7)P tricell probe: pressuremeter with three expanding cells, one of which, the central cell, is the measuring cell.

(8)P volume displacement type pressuremeter: pressuremeter fitted with a volume change gauge to measure the change in volume of the expanding section.

(9)P radial displacement type pressuremeter: pressuremeter fitted with displacement transducers to measure the change in radius or diameter of the expanding section.

(10)P membrane stiffness: pressure required to inflate the membrane in air.

(11)P membrane compression: change in thickness of the membrane as the pressure is increased.

(12)P system expansion: change in volume of the pressuremeter system, excluding the volume change of the probe, as a result of a pressure change.

(13)P applied pressure: is the pressure applied by the pressuremeter to the walls of the cavity in the soil or rock.

(14)P volume change: change in volume of the expanding cavity.

(15)P volumetric strain: change in volume of the cavity with respect to the original volume of either the cavity or the expanding section of the probe.

(16)P cavity strain: change in radius of the cavity with respect to the original radius of the cavity.

(17)P pressuremeter test curve: plot of the variation of the applied pressure with either the cavity strain or volume change or yhe volumetric strain.

(18) The basic curve can have one of the following forms, shown in figure 4.2. For the MPM test the axes are usually transposed.

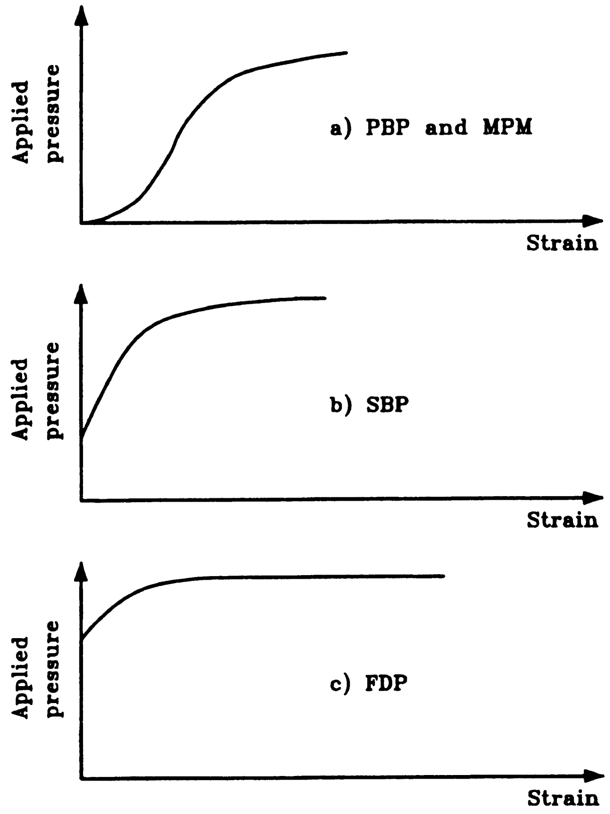


Figure 4.2: Forms of pressuremeter curves

4.3 Equipment

4.3.1 Apparatus

(1)P The expanding section of a pressuremeter, defined in 4.2, shall have a length to diameter ratio greater than 5.

(2)P The diameter of the flexible membrane shall be capable of expansion without bursting by:

- 50 % if the Ménard method is required;
- 50 % if an FDP is required;
- 25 % if a PBP is required;
- 15 % if an SBP is required in soils;
- 10 % if an SBP is required in weak rocks.

(3)P If a volume displacement type probe is used, volume changes shall be measured either in the control unit, at the surface, or in the probe.

(4)P If a radial displacement type probe is used, displacements shall be measured directly on the membrane circumference, at three or more equidistant points in the plane perpendicular to the axis through the centre of the probe.

(5)P The thrust machine and push rods used to jack an FDP into the ground shall comply with the specification for cone penetration tests.

4.3.2 Measuring equipment

(1)P Volume measuring gauges shall be capable of measuring changes in volume equal to 100 % or 50 % of the initial volume of the expanding section of the probe depending on the test method specified (100 % for the Ménard method).

(2)P The applied pressure shall be measured either in the control unit at the surface or in the probe and the gauge or transducer shall have a sensitivity compatible with the pressure range required for the ground being tested.

(3)P The working range of a pressure transducer or gauge shall be suitable for the ground to be tested.

(4)P The resolution of all electrical measuring devices and any associated data acquisition systems shall be within 0,1 % of their full working range. The resolution of all other measuring devices shall be within 1% of their full working range.

4.4 Test procedure

4.4.1 Calibrations

(1)P Calibrations for all probes shall be reliable, repeatable, accurate and traceable to standards.

(2)P Results of all calibrations shall be available on site for inspection.

(3)P The following calibrations shall be required:

- volume displacement type probes:
  - pressure transducers or gauges;
  - volume displacement gauges;
  - system expansion;
  - membrane stiffness;
  - membrane compression (if testing rock);

- radial displacement type probes:
  - pressure transducers;
  - displacement transducers;
  - membrane stiffness;
  - membrane compression (if testing rock).

(4)P Transducers and gauges shall be calibrated prior to the start and following the completion of the testing programme. For MPM testing this only applies if no calibration has been performed within the last 6 months.

(5)P Calibration of membrane stiffness shall be carried out prior to the start of testing on site and after the completion of every borehole. A new membrane shall be calibrated before use.

(6)P The displacement and pressure transducers shall be re calibrated following any repair of the transducers readout/control unit and connecting cables.

4.4.2 Installation

(1)P The probe shall be installed in such a manner that the disturbance to the surrounding ground is kept to a minimum for the type of probe.

(2)P Boring to form a test hole below the base of a borehole for the insertion of an MPM or PBP shall be carried out using equipment, techniques and a flushing medium consistent with achieving minimum disturbance. For any particular test hole sampling requirements shall be specified.

(3)P The boring techniques and flushing medium used with a SBP shall be consistent with achieving minimum disturbance to the ground surrounding the probe.

(4)P The initial volume of a deflated volume displacement type probe shall be determined at the surface before it is lowered into the borehole.

(5) The orientation of the probe may be recorded prior to the start of insertion into the borehole and before it is removed, particularly when radial displacement probes are used.

4.4.2.1 Ménard and other prebored pressuremeters

(1)P The diameter of the test hole shall be kept close to the diameter of the deflated probe, but shall not be less than the diameter of the deflated probe.

(2)P If a test hole is to be used for one test only, then the probe shall be inserted into the test hole within 60 min after completion of drilling the test hole, provided the depth to the test hole does not exceed 30 m. For holes below that, more time may be used.

(3)P If the borehole is of a larger diameter than the test hole then the distance between the top of the expanding section of a probe and the base of the borehole shall be at least half the length of the expanding section.

(4)P For a long test hole in which a number of tests will be carried out, all boring and testing shall be completed within one working shift.

4.4.2.2 Self-boring pressuremeter

(1)P The SBP shall be bored continuously from the ground surface or from the base of a borehole. It shall be bored a sufficient distance beyond a previous test position or beyond the base of a borehole into ground that is unaffected by either the previous test or the drilling of the borehole.

(2)P The minimum distance between test locations shall be equal to twice the distance between the centre of the expanding section and the base of the probe. If a length greater than that distance is required, then the length to be self-bored shall be specified.

(3)P During self-boring of a SBP the setting, type and rotational speed of the cutter, type and characteristics of drilling fluid, the drilling fluid pressure, rate of advance and ram pressure shall all be adjusted to ensure minimum disturbance without causing undue risk of damage to the equipment.

(4)P In free draining soils an expansion test shall be started as soon as practically possible following the completion of self-drilling. In all remaining soils and weak rocks there shall be a minimum of 30 minutes and a maximum of one hour between completion of self-boring and the start of the expansion test.

4.4.2.3 Full displacement pressuremeters

(1)P When a FDP incorporates an electrical cone, it shall be jacked into the soil between test levels at (20 ± 5) mm/s either from the base of a borehole or from the surface.

(2)P An expansion test shall be started as soon as practically possible following the completion of jacking.

4.4.3 Testing procedure

(1)P The expanding section of the pressuremeter shall be pressurised until the specified diameter as defined in 4.3.1(2)P is reached. The maximum pressure capacity of the probe shall be consistent with the pressure required to inflate the expanding section in the particular ground conditions as stated in 4.3.(2)P. For safety reasons the expansion may be terminated if either the maximum pressure capacity of the probe is reached or if any of the displacement transducers has reached its full working range or the maximum safe expanded volume is reached.

(2) The pressuremeter may be unloaded to produce an unload-reload cycle. The rates of unloading and reloading should be specified.

(3) Prior to unloading there can be a period during which the pressure or displacement is held constant.

(4) The reduction in stress during any unload-reload cycle should be specified.

(5) The strain at the start of any additional unload-reload cycles may be specified.

(6)P The output from the transducers shall be recorded at a minimum frequency of 10 s. intervals throughout the test if automatic recording is used, or 30 s. if manual readings are taken or shall be specified.

(7) For MPM tests recordings or readings are conventionally taken at 15 s, 30 s and 60 s after pressure application is completed.

4.4.3.1 Stress controlled tests - Ménard method

(1)P With the Ménard stress controlled method tests shall be carried out at a constant rate of pressure increase. The increments shall be adjusted to ensure that there are at least seven increases in pressure throughout the loading stage, however ten increases are preferred.

(2)P Each pressure increment shall be held constant for 1 min.

4.4.3.2 Other stress controlled tests

(1)P Incremental stress controlled tests shall be carried out at a constant rate of pressure increase.

(2) One unload-reload cycle, if required, can be included in the loading sequence when either the cavity has increased in diameter by between 1 % and 3 %, or when the pressure has reached 10 MPa, whichever occurs first.

(3)P The increments shall be no greater than 5 % of the maximum capacity of the probe and shall be adjusted to ensure that there are at least fifteen increases in pressure throughout the loading stage (excluding any unload-reload cycles).

(4) The size of the pressure increments may be adjusted during the early stage of the test to ensure that a sufficient number of readings can be taken to accurately define
when the membrane comes in contact with the side of the hole and shall be no greater than 0,2 MPa.

(5)P Each pressure increment shall be held constant for 1 min.

4.4.3.3 Strain controlled tests

(1)P Strain controlled tests undertaken with a PBP shall be carried out using constant increments of volumetric or cavity strain. The increments shall be adjusted to ensure that there are at least twenty increases in volume throughout the loading stage.

(2)P Each increment shall be maintained for 1 min.

(3)P Strain controlled tests undertaken with an SBP shall be carried out at a constant rate of stress increase during the early stage of the test. Once expansion starts, a constant rate of strain of 1 % per minute shall be used.

(4)P Strain controlled tests undertaken with an FDP shall be carried out at a constant rate of stress increase during the early stage of the test. Once expansion starts, a constant rate of strain shall be used.

(5)P A sufficient number of readings shall be taken to define accurately the pressure at which expansion starts.

(6) An unload-reload cycle, if required in a PBP test, should be carried out when either the hole has increased in diameter by between 1 % and 3 % or when the pressure has reached 10 MPa, whichever occurs first.

(7) An unload-reload cycle, if required in an SBP test, should be carried out when the hole has increased in diameter by between 1 % and 3 %.

(8) An unload-reload cycle, if required in an FDP test, may be carried out once constant pressure conditions are approximately achieved.

4.5 Interpretation

4.5.1 Data reduction of pressuremeter tests

(1)P The applied pressure, corrected for membrane stiffness if necessary, shall be converted to stress.

(2)P If a radial displacement type pressuremeter is used, the displacement readings shall be converted to cavity strain and, if testing weak rock, corrected for membrane compression and thinning.

(3)P If a volume displacement type pressuremeter is used (e.g. Ménard) then the volume reading shall be corrected for system expansion.

4.5.2 Interpretation of the Ménard test

(1)P A plot of corrected volume change against corrected applied pressure expressed in stress shall be produced.

(2)P The Ménard pressuremeter modulus E_m and limit pressure p_{LM} shall be determined using the method illustrated in figure 4.3.

(3)P p_{LM} is defined as the pressure required to double the total volume of the cavity from the point (p_r, V_r) (see fig. 4.3). The pressure p_r is the pressure where $d(\Delta V)/dp$ is a minimum and V_r is the corresponding value of the injected volume. V_c is the deflated volume of the probe.

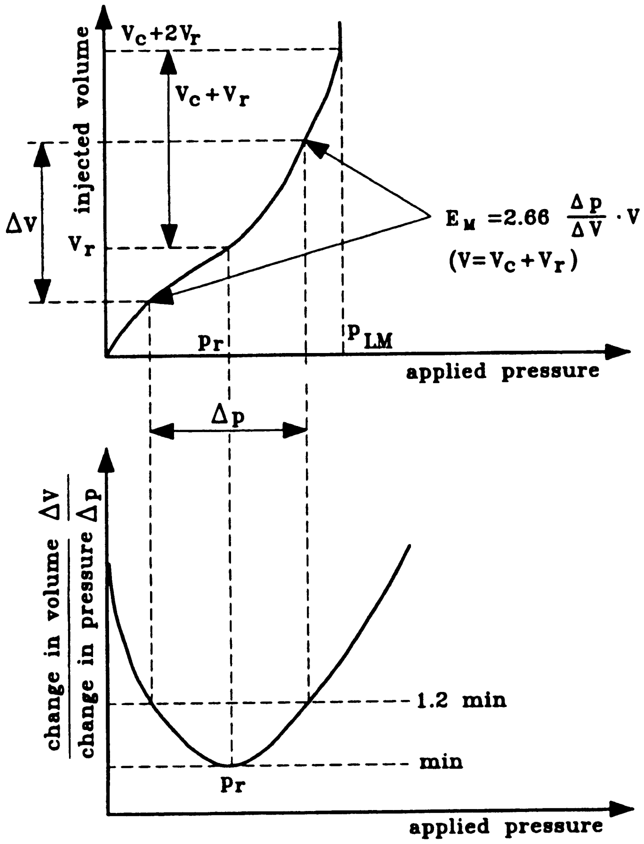


Figure 4.3: Interpretation of E_M and p_{LM} for an MPM test

4.6 Reporting the results

(1)P The following information shall be submitted prior to starting site work:

- full details of the pressuremeter and testing equipment;
- details of the proposed drilling equipment and flushing medium to be used for drilling boreholes;
- description of the methods of carrying out all the drilling operations for forming test holes;
- typical test data sheets and forms for presenting final results;
- reference description of the methods for carrying out the tests.

(2)P In addition to the requirements given in 2.6 the report shall include the following information to be submitted for each test on request:

- depth of the top and bottom of the test hole, and depth of the displacement measurement axes;
- details of bored, drilling (and self-drilling if applicable), including date and time of start and finish of all drilling, description and estimate of any drilling fluid returns and depth and size of casing used, if any;
- the output of the transducers recorded prior to and during installation and on removal from the borehole for any PBP;
- tabulated output of the transducers during a test, if required, time of start and finish of test and rates of stress and/or strain on magnetic media;
- the calibration results used to convert the test data to engineering units;
- tabulated calibrated test data on magnetic media on request if applicable;
- a plot of applied pressure against the volumetric or average cavity strain expressed as a percentage.

(3)P All information shall be presented. All graphical plots shall be presented on a scale which sensibly fills the entire page.

(4)P Values of E_M and p_{LM} if a Ménard test is carried out shall be submitted.

(5) In table 4.1 a list of additional plots is given.

Table 4.1: A list of additional plots

Probe	Ground type	Abscissa	Ordinate
radial displacement type			
self-bored, pushed in	all	cavity strain for each arm	applied pressure
prebored	all	cavity strain for each pair of arms	applied pressure
self bored	all	initial cavity strain for each arm	applied pressure
all	all	cavity strain for unload- reload cycle for each arm	applied pressure
all	clay	logarithm of cavity strain for each arm	applied pressure
all	sands	natural logarithm of current cavity strain for each arm	natural logarithm of effective applied pressure
volume displacement type (except MPM)[*)]			
prebored	all	volume change	applied pressure
prebored	all	rate of change of volume	applied pressure
[*)] For MPM tests the pressure is plotted as abscissa and the volume change as ordinate.			

4.7 Derived values of geotechnical parameters

(1)P When an indirect or analytical method is used the geotechnical parameters of shear strength and shear modulus shall be derived from the pressuremeter curve.

(2)P When a direct or semi-empirical method is used all the features of the method shall be taken into account.

4.7.1 Derived values for the calculation of bearing capacity resistance of spread foundations

(1) When the sample semi-empirical method of annex C in ENV 1997-1 is used the specification for the Ménard pressuremeter may be followed.

(2) An example of the calculation of the bearing resistance is given in annex C.1.

(3) When the sample analytical method of annex B in ENV 1997-1 is used, the strength of the soil may be determined using empirical and theoretical methods but only on the basis of local experience.

(4) The angle of shearing resistance, ϕ', may be determined from an SBP test in non cohesive soils by theoretical methods and from FDP and PBP tests using empirical correlations but only on the basis of local experience.

4.7.2 Derived values for the calculation of settlement of spread foundations

(1) The settlement of spread foundations may be determined from stress controlled tests (Ménard method, see 4.4.3.1) using a semi-empirical method.

(2) An example of the calculation of the settlement of a spread foundation is given in annex C.2.

(3) When the sample analytical methods of annex D in ENV 1997-1 are used, the stiffness of the soil may be determined using theoretical methods but only on the basis of local experience.

4.7.3 Pile foundations

(1) The ultimate bearing resistance of piles may be evaluated directly from stress controlled tests (Ménard method, see 4.4.3.1).

(2) An example of the calculation of the ultimate bearing resistance is given in annex C.3.

(3) When the ultimate bearing resistance of a pile is evaluated indirectly from pressuremeter test results an analytical method may be applied to derive values of base and shaft resistance based on local experience.

5 STANDARD PENETRATION TEST (SPT)

5.1 General

(1)P This method covers the determination of the resistance of soils at the base of a borehole to the dynamic penetration of a split barrel sampler and the obtaining of disturbed samples for identification purposes.

(2)P The basis of the test consists in driving a sampler by dropping a hammer of 63,5 kg mass on to an anvil or drive head from a height of 760 mm. The number of blows (N) necessary to achieve a penetration of the sampler of 300 mm (after its penetration under gravity and below a seating drive) is the penetration resistance.

(3) The test is used mainly for the determination of the strength and deformation properties of cohesionless soils, but some valuable data may also be obtained in other types of soils.

(4)P The tests shall be carried out in accordance with a method that complies with the requirements given in this section.

(5)P The test method used shall be reported in detail with the test results.

(6) The test method may be reported by reference to a standard.

(7)P Any deviation from the requirements given below shall be justified and in particular its influence on the results of the test shall be commented upon.

(8) Experience of deviations exists with respect to the following points:

- use of a solid 60° cone instead of the standard shoe when dealing with gravelly materials;
- use of a barrel with an internal diameter larger than that of the shoe, in order to place an inside liner flush with the shoe where the sample is recovered;
- an encased hammer assembly operating directly on top of the sampler in the bottom of the borehole.

5.2 Definitions

(1)P drive-weight assembly: a device consisting of the hammer, hammer fall guide, the anvil, and the hammer drop system.

(2)P anvil or drive head: that portion of the drive-weight assembly which the hammer strikes and through which the hammer energy passes into the drive rods.

(3)P drive rods: rods that connect the drive-weight assembly to the sampler.

(4)P Energy ratio-ER_r: ratio between the actual energy delivered into the drive rod, immediately below the anvil, and the theoretical free fall energy of the hammer, expressed in percentage.

(5)P hammer: that portion of the drive-weight assembly consisting of the 63,5 kg impact weight which is successively lifted and dropped to provide the energy that accomplishes the sampling and penetration.

(6)P N-value: the number of blows required to drive the sampler. The N-value, reported in blows per 300 mm, equals the sum of the number of blows required to drive the sampler over the depth interval of 150 mm to 450 mm from the base of the borehole (and after its eventual penetration under gravity).

(7)P N_{60}-value: N-value corrected to a reference energy ER_r of 60 %.

(8)P $(N_1)_{60}$-value: N-value corrected to a reference energy ER_r of 60 % and an effective vertical stress σ = 100 kPa.

5.3 Equipment

5.3.1 Boring equipment

(1)P The boring equipment shall be capable of providing a clean hole to ensure that the penetration test is performed on essentially undisturbed soil. The correction for the diameter must be considered.

(2) The diameter of the borehole should not be larger than 150 mm.

5.3.2 Sampler

(1)P The steel sampler shall have the dimensions indicated in figure 5.1 and shall be provided with a non-return valve with sufficient clearance to permit the free flow of water or mud during driving.

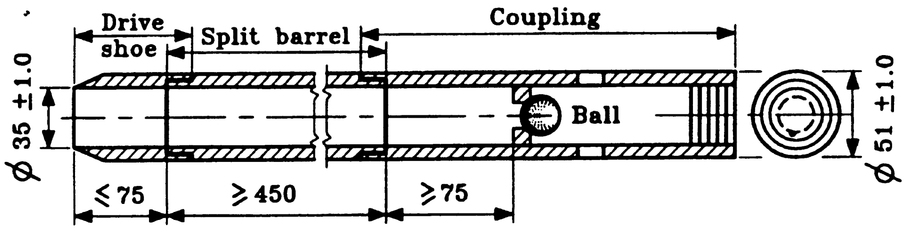


Figure 5.1: Longitudinal cross section of an SPT sampler (dimensions in mm)

5.3.3 Drive rods

(1)P The drive rods shall have a stiffness that prevents buckling during driving. Rods with a mass of more than 10,0 kg/m mass shall not be used.

5.3.4 Drive weight assembly

(1)P The drive weight assembly, of an overall mass not exceeding 115 kg, shall comprise:

- a steel hammer of 63,5 kg ± 0,5 kg conveniently guided to ensure minimal resistance during the drop;
- an automatic release mechanism which will ensure a constant free fall of (760 ± 10) mm, a negligible speed of the hammer when released, and no induced parasitic movements in the drive rods;
- a steel drive head or anvil rigidly connected to the top of the drive rods. It may be an internal part of the assembly, as with safety hammers.

5.4 Test Procedure

5.4.1 Preparation of the borehole

(1)P The borehole shall be clean and essentially undisturbed at the test elevation and without an upward water pressure gradient.

(2)P When boring bits are used, they shall be provided with side discharge and not with bottom discharge, from a safe distance of the test elevation.

(3)P When testing below the groundwater table, particular care shall be taken to avoid any entry of water through the bottom of the borehole, as this will tend to loosen the soil or even lead to piping. For this purpose, the level of the water or drilling fluid in the borehole shall be maintained at a sufficient distance above the groundwater level at all times, even during withdrawal of the boring tools. Withdrawal shall be performed slowly and with drilling tools providing enough clearance to prevent suction effects at the bottom.

(4)P When a casing is used, it shall not be driven below the level at which the test will start.

5.4.2 Penetration test

(1)P Lower the sampler and the drive rods (with the hammer assembly on top) to the specified bottom of the borehole. The sampler shall be penetrated over an initial or seating drive of 150 mm applying the 63,5 kg hammer falling 760 mm and the number of blows shall be recorded. Then the sampler in the same manner shall be driven over 2 increments of 150 mm. The number of blows needed, or 50 blows, which ever occurs first, shall be recorded during each of these increments. The total number of blows required for the 300 mm penetration after the seating drive is termed the penetration resistance (N).

(2)P If the sampler advances below the bottom of the boring under the static weight of the drive rods and hammer assembly on top, the corresponding penetration shall not be included as seating drive and this information should be reported. In no case shall any material reach the level of the non return valve.

(3)P Representative samples or samples recovered shall be placed in air-tight containers, clearly labelled with all pertinent data.

5.5 Interpretation of the results

5.5.1 Introduction

(1)P Existing design methods of foundations based on the SPT are of empirical nature. Operating methods have been adapted to obtain more reliable results. Therefore, the application of appropriate correction factors for interpreting the results shall be considered. In the following paragraphs, correction factors are given for energy delivered to the drive rods, for the effect of overburden pressure and other circumstances.

5.5.2 Energy delivered to the drive rods

(1) Energy losses are induced by the hammer assembly due to frictional and other parasitic effects, which cause the hammer velocity at impact to be less then the free fall velocity. Further losses of energy are originated by the impact on the anvil, depending on its mass and other characteristics. The type of machine, skill of the operator and other factors can also influence the energy delivered to the drive rods.

(2)P The value of the blow count, N, in sands is inversely proportional to the energy ratio E_{R_r} so that:

$N_a \times ER_{r,a} = N_b \times ER_{r,b}$

(3)P The ER_r-value of the equipment used has to be known if the N-values are going to be used for the quantitative evaluation of foundations or for the comparison of results. A certificate of calibration of the ER_r-value imediately below the driving head or anvil shall be available.

(4)P For general design and comparison purposes in sands, the N-values shall be adjusted to a reference energy ratio of 60 %, by the following expression:

$$N_{60} = \frac{ER_r}{60} N$$

where:
N is the blow count and ER_r is the energy ratio of the specific test equipment.

(5) In table D.1 (annex D) an example is given of the energy ratios of the equipment commonly used in various countries and the corresponding correction factors for normalizing to ER_r = 60 %.

(6)P If a design method for sands has been elaborated for a value of ER_r different from 60 %, the corresponding corrected N-value shall be determined based on the expression shown in 5.5.2 (2).

5.5.3 Energy losses due to the length of rods

(1) If the length of rods is less than 10 m, the energy reaching the sampler is reduced and the correction factors shown in table 5.1 should be applied to the blow count for sands.

Table 5.1: Correction factors in sands due to rod length

Rod length below the anvil [m]	Correction factor, λ
>10	1,0
6 - 10	0,95
4 - 6	0,85
3 - 4	0,75

5.5.4 Effect of overburden pressure in sands

(1) The effect of the overburden pressure in the N-value in sands with reference to the density index I_D may be taken into account by applying to the measured N-value the correction factor C_N given in table 5.2.

(2) Table 5.2: Correction factors C_N due to the effective overburden pressure in sands

Type of sand	Density Index I_D [%]	C_N
Normally consolidated	40 to 60	$\frac{2}{1+\sigma_v'}$
	60 to 80	$\frac{3}{2+\sigma_v'}$
Overconsolidated		$\frac{1,7}{0,7+\sigma_v'}$
(σ_v', in kPa x 10^{-2})		

For an effective overburden pressure of 100 kPa, $\sigma'_v = 1$ and hence $C_N = 1$ and the value of N is then defined as the normalized value N_1.

(2) Values of the correction factor C_N larger than 2,0 and preferably 1,5 should not be applied.

5.5.5 Other correction factors

(1) If the inner diameter of the sampler is 3,0 mm larger than that of the shoe, as in the deviation to this test procedure mentioned in 5.1(7) no correction is necessary if a liner of appropriate thickness is used, such that the inside of the whole sampler is practically flush to a uniform diameter of 35 mm. Nevertheless, attention should be paid to the eventual damage of the liner during driving and its influence on the corresponding blow count. If the liner is omitted, the additional clearance of the inside of the barrel with reference to the shoe leads to N-values between 10 % and 20 % lower in sands.

5.5.6 Use of the correction factors

(1)P Several correction factors have been mentioned in the previous paragraphs. As the existing design methods of foundations based on the SPT are of an empirical nature, only the corresponding correction factors shall be used, unless duly justified.

(2) If all the correction factors corresponding to this test procedure are applied for a design method based on an energy ratio of 60 %, the following value for the final blow count would be obtained (without including the one mentioned in 5.5.5 (1)):

$$N_{60} = \frac{ER_r}{60} \times \lambda \times C_N \times N$$

where:

λ is the correction factor for energy losses due to the rod length in sand

C_N is the correction factor for the effective overburden pressure in sand

5.6 Reporting of results

(1)P In addition to the requirements given in 2.6 the report shall include the following information:

- penetration of the sampler under static weight, if significant;
- number of blows required for each 150 mm increment of penetration (including the seating drive);
- if the drive is terminated before the full penetration in any of the 150 mm interval: record of the depth of penetration for the corresponding 50 blows;
- the original N-value and the corresponding depth interval and soil description;
- the corrections applied, if any, and the corrected N-value;
- boring method and diameter;
- location of the water table, if known, and level of the water or drilling fluid during the preparation and execution of each test;
- dimensions and weight of drive rods used for the penetration test;
- type of hammer and release mechanism and weight of the drive head;
- the energy ratio ER_r;
- the N-values should be reported in the boring log at each corresponding elevation, with a graphical representation.

5.7 Derived values of geotechnical parameters

5.7.1 General criteria

(1) When dealing with cohesionless sands, a wide empirical experience in the use of this test is available, such as for the quantitative evaluation of the density index, the bearing capacity and the settlement of foundations, even though the results should be considered as only a rough approximation. Most of the existing methods are still based on uncorrected or partly corrected values.

(2) There is no general agreement on the use of the SPT in clayey soils. In principle it should be restricted to a qualitative evaluation of the soil profile or to a qualitative estimate of the strength properties of the soil. Nevertheless it may sometimes be used in a quantitative way, under well known local conditions, when directly correlated to other appropriate tests.

5.7.2 Bearing capacity in sands

(1) Bearing capacity problems may be solved through the determination of the density index, and the effective angle of shearing resistance, N'.

5.7.2.1 Derived values for the density index

(1) In annex D.2 an example is given of the relationship between N_{60} and $(N_1)_{60}$ versus the density index I_D.

(2) The resistance of sand to deformation is greater the longer the period of consolidation. This "ageing" effect is reflected in higher blow counts.

(3) Overconsolidation increases the blow counts, for the same values of I_D and F_v'.

(4) In annex D.2 the effect of both ageing and overconsolidation is illustrated through some sample correlations.

5.7.2.2 Derived values for the angle of shearing resistance N'

(1) The angle of shearing resistance may be derived directly from the value of N through empirical correlations.

(2) It can also be estimated using the density index as an intermediate parameter. In annex D.3 a sample correlation of this type is shown.

5.7.3 Settlement of spread foundations in sand

(1) When a purely elastic method is used, the drained Young's modulus E_m may be derived from the N-values through empirical correlations.

(2) Alternatively, the density index may be derived based on the N_{60}-value, and then use an appropriate correlation to obtain E_m, through the density index.

(3) The direct methods are based on comparisons of the N-values and results of plate loading test or records of measured settlements of foundations. Allowable bearing pressures for a maximum settlement of 25 mm or the settlement corresponding to a given applied pressure can be obtained through the corresponding procedures with reference to the width of the footing, its embedment in the ground and groundwater table position.

(4) A sample method for the calculation of the settlements originated by spread foundations in sand is given in annex D.4.

5.7.4 Pile foundations in sand

(1)P When the ultimate bearing resistance of piles is evaluated from SPT results according to 7.6.3.3 (4) of ENV 1997-1, calculation rules based on established correlations between the results of static load test and SPT results shall be used.

6 DYNAMIC PROBING TEST (DP)

6.1 General

(1)P This method covers the determination of the resistance of soils and soft rocks in situ to the dynamic penetration of a cone. A hammer of a given mass and falling height is used to drive the cone. The penetration resistance is defined as the number of blows required to drive the penetrometer over a defined distance. A continuous record is provided with respect to depth but no samples are recovered.

(2)P Four procedures are included, covering a wide range of specific work per blow: DPL, DPM, DPH and DPSH.

- Dynamic probing light (DPL): test representing the lower end of the mass range of dynamic penetrometers included in this document and defined in table 6.1.
- Dynamic probing medium (DPM): test representing the medium mass range of dynamic penetrometers included in this document and defined in table 6.1.
- Dynamic probing heavy (DPH): test representing the medium to very heavy mass range of dynamic penetrometers included in this document and defined in table 6.1.
- Dynamic probing superheavy (DPSH): test representing the upper end of the mass range of dynamic penetrometers included in this document and defined in table 6.1, closely related to the dimensions of the SPT.

(3) The results of this test are specially suited for the qualitative evaluation of a soil profile or as a relative comparison of other in situ tests. They may also be used for the determination of the strength and deformation properties of soils, generally of the cohesionless type, through appropriate correlations.

(4)P The tests shall be carried out in accordance with a method that complies with the requirements given in this section (see also 1.1, which especially applies to well established national equipment standards of corresponding countries).

(5)P The test method used shall be reported in detail with the test results.

(6) The test method may be reported by reference to a published standard.

(7)P Any deviation from the requirements given below shall be justified and in particular its influence on the results of the test shall be commented upon.

(8) Experience of deviations exists with respect to:

- hammer mass: e.g. 63,5 kg instead of the 50 kg specified for the DPH;
- falling height;
- dimensions of the cone: e.g. an area of 15 cm^2 instead of the 10 cm^2 specified for the DPM.

(9) In locations with special difficulties of accessibility lighter equipment and procedures other than those specified below may be used.

6.2 Definitions

(1)P anvil or drive head: that portion of the drive-weight assembly which the hammer strikes and through which the hammer energy passes into the drive rods.

(2)P cone: pointed probe of standard dimensions used to measure the resistance to penetration (see figure 6.1).

(3)P drive rods: rods that connect the drive-weight assembly to the cone.

(4)P drive-weight assembly: a device consisting of the hammer, hammer fall guide, the anvil and the drop system.

(5)P Energy ratio-ER_r: ratio of the actual energy delivered by the drive-weight assembly into the drive rod, immediately below the anvil, and the theoretical free fall energy of the hammer, expressed in percentage.

(6)P hammer: that portion of the drive-weight assembly which is successively lifted and dropped to provide the energy that accomplishes the penetration of the cone.

(7)P N-values: number of blows required to drive the penetrometer over a defined distance, expressed in cm by the corresponding subindex (N_{10} or N_{20}).

6.3 Equipment

6.3.1 Cone

(1)P The cone of steel or cast iron shall have an apex angle of 90° and an upper cylindrical extension mantle and transition to the extension rods as shown in Figure 6.1 and with the dimensions and tolerances given in Table 6.1. The cone may be sacrificial or retained for recovery.

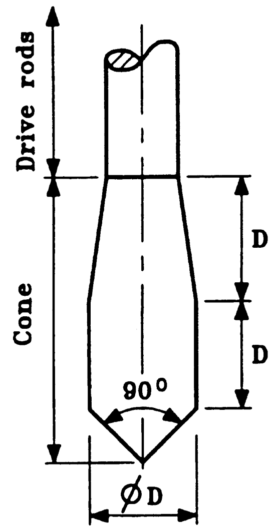


Figure 6.1: Cone for dynamic probing

6.3.2 Drive rods

(1)P The rod material shall be of a high-strength steel with the appropriate characteristics for the work to be performed without excessive deformations and wear. The rods shall be flush jointed. Dimensions and masses of the drive rods are given in table 6.1.

(2) Solid rods may be used, hollow rods should be preferred in order to reduce weight.

Table 6.1: Dimensions and masses for the four types of dynamic probing apparatus

	DPL (light)	DPM (medium)	DPH (heavy)	DPSH (super heavy)
Driving device				
hammer: mass m (kg) height of fall h (mm) length/diameter (D_h) ratio	10 ± 0,1 500 ± 10 ≥ 1 ≤ 2	30 ± 0,3 500 ± 10 ≥ 1 ≤2	50 ± 0,5 500 ± 10 ≥ 1 ≤ 2	63,5 ± 0,5 750 ± 20 ≥1 ≤2
Anvil				
diameter, d, (mm) mass (kg) (max.) (guide rod included)	$100<d<0,5\ D_h$ 6	$100<d<0,5\ D_h$ 18	$100<d<0,5\ D_h$ 18	$100<d<0,5\ D_h$ 30
90° Cone				
nominal base area A (cm^2) base diameter (D) new (mm) base diameter, worn (mm) (min.) mantle length (mm) tapper angle upper (deg.) length of cone tip, (mm) tip max. Permissible wear (mm)	10 35,7 ± 0,3 34 35,7 ± 1 11 17,9 ± 0,1 3	10 35,7 ± 0,3 34 35,7 ± 1 11 17,9 ± 0,1 3	5 43,7 ± 0,3 42 43,7 ± 1 11 21,9 ± 0,1 4	20 51 ± 0,5 49 51 ± 2 11 25,3 ± 0,4 5
Drive rods				
mass (kg/m) (max) diameter OD (mm) (max) rod deviation: lowermost 5 m, % remainder, %	3 22 0,1 0,2	6 32 0,1 0,2	6 32 0,1 0,2	6 32 0,1 0,2
specific work per blow				
(mgh/A) in kJ/m^2	50	150	167	238

6.3.3 Driving device

(1)P Dimensions and masses of the components of the driving device are given in table 6.1. The following requirements shall be fulfilled:

– the steel hammer must be conveniently guided to ensure minimal resistance during the drop;

– the automatic release mechanism shall ensure a constant free fall, with a negligible speed of the hammer when released and no induced parasitic movements in the drive rods;

– the steel drive head or anvil shall be rigidly connected to the top of the drive rods.

6.4 Test procedure

(1)P The drive rods and the cone shall be driven vertically and without undue bending of the protruding part of the extension rods above the ground.

(2)P The penetrometer shall be continuously driven into the ground. The driving rate should be kept between 15 and 30 blows per minute, except when penetrating sand and gravel, where the driving rate may be increased up to 60 blows per minute. All interruptions longer than 5 minutes shall be recorded.

(3)P The rods shall be rotated 1½ turns every 1,0 m.

(4) For rotating the rods, a torque measuring wrench (capacity > 200 Nm, graduations < 5 Nm) should be used.

(5) To decrease skin friction, drilling mud or water may be injected through horizontal or upwards holes in the hollow rods near the cone. A casing is sometimes used with the same purpose.

(6)P The number of blows shall be recorded every 100 mm for the DPL, DPM and DPH (N_{10}) and every 200 mm for the DPSH (N_{20}).

(7) The normal range of blows, especially in view of any quantitative interpretation of the test results, is between $N_{10} = 3$ and 50 for DPL, DPM and DPH and between $N_{20} = 5$ and 100 for DPSH. The rebound per blow should be less than 50 % of the penetration per blow. In cases beyond these ranges, when the penetration resistance is low, e.g. in soft clays, the penetration depth per blow may be recorded. In hard soils or soft rocks, where the penetration resistance is very high, the penetration for a certain number of blows may be recorded.

(8) For obtaining reliable results, the maximum investigation depths recommended are: 8 m for DPL, 20 to 25 m for DPM and 25 m for DPH.

6.5 Interpretation of test results

(1)P The test results shall be interpreted in one of the following ways:

- in terms of N_{10} for DPL, DPM, and DPH or N_{20} for DPSH;
- by determining the unit point resistance (r_d) or the dynamic point resistance (q_d) using the following formulae:

$$r_d = \frac{mgh}{Ae}$$

$$q_d = \frac{m}{m + m'} \times r_d$$

where:

r_d and q_d	are resistance values in Pa;
m	is the mass of the hammer in kg;
g	is the acceleration due to gravity in N/kg;
h	is the height of fall of the hammer in m;
A	is the area at the base of the cone in m^2;
e	is the average penetration, imm per blow ($0,1/N_{10}$ from DPLm DPM, and DPH, and $0,2/N_{20}$ from DPSH);
m'	is the total mass of the extension rods, the anvil and the guiding rods, in kg.

(2) The value of r_d is an assessment of the driving work done in penetrating the ground. To calculate q_d-values, the r_d-values are modified to take into account the inertia of the driving rods and the hammer after impact on the anvil. q_d should thus allow comparison of different equipment configurations.

(3) Appropriate correction factors may be applied to take into account the friction on the rods.

(4) Energy losses are originated during driving due to the same factors described for the SPT test (5.5.2). Therefore, when this test is used for quantitative evaluation purposes, it is recommended to know by calibration the actual energy ratio ER_r transmitted to the drive rods.

6.6 Reporting of results

(1)P In addition to the requirements given in 2.6 the test report shall include the following information:

- the type of dynamic probing: DPL, DPM, DPH or DPSH; all divergences from the normal procedures of applied these tests shall be described in detail;
- a graphic representation with respect to depth of the following data:
  - number of standard blows to drive the cone 100 mm for the DPL, DPM and DPH or 200 mm for the DPSH, and/or the values of r_d or q_d;
  - the maximum torque required to rotate the penetrometer at each test level (in Nm), if measured;
  - all interruptions during the work, longer than 5 minutes;
- the use of any separate precaution against friction such as casing, drilling mud or water;
- the use of corrections to take account of friction along the rods, if any;
- details of any unusual event during driving, e.g. penetration without blows, temporary obstructions, artesian conditions.

6.7 Derived values of geotechnical parameters

(1) If dealing with cohesionless soils, it is possible to obtain correlations with some geotechnical parameters and in situ tests and use the results in a quantitative evaluation for foundations design, provided the friction along the rods is negligible, or duly corrected.

(2) If dealing with cohesive soils, the quantitative use of the results is more controversial and should be employed only under well known local conditions and supported by specific correlations. The skin friction during the test is a factor of special concern with this type of soils and should be duly taken into account.

(3) Several correlations have been established among the different dynamic probing tests and between them and other tests or geotechnical parameters. In some cases the friction along the rods has been eliminated or corrected, but the actual energy transmitted to the probe has not been measured. Therefore they cannot be considered valid in general. Nevertheless, some of them are included in annex E.

7 WEIGHT SOUNDING TEST (WST)

7.1 General

(1)P The weight sounding penetrometer consists of a screw-shaped point, rods, weights or other loading system and a handle or a rotating device. The weight sounding test is made as a static sounding in soft soils when the penetration resistance is less than 1 kN. When the resistance exceeds 1 kN the penetrometer is rotated, manually or mechanically, and the number of halfturns for a given depth of penetration is recorded.

(2) The weight sounding test is primarily used to give a continuous soil profile and an indication of the layer sequence. The penetrability in even stiff clays and dense sands is good.

(3) The weight sounding test may also be used to estimate the density of cohesionless soils and to estimate the undrained shear strength of cohesive soils.

(4)P The tests shall be carried out in accordance with a method that complies with the requirements given in this section.

(5)P The test method used shall be reported in detail with the test results.

(6) The test method may be reported by reference to a standard.

(7)P Any deviation from the requirements given below shall be justified and in particular its influence on the results of the test shall be commented upon.

7.2 Definitions

(1)P weight sounding resistance: either the smallest standard load for which the penetrometer sinks without rotation, or the number of halfturns per 0,2 m of penetration when the penetrometer has its maximum load and is rotated.

(2)P manual weight sounding test: test made by rotating the penetrometer by hand using a handle. The penetrometer is loaded by weightpieces.

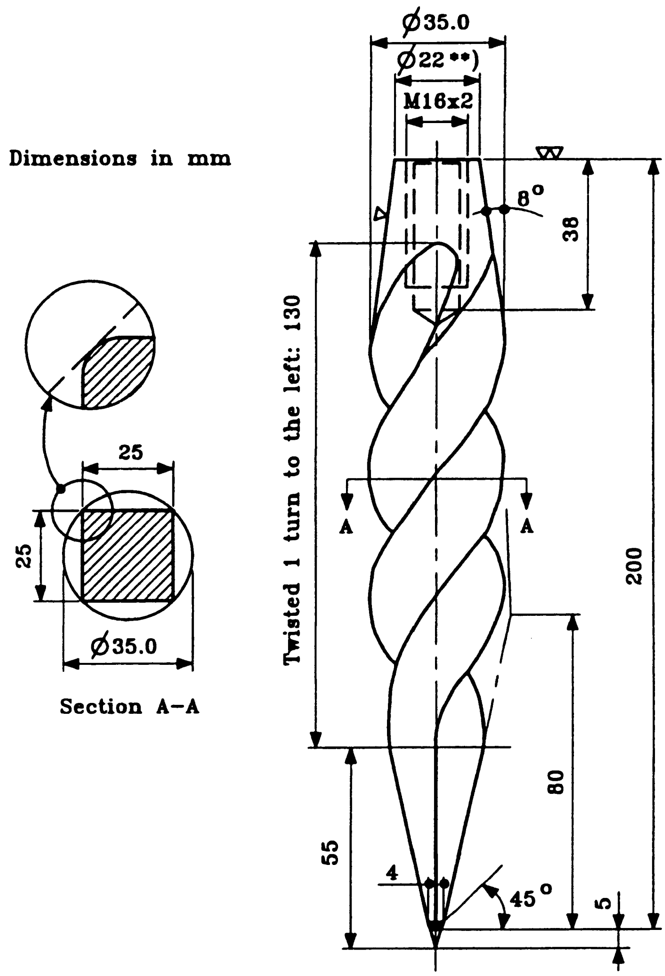
(3)P mechanized weight sounding test: test in which rotating of the penetrometer is made mechanically. The penetrometer is loaded mechanically or by dynamometer or by weightpieces.

7.3 Equipment

7.3.1 Penetrometer tip

(1)P The diameter of the circumscribed circle of the screw-shaped point shall be 35 mm. The length of the point shall be 200 mm. The point is twisted one turn to the left over a length of 130 mm and it has a pyramidical tip as shown in figure 7.1.

(2)P The diameter of the circumscribed circle for the worn point shall not be less than 32 mm. Maximum allowable shortening of the point is 15 mm due to wear. The tip of the point shall not be bent or broken.



**) for 22 mm rods

Figure 7.1: The point of the weight sounding penetrometer

7.3.2 Weights or loading system

(1)P The weight-pieces for manual weight sounding test shall comprise of one 50 N clamp, two 100 N weights and three 250 N weights, total 1000 N.

(2) The weights for mechanized test can be replaced by a dynamometer with the measuring range from 0,05 kN up to 1,00 kN.

(3)P The maximum allowable deviation from the standard loads and the dynamometer scale shall be ± 5 %.

7.3.3 Rods

(1)P The diameter of the rods shall be 22 mm.

(2) The length of one rod should be at least 1 m. For mechanized weight sounding test the length of the rod can be 1,0 to 2,0 m.

(3)P The deviation from the straight axis shall not exceed 4 mm per m for the lowest 5 m of the rods and 8 mm per m for the remainder. The allowable eccentricity of the coupling shall not be more than 0,1 mm. The angular deviation for a joint between two straight rods shall not be more than 0,005 rad.

7.4 Test procedure

7.4.1 Use of preboring and casing

(1)P The possible need to prebore through the upper stiff or dense soil layers shall be estimated in each case.

(2) Preboring is often required through a dry crust or through a fill in order to minimize skin friction along the rods.

7.4.2 Manual weight sounding

(1)P When the penetrometer is used as a static penetrometer in soft soils the rod shall be loaded in steps using the following standard loads: 0 kN, 0,05 kN, 0,15 kN, 0,25 kN, 0,50 kN, 0,75 kN, 1,0 kN. The maximum standard load is 1,0 kN.

(2)P The load shall be adjusted in the standard steps to give a rate of penetration of about 50 mm per second.

(3)P If the penetration resistance exceeds 1 kN or the penetration rate at 1 kN is less than 20 mm per second the rod shall be rotated. The load of 1 kN is maintained and the number of half turns required to give 0,2 m of penetration shall be counted.

(4)P The rod must not be rotated when the penetration resistance is less than 1 kN.

(5) The sounding may be terminated by striking the rod with a hammer or by dropping some of the weights onto the clamp in order to check that the refusal is not temporary.

7.4.3 Mechanized weight sounding

(1)P The test shall be carried out in a similar manner as for the manual sounding. The rate of rotation shall not exceed 50 turns per minute.

(2) The rate of rotation should be between 15 and 40 turns per minute.

(3) The applied load should be measured by a dynamometer or a measuring cell attached to the machine.

(4)P During the sounding, vibrations from the engine shall be kept in such level, that it does not affect the measured penetration resistance.

7.5 Interpretation of test results

(1)P The penetration resistance is given by the standard loads in stages (kN) and when rotated, loaded with the maximum standard load, by the number of halfturns per 200 mm of penetration.

(2) Differences between manual and mechanical operated tests may occur. Where this may be the case, e.g. when estimating the relative density of loose cohesionless soils, comparisons between manual and mechanized tests are recommended.

(3) The penetration resistance is strongly influenced by the shaft friction of the rods.

7.6 Reporting of the test results

(1)P In addition to the requirements given in 2.6 the test report shall include the following information:

- sounding method;
- the type of the loading device;
- the type of the rotating equipment and the rate of rotation;
- preboring, diameter and depth of the borehole, if preboring is used;
- diameter of the casing tube and depth of casing, if used;
- diameter of the rods;
- the penetration depth for every standard load during the static sounding phase;
- the number of halfturns required for every 0,2 m of penetration during the rotating phase; in cases when a full section of 0,2 m is not penetrated the number of halfturns and corresponding penetration;
- the depth of penetration and number of blows during driving if the penetrometer is driven by blows of a hammer or some of the weights;
- interruptions during the test;
- all observations which may help in the interpretation of the test results.

7.7 Derived values of geotechnical parameters

(1)P When the bearing capacity or the settlement of a spread foundation is evaluated from weight sounding test results and analytical method shall be used.

(2) When the sample analytical method for bearing capacity in annex B of ENV1997-1 is used, the angle of shearing resistance N' may be determined from correlations with weight sounding resistance. Such correlations should be based upon comparable experience, relevant to the design situation. Annex F presents a sample correlation, derived for quartz and feldspar in a European region.

(3) When the theoretical elastic method in annex D of ENV1997-1 is used for calculating settlements of spread foundations from weight sounding results, the drained (long term) Young's modulus E_m may be determined from weight sounding resistance on the basis of local experience. In the case of quartz and feldspar sands derived values as in annex F, for example, may be used to estimate a value of the angle of shearing resistance N' from the weight sounding resistance.

(4) In cohesionless soils the weight sounding resistance may be used also in direct estimation of the bearing capacity of spread foundation and piles.

(5) In cohesive soils the weight sounding resistance may be used to estimate the undrained shear strength of soil, based on local experience, considering the sensitivity of the soil and water conditions in the borehole.

8 FIELD VANE TEST (FVT)

8.1 General

(1)P The field vane test is an in situ test and it is carried out with a rectangular vane, consisting of four plates fixed at 90° angles to each other, pushed into the soil to the desired depth and rotated.

(2) This section covers the field vane test used in soft and very soft cohesive soils for the determination of the undrained shear strength and the sensitivity of the soil. Field vane test may also be used for the determination of the undrained shear strength in stiff clays, silts and clay till. The reliability of test results varies depending on the type of soil.

(3) After extensive rotation of the vane, whereby the soil along the failure surface becomes thoroughly remoulded, the remoulded shear strength value can be measured and the soil's sensitivity can be calculated.

(4)P The tests shall be carried out in accordance with requirements given in this section.

(5)P The test method used shall be reported in detail with the test results.

(6) The test method may be reported by reference to a standard.

(7)P Any deviation from the requirements given below shall be justified and in particular its influence on the results of the test shall be commented upon.

(8) Experience of deviations exists with respect to shape of vanes.

8.2 Definitions

(1)P maximum torque $T_{max,u}$: torque required to obtain failure along the failure surface. M_{max} is the torque for undisturbed strength value.

(2)P maximum torque for remoulded conditions T_{max}: torque required tot obtain failure along the failure surface is remoulded conditions.

(3)P test depth: depth at mid height of the vane.

(4)P time to failure: time between the first application of torque to the vane until the moment when maximum torque is reached when measuring the undisturbed strength value.

(5)P undisturbed shear strength value (c_v): the shear strength value in undisturbed conditions.

(6)P remoulded shear strength value (c_{rv}): shear strength value after remoulding the soil.

(7)P sensitivity according to field vane test (S_{tv}): relation between the undisturbed shear strength value and the remoulded shear strength value.

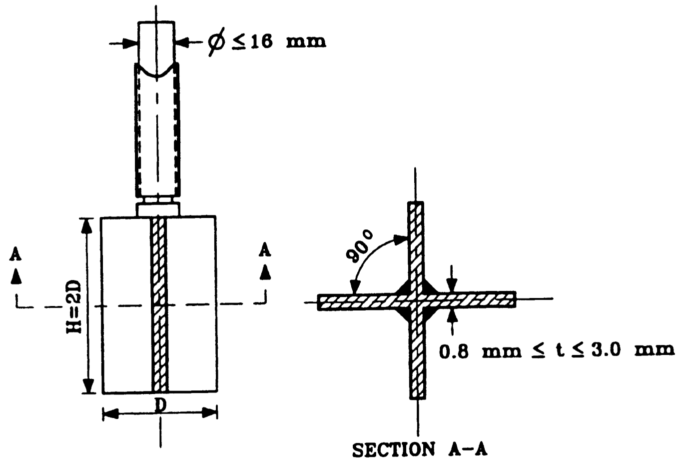


Figure 8.1: Design of the vane

8.3 Equipment

8.3.1 Vane

(1)P The vane shall consist of four rectangular plates fixed at 90° angles to each other, see figure 8.1. The blades shall be parallel with the extension rods and no distortion is allowed.

(2)P The relation between the height (*h*) and the diameter (*d*) of the vane shall be 2,0 for standard vanes.

(3) A maximum vane size (*d* x *h*) of 100 mm x 200 mm for very soft soils and a minimum vane size of 40 mm x 80 mm for dense soils are commonly used for standard vanes.

(4)P The blade thickness (*t*) shall not exceed 3,0 mm but shall not be less than 0,8 mm. The diameter of the vane shaft, as well as possible welding seams in the centre of the vane shall be small enough to avoid disturbing effects to the measured shear strength value.

(5) In very sensitive clays the blade thickness should not exceed 2,0 mm to minimize the disturbance of the soil during pushing the vane into the soil.

(6)P If the vane is filled with a protective casing, the length of the protrusion at the test shall be at least 5 times the diameter of the vane.

(7)P The diameter of the vane shaft close to the vane should be less than 16 mm. However, vane shaft shall be of such rigidity that it does not twist significantly under full load conditions.

(8) It is recommended that the vane is filled with a device making it possible to separate the torque of the vane from that of the extension rods. A casing or slip coupling can be used.

8.3.2 Extension rods

(1)P Extension rods shall have a diameter and torsional stiffness large enough to transmit the torque generated during the test to the vane.

(2) The extension rods should have a diameter of at least 20 mm.

(3)P The rods shall be straight. The eccentricity of the threads at the rod joints shall be less than 0,1 mm. Then maximum permitted bending for rods or for two jointed rods is 2 mm over each 1 m of length, measured as height of arch.

(4)P If casing tubes are used in order to prevent buckling of the rods, the inner diameter of the tubes shall be large enough to minimize the friction along the rods. When using casing tubes the friction along the rods shall be measured.

8.3.3 Equipment for rotation and recording instrument

(1)P The equipment for rotation of the vane shall be designed to provide rotation at a given and constant rate.

(2)P The recording instrument shall be designed so that it is possible to read off accurately the maximum torque.

(3)P The recording instrument shall be calibrated at least once every six months or when it has been damaged, overloaded or repaired.

(4) Continuous automatic recording is recommended. For interpretation of the test results a graph of the measured torque versus the rotation angle should be made.

(5) The measuring range for the necessary measurements of the angle of rotation should be 360°, with a reading of 1°.

8.4 Test procedure

8.4.1 Predrilling and pushing down the vane

(1)P Predrilling shall be made through eventually possibly dry crust or fill when a vane test is to be carried out in the soil below such layers.

(2)P When using an outer system with a casing protecting the vane, the water pressure in the casing system shall be the same as that in the soil at the test level.

(3)P The vane shall be pushed down, if possible without use of blows or vibration. Rotation is never permitted. The pushing rate shall be constant and not exceeding 20 mm/s.

(4) In stiff clays, silts and in clay till driving may be needed to get the vane to the desired depth.

(5)P The distance between investigation points shall be at least 2,0 m in plan in case of test depths greater than 5 m.

(6)P The first test shall be conducted at a depth of at least 0,5 m below the ground surface or at a depth of at least 5 times the diameter of a predrilled hole below its base.

(7)P The minimum vertical distance for two tests conducted in the same borehole shall be at least 0,5 m.

8.4.2 Vane shear test

(1)P The time from the moment when the desired test depth has been reached to the beginning of the vane test (waiting time) shall be at least 2 min and no more than 5 min.

(2)P The vane shall be loaded by application of torque in such a way that failure of the soil occurs in undrained conditions. The vane shall be rotated at a constant rate.

(3) A guiding value for the rotating rate of the vane to fulfil the criteria given above in cohesive soils is 0,1°/s to 0,2°/s (6°/min to 12°/min).Rotation rates up to 0,5°/s may be used for soft cohesive soils with low sensitivity.

(4)P The test shall be run in such a way that the skin friction along the rods can be separated.

(5)P After failure has occurred and the maximum torque has been recorded, the vane shall be rotated rapidly at least ten turns in order to remould the soil at the failure surface, after which a new test is performed immediately according to the instructions above. The constant value of the torque at the remoulded state is recorded.

8.5 Interpretation of test results

(1)P For standard vanes with d/h = 1:2 and with the failure surface shown in figure 8.2, the undisturbed shear strength value of the soil shall be determined by using the formula:

$$c_{fv} = 0{,}273 \frac{T_{max;u}}{D^3}$$

where:

$T_{max;u}$ is the maximum torque on the vane.

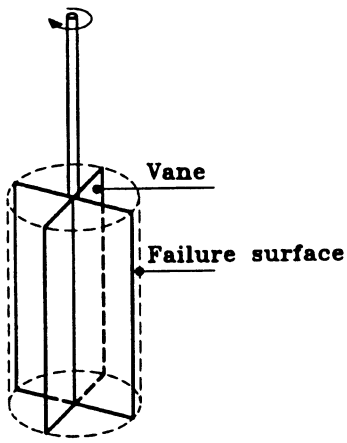


Figure 8.2: Assumed failure surface for standard vane

2)P The failure surface for a rectangular vane shall be assumed to be a cylindrical surface circumscribing the vane and two planes on the top and bottom of that cylinder respectively. The shear strength shall be assumed to be fully mobilized, constant and uniform at failure around the perimeter and across the ends of the cylinder, see figure 8.2.

(3)P The undrained shear strength (c_u) of the soil shall be obtained from the undisturbed shear strength value after correction with regard to the soil's liquid limit, plasticity index or the effective vertical stress.

(4)P $T_{max;u}$ shall be reduced by the friction along the rods.

(5)P The remoulded shear strength value (c_{rv}) shall be determined by the same formula replacing $T_{max;u}$ with $T_{max;r}$.

(6) By registering the torque as a function of torsion, information is obtained on the mode of the failure of the soil. By using different shapes of vanes, it is possible to evaluate the anisotropy of the soil.

8.6 Reporting of the results

(1)P In addition to the requirements given in 2.6 the test report shall include, for both manual and automatic recording, the following information:

– type of test equipment and of the torque measuring device;
– vane size;
– diameter of the extension rods;
– number of the torque measuring device (to check the calibration data);
– date of the last calibration and calibration factor for the instrument;
– test depth;
– rotation rate of the vane during the test;
– maximum torque on the vane in undisturbed and torque in remoulded state, a reading or a graph;
– time to failure (the time for the activated vane only);
– observations made in connection with the test, as well as events or details that may influence the test results.

8.7 Derived values of geotechnical parameters

(1)P If the bearing capacity of a spread foundation, the ultimate bearing resistance of piles or stability of slopes is evaluated based on vane test results, an analytical method shall be used.

(2)P In order to obtain derived values for the undrained shear strength c_{fu}, the undisturbed shear strength value shall be corrected with consideration to empirical experience. The correction factor shall be determined based on local experience.

(3) Existing correction factors are usually related to the liquid limit or plasticity index and the effective vertical stress.

(4) In annex G examples are given of such correction factors.

9 FLAT DILATOMETER TEST (DMT)

9.1 General

(1)P The flat dilatometer test DMT covers the determination of the in situ strength and deformation properties of fine grained soils using a blade shaped probe having a thin circular steel membrane mounted flush on one face.

(2) Results of DMT tests are mostly used to obtain information on soil stratigraphy, in situ state of stress, deformation properties and shear strength.

(3)P The basis of the test consists of inserting vertically into the soil a blade-shaped steel probe with a thin expandable circular steel membrane mounted flush on one face and determining, at selected depths or in a semi-continuous manner, the contact pressure exerted by the soil against the membrane when the membrane is flush with the blade and subsequently the pressure exerted when the central displacement of the membrane reaches 1,10 mm.

(4) The DMT test is most appropriate in clays, silts and sands where particles are small compared to the size of the membrane.

(5)P The tests shall be carried out in accordance with a method that complies with the requirements given in this section.

(6)P The test method used shall be reported in detail with the test results.

(7) The test method may be reported by reference to a published standard.

9.2 Definitions

9.2.1 Equipment and testing procedure

(1) The main parts of a dilatometer equipment are defined in figure 9.1.

(2)P dilatometer blade or dilatometer probe: blade-shaped steel probe that is inserted into the soil to run a DMT test.

(3)P membrane: circular steel membrane that is mounted flush on one face of the blade and is expanded when applying a gas pressure at its back.

(4)P switch mechanism: apparatus housed inside the blade, behind the membrane, capable to activate and disconnect an electric contact which in turn shall respectively set off and on an audio and/or visual signal when the membrane expands and reaches two preset deflections equal to 0,05 mm and 1,10 mm respectively.

(5)P pneumatic-electric cable: cable that connects the control unit to the blade, delivers gas pressure at the back of the membrane, and provides electric continuity between the control unit and the switch mechanism.

(6)P control and calibration unit: set of suitable devices capable of supplying gas pressure to the back of the membrane and measuring the pressure when the switch mechanism activates and disconnect the electric contact behind the membrane.

(7)P earth wire: wire connecting the control unit to the earth.

(8)P pressure source: pressurized gas tank filled with any dry nonflammable and noncorrosive gas.

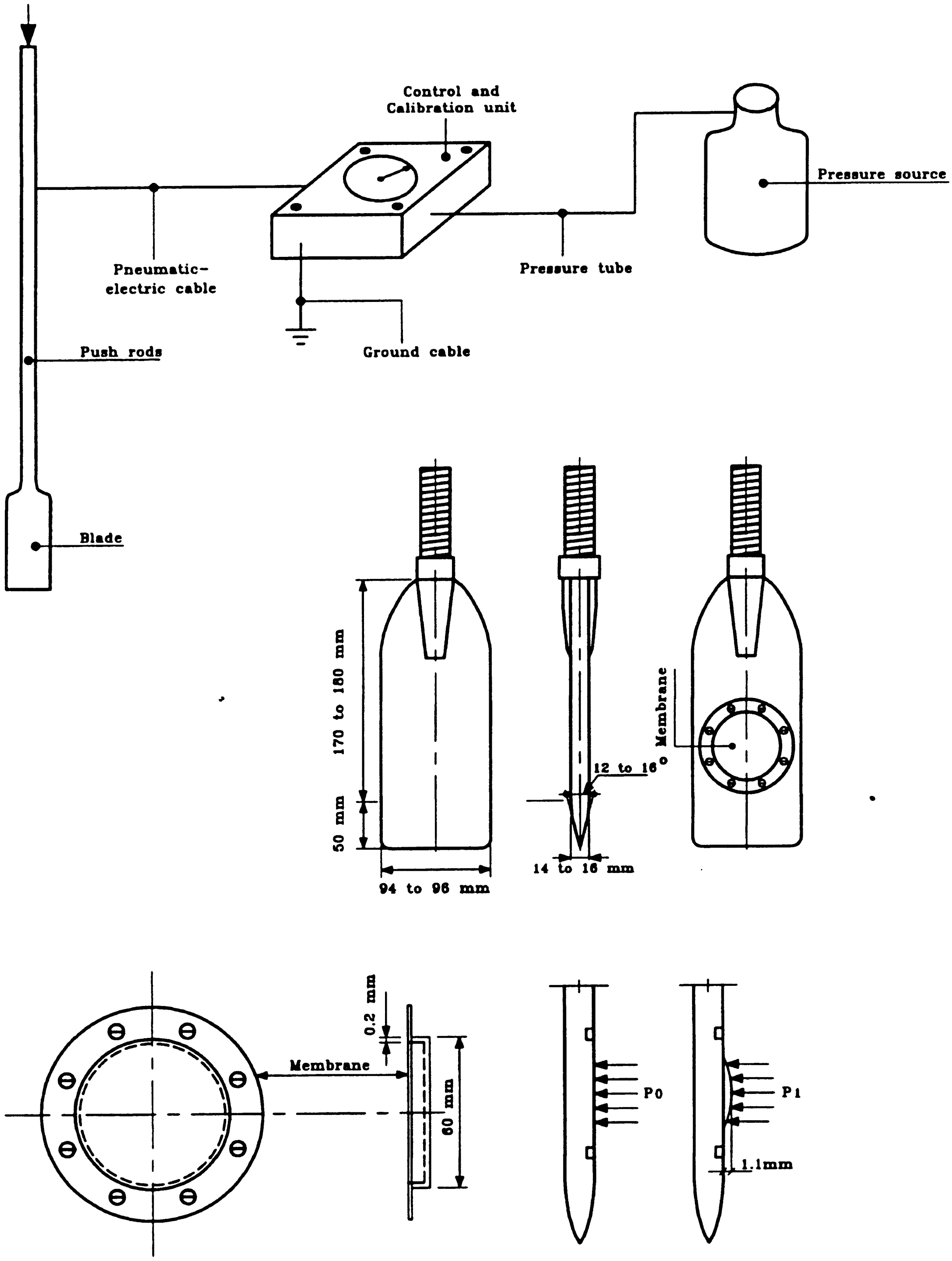


Figure 9.1: Dilatometer equipment and definition of calculated in situ soil pressure

(9)P membrane calibration: the procedure to determine the membrane calibration pressures equal to the suction and the pressure that must be applied in air to the back of the membrane to retract its centre to 0,05 mm expansion or to expand it 1,10 mm respectively.

(10)P dilatometer profiling: the execution of a sequence of dilatometer tests from the same station at ground level along a vertical direction at closely spaced intervals with depth increments ranging between 150 mm and 300 mm.

9.2.2 DMT parameters

(1)P The following parameters are defined:

- **A-pressure:** the pressure (P_A) that must be applied to the back of the membrane to expand its centre 0,05 mm in soil;
- **B-pressure:** the pressure (P_B) that must be applied to the back of the membrane to expand its centre 1,10 mm in soil;
- **A-membrane-calibration-pressure:** the suction, (ΔP_A) recorded as a positive value, that must be applied to the back of the membrane to retract its centre to the 0,05 mm deflection in air;
- **B-membrane-calibration-pressure:** the pressure (ΔP_B)that must be applied to the back of the membrane to expand its centre to the 1,10 mm deflection in air;
- **$\Delta P_{A;avg}$ and $\Delta P_{B;avg}$:** the averaged values of the membrane calibration pressures obtained from the respective values of ΔP_A and ΔP_B measured before and after each dilatometer profiling or single dilatometer test;
- **Z_m-pressure:** gauge pressure deviation from zero when venting the blade to atmospheric pressure;
- **soil pressure p_0:** soil pressure against the membrane when it is flush with the blade (e.g. at zero expansion), also termed contact pressure, as shown in figure 9.1;
- **soil pressure p_1:** soil pressure against the membrane when its centre is expanded 1,10 mm as shown in figure 9.1;
- **in situ pore water pressure prior to blade insertion u_0:** in situ pore water pressure prior to blade insertation at the elevation of the centre of the membrane;
- **the in situ effective vertical stress P_{v0}:** vertical stress prior to blade insertion at the elevation of the centre of the membrane;
- **dilatometer material index I_{DMT}:** an index related to the type of soil;
- **dilatometer horizontal stress index K_{DMT}:** an index related to the in situ horizontal stress;
- **dilatometer modulus E_{DMT}:** a parameter related from theory to the modulus of elasticity of the soil.

9.3 Equipment

9.3.1 Dilatometer equipment

(1)P The equipment for dilatometer testing shall comprise:

- dilatometer blade with suitable threaded adaptor to connect to push rods;
- pneumatic-electrical cable;
- earth wire;
- control and calibration unit;
- pressure source.

(2)P The dimensions of the blade, of the apex angle of the penetrating edge and of the membrane shall be comprised within the limits indicated in fig. 9.1.

(3)P The pneumatic-electrical cable, which provides pneumatic and electrical continuity between the control unit and the dilatometer blade, shall have stainless steel connectors with wire insulators to prevent short circuit and washers to prevent gas leakage.

(4)P The control and calibration unit shall accomplish the following:

- it shall be earthed;
- it shall control the rate of gas flow while monitoring and measuring the pressure of gas transmitted from the control unit to the blade and the membrane;
- it shall signal the instants when the electric switch changes from on to off and vice versa.

(5)P The pressure measurement devices of the control and calibration unit shall allow to determine the pressure applied to the membrane with intervals of 10 kPa and a reproducibility of 2,5 kPa at least for pressures lower than 500 kPa.

(6)P The pressure source shall be provided with a suitable regulator, valves and pressure tubing to connect to the control unit.

9.3.2 Insertion apparatus

(1)P The equipment for inserting the dilatometer blade shall comprise:

- thrust machine to insert and advance the dilatometer blade into the soil;
- push rods with suitable adaptor to connect to the blade;
- hollow slotted adaptors for lateral exit of the pneumatic-electrical cable.

(2)P The thrust machine shall be capable of advancing the blade vertically with no significant horizontal or torsional forces.

(3) Penetration rates in the range of 10 mm/s to 30 mm/s should be applied. Driving should be avoided except when advancing the blade through stiff or strongly cemented layers which cannot be penetrated by static push.

(4)P Push rods shall be straight and resistand against buckling.

9.4 Test procedure

9.4.1 Calibration and checks

(1)P All the control, connecting and measuring devices shall be periodically checked and calibrated against a suitable reference instrument to assure that they provide reliable and accurate measurements.

(2)P The dilatometer blade and membrane shall be checked before penetrating in the soil. The blade shall be mounted axially with the rods. It shall be planar and coaxial and have a sharp penetration edge. The membrane shall be clean of soil particles, free of any deep scratches, wrinkles or dimples and expand smoothly in air upon pressurization.

(3)P The maximum out of plane deviation of the blade, defined as the maximum clearance under a 150 mm long straight edge placed along the blade parallel to its axis, shall not exceed 0,5 mm; the maximum coaxiality error of the blade, defined as the deviation of the penetration edge from the axis of the rods to which the blade is attached, shall not exceed 1,5 mm.

(4)P The control unit and the tubing shall be checked for leaks before starting a sequence of dilatometer profilings by plugging the blade end of the pneumatic-electrical cable and checking for any pressure drop in the system. Leakage in excess of 100 kPa/min shall be considered unacceptable and shall be repaired before testing begins.

(5) With the dilatometer equipment assembled and ready for testing the switch mechanisms should be checked by hand pushing the membrane flush with the blade to check that the audio and/or visual signals on the control unit are activated.

9.4.2 Membrane calibration procedure

(1)P The membrane shall be calibrated to measure the values of the ΔP_A -suction and ΔP_B -pressure with the dilatometer equipment assembled and ready for testing immediately before inserting the blade into the soil and upon retrieval to the ground surface, both when running a dilatometer profiling or even a single test.

(2)P If the values of the membrane calibration pressures ΔP_A and ΔP_B, obtained before penetrating the blade into the soil, fall outside the limits ΔP_A = 5 kPa to 30 kPa and
ΔP_B = 5 kPa to 80 kPa respectively, the membrane shall be replaced before testing commences.

(3) After a membrane has been replaced, the new one should be exercised to improve the stability of the ΔP_A and ΔP_B values. Such exercising may consist in pressurizing the membrane in air to 500 kPa for a few seconds. Care should be taken to avoid overexpansion and permanent deformations of the membrane.

(4)P After any membrane calibration the values of ΔP_A and ΔP_B shall be promptly recorded. All the obtained values of ΔP_A and ΔP_B shall be available on site.

(5)P During calibration the audio and/or visual signal activated by the electric switch shall stop and return sharply and unambiguously while sensing the 0,05 mm and 1,10 mm expansions respectively.

(6) When testing soft soils the membrane calibration procedure should be performed more than once to assure that stable values of ΔP_A and ΔP_B, falling within the prescribed limits, are constantly determined.

9.4.3 Performing the test

(1)P After the blade has been inserted into the soil and advanced to the selected test depth, the load applied to the push rods shall be released and the blade pressurized without delay to expand the membrane.

(2)P The rate of gas flow to pressurize the membrane shall be such that the P_A- reading is obtained within 20 s from reaching the test depth and the P_B- reading is obtained within 20 s after the P_A-reading.

(3)P Once P_B- has been determined the membrane shall be depressurized immediately, in order to prevent further expansion and permanent deformations, and the blade advanced to the next test depth or retrieved to the ground surface.

(4) Depending on the system used to advance the blade, the pneumatic-electric cable connected to the blade should be pre-threaded through the push rods for protection or left outside, using a slotted adaptor to egress it, and taped to the rod every 1 m.

(5) If a friction reducer is used to limit the thrust needed to advance the blade, it should be located at least 200 mm above the centre of the membrane.

(6)P After the blade has been retrieved to the ground surface and the membrane calibration procedure performed the values of ΔP_A and ΔP_B shall be recorded and compared with those measured previously. If the values of ΔP_A and ΔP_b measured before inserting the blade into the soil and after retrieval to the ground surface differ by more than 25 kPa then the test performed between the two successive calibration procedures shall be discarded.

9.5 Interpretation of the results

(1) Results of DMT tests can be interpreted using well established correlations to determine the subsoil stratigraphy, the deformation properties of cohesionless and cohesive soils, the in situ state of stress and the undrained shear strength of cohesive soils.

(2) The interpretation of the results of DMT tests requires a knowledge of the in situ pore water pressure u_0 and the effective vertical stress σ'_{v0} prior to blade insertion. The value of u_0 at any test depth should be determined from reliable pore water pressure measurements. The value of σ'_{v0} at any test depth should be estimated from the unit weight of the soil layers above that depth.

(3) When interpreting the results of DMT tests the values of p_0, p_1, u_0 and σ'_{v0} should correspond consistently to the same test location and membrane depth.

(4) The soil pressure p_1 against the DMT membrane when its centre is expanded 1,10 mm should be determined using the following relationship:

$$p_1 = p_B - \Delta P_{B;avg} - Z_m.$$

(5) The soil pressure p_0 against the DMT membrane when its centre is flush with the blade should be determined with a linear backextrapolation from the soil pressure against the membrane at the two preset deflections, 0,05 mm and 1,10 mm, hence using the following relationship:

$$p_0 = 1{,}05\ (P_A + \Delta P_{A;avg} - Z_m) - 0{,}05\ p_1.$$

(6) The material index I_{DMT}, the horizontal stress index K_{DMT} and the dilatometer modulus E_{DMT} should be calculated using the following relationships:

– $I_{DMT} = (p_1 - p_0) / (p_0 - u_0)$,
– $K_{DMT} = (p_0 - u_0) / \sigma'_{v0}$,
– $E_{DMT} = 34{,}7\ (p_1 - p_0)$.

9.6 Reporting of the results

(1)P In addition to the requirements given in 2.6 the test report shall include the following information:

– rig and rod types;
– characteristics of systems used to advance the blade;
– predrilling depth and system to support the borehole if any;
– diameter and location of friction reducer if used;
– thrust applied to the push rods and at the top of the blade if measured;
– elevation of the groundwater table;
– procedures to calculate the pore pressure against the membrane at each test elevation;
– characteristics of the measuring system to obtain the in situ pore pressure when relevant;
– type and size of dilatometer blade and membrane;
– zero readings of pressure measurement devices;
– values of the ΔP_A - and ΔP_B -calibration-pressures measured before and after each dilatometer profiling or single test and corresponding average values;
– tabulated output of values of P_A- and P_B-pressure-readings;
– tabulated output of the calculated values of p_0- and p_1-pressure;
– any relevant observation of the operator such as incidents, equipment damage during testing, repairs and replacements, details not included in the above list which may affect the interpretation of test results.

9.7 Derived values of geotechnical parameters

9.7.1 Bearing capacity of spread foundations

(1)P When the bearing capacity of spread foundations is evaluated based on DMT results, an analytical method shall be used.

(2) When the sample analytical method of annex B in EC 7 Part 1 is used, the derived value of the undrained shear strength c_u of non cemented clays, for which the DMT test results show $I_{DMT} < 0{,}8$, can be determined using the following relationship:

$$c_u = 0{,}22\ \sigma'_{v0}\ (0{,}5\ K_{DMT})^{1{,}25}$$

or any other well documented relationship based on local experience.

9.7.2 Settlement of spread foundations

(1) When applying the adjusted elasticity method of annex D in ENV1997-1 the one dimensional settlement of spread foundations may be calculated using values of the one dimensional tangent modulus E_{oed} determined from results of DHT tests as shown in annex H. In cohesive soils such procedure should be applied when the stress increase induced by the foundation load is less than the preconsolidation pressure.

9.7.3 Pile foundations

(1)P When the ultimate bearing resistance of piles is evaluated from DMT results an analytical calculation method shall be applied to derive the values of base and shaft resistance.

10 ROCK DILATOMETER TEST (RDT)

10.1 General

(1)P This test covers the determination of the in situ deformability of rocks from measurements of the radial expansion of a borehole section under a known uniform radial pressure applied by means of a cylindrical dilatometer probe.

(2)P The basis of the test consists of inserting a cylindrical probe, having an outer expandable flexible membrane, into a borehole, and measuring, at selected intervals or in a semi-continuous manner at closely spaced test locations, the radial displacement of the borehole while inflating the probe under a known radial pressure.

(3) This test is used mainly in hard rock formations to obtain profiles of deformability variations with depth.

(4)P The tests shall be carried out in accordance with a method that complies with the requirements given in this section.

(5)P The test method used shall be reported in detail with the test results.

(6) The test method may be reported by reference to a published standard.

10.2 Definitions and symbols

10.2.1 Dilatometer probe and testing apparatus

(1) The main components of a dilatometer apparatus for testing in rocks are defined in fig. 10.1.

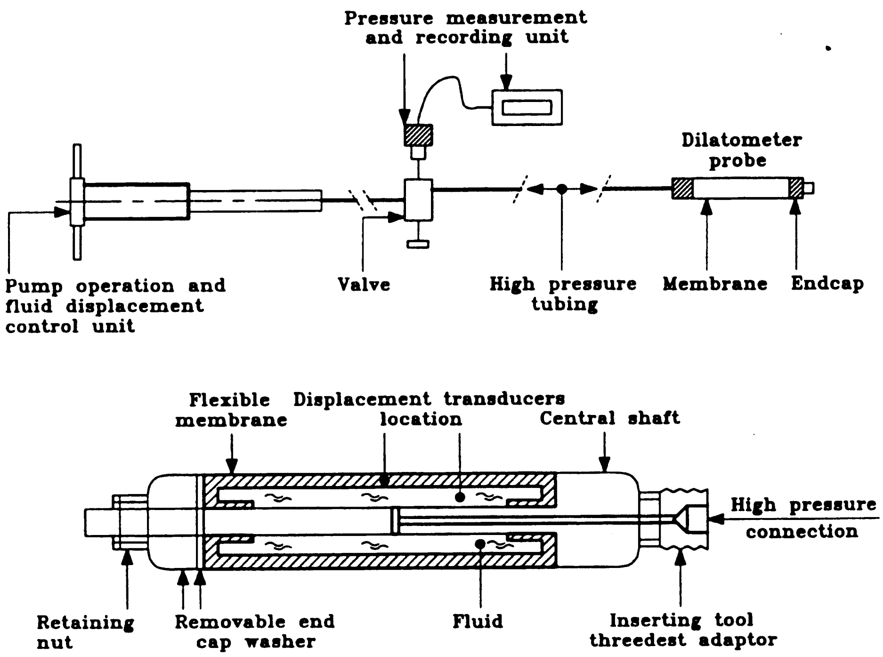


Figure 10.1: The main components of a dilatometer apparatus in rocks

(2)P probe: cylindrical device consisting of an outer flexible membrane mounted on an inner steel core that can be filled with a pressurized fluid so that the membrane can be inflated from the inside and expanded against the wall of the borehole to apply a uniform radial pressure.

(3)P membrane stiffness: pressure that shall be applied inside the probe to inflate and expand the membrane in air.

(4)P membrane compression: change in thickness of the membrane as the pressure inside the probe is increased.

(5)P calibration cylinder: cylinder made of a material of known elastic properties and having an inner diameter slightly larger than the deflated diameter of the probe and length equal to that of the probe.

(6)P hydraulic system stiffness: increase of pressure applied at ground surface to the dilatometer apparatus that corresponds to a given volume increase of the hydraulic pressurizing system measured at ground surface.

(7)P borehole dilation ΔD: average increase of borehole diameter D along a test section under the applied pressure;

(8)P seating pressure p_s: minimum pressure that shall be applied before expanding the probe to ensure permanent contact of the flexible membrane with the borehole wall with no sliding;

(9)P pressure-dilation graph: plot of applied pressure and corresponding dilation such as p_i versus ΔD, p_i versus probe volume.

10.2.2 Symbols of dilatometer parameters

(1)P The following parameters are defined:

- p_i: the pressure inside the probe during testing;
- p_c: the pressure applied to the borehole wall during testing;
- ν_z; the Poisson's ratio of the tested rock;
- E_d: the dilatometric modulus of deformation of a borehole test section;
- E_{d1}: the secant dilatometric modulus of a borehole test section corresponding to a finite increase of pressure Δp_c.

10.3 Equipment

10.3.1 Drilling equipment

(1)P The drilling equipment shall be capable of providing a clean and smooth walled borehole having a diameter slightly larger than the deflated diameter of the probe and smaller than the diameter of the probe when expanded in air under the expected range of pressures that will be applied during testing.

(2) When necessary the borehole wall should be supported by casing, except in the test section, or by cementation and subsequent redrilling. During cementation the head of fluid grout should not exceed 3 m to avoid pressure grouting. After redrilling and prior to testing the cement lining should be thinner than 1 mm.

10.3.2 Pressurizing system

(1)P The hydraulic system to pressurize the dilatometer probe shall be capable of storing and displacing the necessary volume of fluid to fill, inflate and deflate the probe, applying all specified ranges of fluid pressure and also performing cyclic loading and stress relaxation testing when required.

(2) For measurements in hard rock formations the application of pressures as high as 20 MPa should be expected.

(3)P The stiffness of the hydraulic system shall be such that the dilation of the system is minimal if the radial displacements of the borehole are backcalculated from measurements of volume changes taken at ground level.

(4) Glycerin, ethylene glycol, water and hydraulic oil should generally be used as pressurizing fluids.

10.3.3 Measuring system

(1)P The measuring system shall be capable of determining the applied fluid pressure with a sensitivity better than 2% and the volume of fluid displaced to pressurize the probe with an accuracy of 1% of the probe volume over the full range of pressures to be employed during the testing program.

(2)P When borehole displacements are obtained from direct measurements against the borehole wall, as with displacement transducers, the measuring device shall be capable of determining the borehole diameter with an accuracy of at least 0,02 mm.

10.3.4 Calibration equipment

(1)P At least one calibration cylinder of known elastic properties with internal diameter equal to that of the borehole and length equal to that of the probe shall be available on site to determine the stiffness of the dilatometer testing system.

(2) The calibration cylinder should have a stiffness similar to that of the rock mass to be tested and should allow calibration over the full range of pressures specified for the testing program.

(3) Two or more calibration cylinders of different stiffness should be used to improve accuracy and precision of the calibration procedure.

10.3.5 Dilatometer probe

(1)P The probe shall be fitted with the necessary components to ensure pressure and leakage control, system bleeding and connection with rods, high pressure tubing and cable to pressurize and position the probe inside the drillhole.

(2) The flexible membrane should be strong enough not to be damaged when inserted into and withdrawn from the borehole but flexible enough to transmit not less than 90% of the pressure acting inside the probe.

(3)P For testing in a hard rock formation a probe equipped with the necessary measuring devices that allow to measure borehole dilations directly against the borehole wall shall be used. The measuring devices shall be mounted at mid point of the membrane and equally spaced.

(4) Probes not equipped with the necessary devices to measure borehole dilations directly against the borehole wall should be used only when testing in weak rocks or when large deformations compared to the probe volume are expected.

10.4 Test procedure

10.4.1 Calibration and checks

(1)P All measuring devices shall provide reliable and accurate measurements.

(2)P Prior to performing each test series, and periodically during a testing program or after major repair, the complete dilatometer equipment shall be thoroughly checked, with the probe and the hydraulic and pressurizing systems filled with the pressurizing fluid, thoroughly bled to remove any entrapped air and pressurized to check for leaks.

(3)P The probe shall be inflated in air, without confinement, and measurements taken to obtain an unconfined pressure-dilation relationship from which the stiffness of the flexible membrane shall be determined.

(4)P When using probes fitted with displacement transducers that measure the dilation of the membrane relative to the inner cove, instead of relative to the borehole wall the membrane compression shall be determined prior to testing.

(5)P When using equipment that allows measurements of fluid pressure and of volume of displaced fluid at ground surface only, the hydraulic system stiffness shall be determined prior to testing.

(6)P The hydraulic system stiffness shall be determined from knowledge of the stiffness of the calibration cylinder and of the overall stiffness of the hydraulic system plus calibration cylinder.

(7)P The overall stiffness of the hydraulic system plus calibration cylinder shall be determined from a pressure - dilation graph obtained with the probe inserted in the calibration cylinder by increasing the fluid pressure in increments and measuring the corresponding volume of displaced fluid. The pressure shall be increased through the range of pressures specified for the required testing program to obtain at least five measurements of pressure and corresponding volume of displaced fluid.

10.4.2 Performing the test

(1)P After the hole is completed the diameter and clearance shall be checked and recorded prior to inserting the probe.

(2) Borehole clearance should be checked using a cylindrical gauge of the same size as the probe. The wall of the borehole should be checked for fissures, voids and rock fragments, that might damage the flexible membrane or trap the probe.

(3)P The position of the probe at the required test depth shall be measured with an accuracy of ± 50 mm and recorded.

(4) When a continuous profile of deformability is required the vertical depth increment between successive test elevations should vary between 1 m and 5 m.

(5)P After the probe has been positioned at the required test depth, it shall be inflated to apply at first the seating pressure that shall be the minimum pressure throughout the test.

(6)P With the probe seated the pressure shall be increased in at least five approximately equal increments to the maximum possible value not exceeding the safe operation pressure of the testing equipment.

(7)P Measurements of the applied pressure and of the volume of displaced fluid or of the borehole dilation shall be taken and recorded at each pressure increment. Each increment shall be held for a period sufficiently long to determine whether the rock behaviour is time dependent.

(8) Time dependent behaviour and creep rates should be evaluated by maintaining the volume of fluid inside the probe constant while the drop in pressure with time is recorded or by maintaining the fluid pressure constant while the increase of borehole dilation with time is recorded.

(9)P The maximum pressure shall be maintained constant for at least 10 min. and measurements of volume of displaced fluid or of borehole dilation shall be taken and recorded throughout this length of time.

(10) Measurements of the applied pressure and of the volume of displaced fluid or of the borehole dilation should also be taken and recorded during unloading.

10.5 Interpretation of the results

(1) The results of cylindrical dilatometer tests can be used to determine the deformation properties and the creep properties of the in situ rock at the test location in intact rocks.

(2) In fragile or clayey rocks and in fractured or closely jointed formations, where core recovery is poor or inadequate for purpose of obtaining representative samples for laboratory testing, the cylindrical dilatometer test can be used for rapid index logging of boreholes and for comparisons of relative deformabilities of different rock strata.

(3) The interpretation of cylindrical dilatometer tests requires a knowledge of the Poisson's ratio of the tested rock.

(4) In hard and non fractured rocks and when the applied pressure and corresponding borehole dilation are measured directly against the borehole wall the dilatometric modulus of deformation E_d of a borehole test section can be determined from the slope of the straight portion of a pressure-strain graph for the range of pressures greater than the seating pressure as shown in figure 10.2.

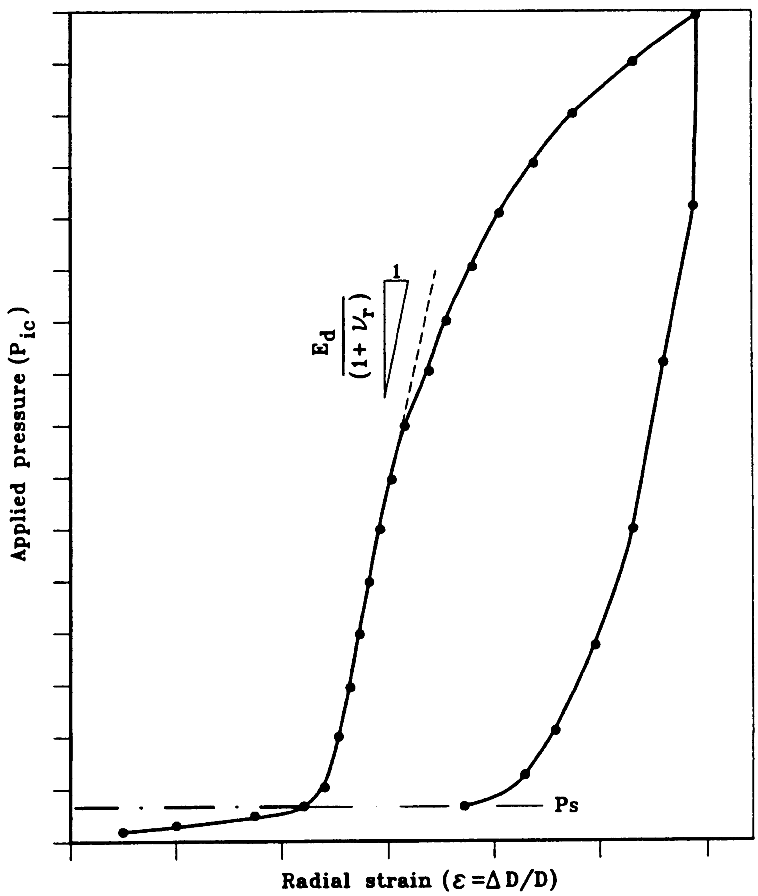


Figure 10.2: Definition of dilatometric modulus (P_s = seating pressure)

(5) The value of the secant dilatometric modulus E_{d1} expressed in MPa and corresponding to a pressure increment Δp_c can be determined from the graph shown in figure 10.2 or using the following relationship:

$$E_{d1} = (1 + \nu_r) \times D \times (\Delta p_c / \Delta D)$$

where:

Δp_c is the pressure applied to the bore hole wall, in Mpa;
ΔD is the bore hole dilatation, in m;
D is the borehole diameter prior to testing, in m.

(6)P When the applied pressure is measured inside the probe the value of p_c shall be determined from the corresponding value of p_i corrected to account for membrane stiffness before determining the values of E_d and E_{d1}.

(7)P When borehole dilation is determined from measurements against the internal wall of the membrane, these measurements shall be corrected to account for membrane compression before determining the values of E_d and E_{d1}.

(8)P When the pressure applied to the borehole wall and the corresponding borehole dilation are determined from measurements of fluid pressure and of volume of displaced fluid taken at ground surface these measurements shall be corrected to account for hydraulic system stiffness, membrane stiffness and membrane compression.

10.6 Reporting of the results

(1)P In addition to the requirements given in 2.6 the test report shall include the following information:

- borehole diameter and length;
- detail of the drilling program including method and equipment used;
- a geotechnical log of the drill core showing rock type and properties, ground water level, location of test sections and of cased and cemented sections, if any;
- details of equipment and procedure for testing and calibration together with results of calibration including ambient air temperature at time of calibration;
- tabulated test readings of the applied pressure and corresponding volume of displaced fluid or of the measured borehole dilation for each transducer obtained at each test depth within the boreholes and for each pressure range and time interval;
- plots of pressure-dilation curves obtained during calibration and testing for each test depth and for each displacement transducer inside the probe when relevant;
- plots of pressure-dilation curves obtained during calibration and testing for each test depth corrected for system stiffness, membrane stiffness and membrane compression when relevant.

10.7 Derived values of geotechnical parameters

(1) The results of dilatometer tests may be used to check against the serviceability limit state of spread foundations on rocks through a deformation analysis.

(2) When performing a deformation analysis the Young's modulus E may be taken equal to E_d on the assumption that the rock is linearly elastic and isotropic.

11 PLATE LOADING TEST (PLT)

11.1 General

(1)P The plate loading test covers the determination of the vertical settlement and strength properties of soil and rock masses in situ by recording the load and the corresponding settlement when a rigid plate is loading the ground.

(2) This section covers plate loading tests carried out on a thoroughly levelled ground surface or on the bottom of an excavation at a certain depth or the bottom of a prebored, large diameter borehole.

(3) The test is applicable in all soils, fills and rocks but is normally not suitable for very soft cohesive soils.

(4)P The test shall be carried out in accordance with a method that complies with the requirements given in this section.

(5)P The test method used shall be described in detail with the test results.

(6) The test method may be described by reference to a published standard.

(7)P Any deviation from the requirements given below shall be justified and in particular its influence on the results shall be commented upon.

(8) Experience with deviations exists with respect to

- plate size;
- test procedure (incremental loading, constant rate of deformation).

11.2 Definitions

(1)P applied contact pressure p: contact pressure equal to the applied load including the weight of the apparatus acting directly on the plate and the weight of the plate divided by the area of the base of the plate.

(2)P ultimate contact pressure p_u: largest possible contact pressure or contact pressure where the settlement reaches a specified level or increase.

11.3 Equipment

11.3.1 Apparatus

11.3.1.1 Plate

(1)P The plate shall be rigid to avoid bending, and nominally flat on the bottom. The top shall contain a guide to locate the loading column, particularly where the test is to be made in a bore hole. The longitudinal axis of the loading column and the centre of the plate shall coincide and the contact shall be provided with a bearing.

(2)P When performing test loading of a plate in very heterogeneous ground, the plate shall be of such a size that it is not influenced by any random firm or weak spot. The width of the plate (b) shall be at least five, preferably ten times larger than the largest spot.

(3) In this section circular and square shaped plates are treated. For deriving settlement and strength properties, circular plates are preferably used. For direct design, square plates are normally used. If failure is to occur in a certain direction, this can be achieved by using a rectangular test plate. The ratio of the smaller dimension of the rectangular plate and the larger dimension should be more than 0,8; the smaller dimension should normally be more than 1 m.

(4) For circular plates, diameters or more than 0,6 m are normally used.

11.3.1.2 Reaction loading system

(1)P The reaction load shall be designed such that the required contact pressures below the plate can be produced and the required settlements can be achieved.

(2) The reaction load may be created by jacking against a counterweight, against tension piles or anchors or against an existing abutment (see figure 11.1).

(3)P The reaction load or its supports shall be placed sufficiently far from the proposed test position so as to reduce the influence on the results to an acceptable level.

(4) Between the centre of the plate and the reaction system, a distance of 3,5 times the width or diameter of the plate is normally sufficient.

(5)P The loading column shall have sufficient strength to prevent undue buckling under the maximum load and in the case of borehole tests shall be clear of the borehole walls.

(6) The loading column should have ball-joint connections to the reaction beam and to the plate, provided the horizontal stability is secured.

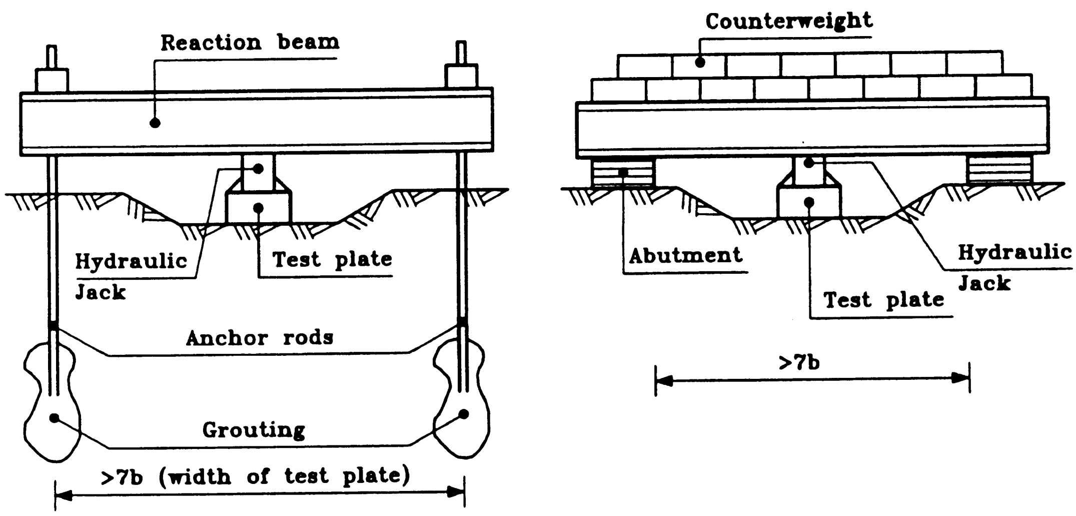


Figure 11.1: System for load application, examples

11.3.2 Measuring equipment

11.3.2.1 Load measurement system

(1)P A load measurement device shall be placed between the reaction loading system and the test plate which shall be calibrated to the expected maximum test load and the required accuracy of the test.

(2)P The accuracy of the load measuring system shall allow to measure any applied load with an accuracy of 5 %.

(3) The load measuring system should be free of hysteresis effects.

11.3.2.2 Settlement measuring system

(1)P The settlement measurements of the centre of the plate shall be made with reference to fixed points which shall not be influenced by the loading of the plate or the reaction loading system.

(2)P The accuracy of the complete settlement measurement system shall be within 2 % or at least 0,1 mm.

(3) If dial gauges or electrical displacement transducers to measure the settlement are used, they should be fixed to a measuring frame which is sufficiently stiff in order to avoid creep and vibration in the frames

(4) The measuring bridge should be protected against wind, sun and frost.

(5) If large plates of more than 1 m width are used, it is recommended to measure the settlements with a telescope level possessing an accuracy of 0,1 mm. Two fixed reference points are to be chosen which are close enough but outside the area influenced by the loading.

(6) The settlement measurement system should be able to measure the average settlement as well as the tilting of the plate. At least a three-point measuring system should be centrically located on the plate; the three measuring points should be equally spaced around the plate.

11.4 Test procedure

11.4.1 Pre-test explorations of ground conditions

(1)P In case the existing ground conditions of the test location are not known, they shall be determined to a depth of more than 5 times (preferably 8 times) the diameter or width of the plate below the test level.

(2) For rock, a depth of less than 5 times the diameter, or the width of the plate may be sufficient for laboratory investigations.

11.4.2 Calibration and checks

(1)P The manometers, the load transducer system and the electric displacement transducers shall be calibrated at least every six months and be checked before each field operation.

(2)P The dial gauges and the displacement transducer system shall be checked before every test at the site by inserting a measuring block of known size under the tip of the gauge.

11.4.3 Preparation of test area

(1)P The ground shall be undisturbed if the test is conducted in natural soil or rock. Disturbed material shall be removed.

(2)P The contact surface of the soil or rock to the plate shall be smooth and horizontally levelled.

(3) If necessary this can be achieved by fill-in material with a higher strength than that of the ground. For tests on cohesive soils, as soon as possible after levelling the ground, the paste of a quick-setting plaster should be poured and spread to obtain a levelled surface not more than 20 mm thick. Immediately after the paste has been spread, the plate should be bedded.

(4) For tests on granular soils, any hollows should be filled with clean dry sand to produce a levelled surface on which to bed the plate.

(5) The final preparation of the test level, in both pits and boreholes should be made by hand, if possible.

(6)P In case several tests are to be conducted within a given area, the distance between the centres of neighbouring plates shall be at least 6 times the diameter or width of the plates.

(7) In the case of a plate of cast in situ concrete, no special preparation of the contact surface of the soil is necessary; in the case of rock, the surface should be cleaned by hand from debris.

11.4.4 Preparation and setting up of loading and measuring apparatus

(1)P When tension piles are used for the reaction, they shall be installed before the test area is exposed. Tension piles shall not influence the test ground.

(2)P The test plate shall not be preloaded during erection of the reaction and force measurement systems. The loading column shall be positioned centrally over the plate and perpendicularly so that the reaction load is applied direct to the plate without eccentricity.

11.4.5 Loading test

11.4.5.1 Incremental loading test

(1)P An incremental loading test shall be conducted if drained loading properties of the soil are to be derived.

(2)P The load shall be applied in equal increments (about ten steps) and shall be kept constant for a certain time within each increment.

(3)P In cohesive soils, primary consolidation shall have taken place at the end of each load increment; this shall be checked by evaluating the time-settlement curves including the creep at the end of each increment.

(4) In non-cohesive soils, each load step is normally applied for 8 min. and the settlements are measured after 1, 2, 4 and 8 min. or continuously 1 to 2 times per min. with an automatic measurement equipment. Sometimes, each load step is applied for 16 min. instead of 8 min.

(5) In the case of tests for direct design, the load increment up to the assumed working load may be extended over a longer time period (one to two months) in order to estimate longterm behaviour.

(6)P The test shall be conducted using loading and unloading cycles if an indication of the relative amounts of reversible and permanent settlements has to be obtained.

(7) While unloading, the decrements may be twice as large as in the case of loading.

(8) Unloading and reloading cycles should be applied before the critical load is reached; complete unloading should be avoided.

11.4.5.2 Constant rate of penetration test

(1) A constant rate of penetration test may be conducted if undrained loading properties of cohesive soil are to be derived. The rate of settlement should be chosen according to the permeability of the soil and to the size of the plate in order to ensure undrained conditions during the test.

(2) The test should be conducted until a settlement equal to fifteen percent of the plate diameter or width has been reached.

11.5 Interpretation of the results

(1) The results of a PLT are presented as applied contact pressure (p_u) versus settlement curves and an indication of the ultimate contact pressure (see figure 11.2).

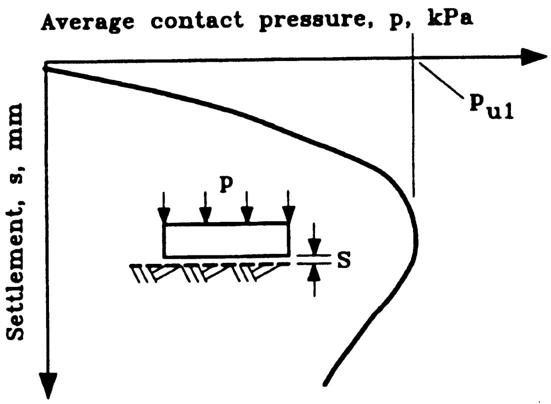


Figure 11.2: Relation between applied contact pressure and settlement for a plate founded on sensitive clay, or dense sand

(2) The ultimate contact pressure (p_u) from the PLT results may be taken as:

- the largest possible pressure p_{u1}, in sensitive clays or dense sands, (see figure 11.2);
- the pressure p_{u2} at which the creep $s_2 = s(t + \Delta t) - s(t)$ increases considerably (see figure 11.3);
- equal to the pressure p_{u3} at a defined settlement e.g. equal to 15 % of the diameter or width of the plate.

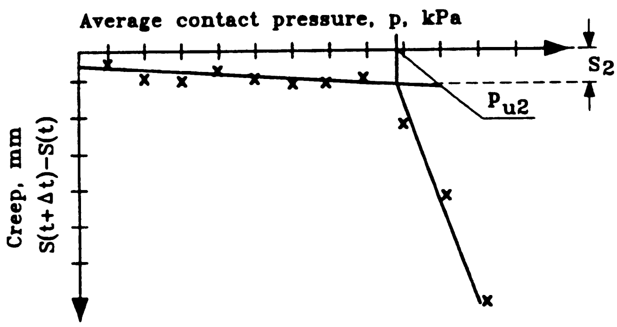


Figure 11.3: Evaluation of the ultimate contact pressure at creep,
(p_{u2} can be an alternative method to determine the ultimate contact pressure p_u)

11.6 Reporting of results

(1)P In addition to the requirements given in 2.6 the report shall include the following information:

- number of the plate;
- size of the plate;
- location;
- method of gaining access to plate test position, e.g. by borehole, test pit etc.;
- size/area of test pit, diameter of borehole etc.;
- elevation of the test plate;
- time (start and end of test);
- description of tested soil or rock;
- description of the test arrangements (reaction and loading systems and settlement measurement system);
- calibration data and checks of measuring equipment;
- rate of settlement during each load increment (loading programme);
- time and applied contact pressure dependent readings of settlements
- applied contact pressure versus settlement graphs including loading and unloading cycles;
- creep during each increment or loading cycle;
- time versus settlement graphs from load increments of interest;
- time versus temperature (at the test plate) readings;
- any relevant observations of the operator which may affect the interpretation of test results.

11.7 Derived values of geotechnical parameters

(1) The results of a PLT may be used to predict the behaviour of spread foundations.

(2) For deriving geotechnical parameters of a homogeneous layer (for use in indirect design), the layer should have a thickness beneath the plate of at least 2 times the width or diameter of the plate.

(3) Results of PLT may only be used for direct design if

- the size of the plate has been chosen considering the width of the planned spread foundation (in which case the observations are transformed directly);
- a homogeneous layer up to 2 times the width of the planned spread foundation exists (in which case the results of smaller sized plates - not considering the planned foundation width - are used to transform the results on an empirical basis to the actual foundation size).

(4) If the sample analytical method for bearing capacity of annex B of ENV 1997-1 is used, the undrained shear strength c_u may be derived from a PLT conducted at a constant rate of penetration.

(5) In annex I.1 an example of the assessment of the derived value of c_u is given.

(6) If the sample adjusted elasticity method for settlement evaluation of annex D of ENV 1997-1 is used, the Young's modulus E_m may be derived from the plate settlement modulus E_{PLT}, based on established experience.

(7) In annex I.2 the determination of E_{PLT} is shown.

(8) The coefficient of subgrade reaction k_s for evaluating settlements may be derived from results of an incremental loading test.

(9) In annex I.3 an example is shown for the calculation of k_s.

(10) For direct design, the results of PLT may be transferred directly to the foundation problem without using any geotechnical parameters.

(11) In order to determine the settlement behaviour of strip foundations from PLT results directly, the plate size is recommended to be of a certain ratio to the footing width.
Recommended plate sizes are tabulated below for different footings:

strip footing	width [m]	0,5	1,0	1,5
rectangular test plate	area [m^2]	1,0	2,25	4,0

(12) In case plate sizes smaller than defined in 11.7(11) have to be used, the settlements of the footing in sand may be derived according to annex I.4.

12 SOIL SAMPLING

12.1 `General

(1)P The aim of soil sampling is to obtain samples for soil identification and for laboratory testing to determine geotechnical properties of the ground.

(2) Important soil properties needed in geotechnical design are strength and deformation properties. In the laboratory these properties can be obtained reliably only from high quality undisturbed samples, representative of each soil layer. From granular materials it is possible to obtain undisturbed samples only using special methods not covered in this section. Strength and deformation properties of these soils are usually supported by in-situ tests and confirmed by disturbed samples. Soil classification properties may be obtained also from disturbed samples.

(3) Commonly used laboratory tests for soil identification and determination of geotechnical properties of soils are covered in ENV 1997-2

12.2 Catagories and concepts

12.2.1 Categories of sampling methods

(1) There are three categories of sampling methods:

- category A sampling methods;
- category B sampling methods;
- category C sampling methods.

(2) By using category A sampling methods, the intention is to obtain samples in which no or only slight disturbance of the soil structure has occurred during the sampling procedure or in handling of the samples. The water content and the void ratio of the soil correspond to that in situ. No change in constituents or in chemical composition of the soil has occurred.

(3) By using category B sampling methods, samples contain all the constituents of the in situ soil in their original proportions and the soil has retained its natural water content. The general arrangement of the different soil layers or components can be identified. The structure of the soil has been disturbed.

(4) By using category C sampling methods, the soil's structure in the sample has been totally changed. The general arrangement of the different soil layers or components has been changed so that the in situ layers cannot be identified accurately. The water content of the sample may not represent the natural water content of the soil layer sampled.

12.2.2 Area ratio and inside clearance of the sample tube

(1)P The area ratio C_a (%) of the sample tube is defined by the following relation:

$$C_a = \frac{D_2^2 - D_1^2}{D_1^2} \times 100$$

where:

D_1 is the inside diameter of the cutting shoe;
D_2 is the greatest outside diameter of the cutting shoe.

(2) The area ratio is one of the factors that determine the mechanical disturbance of the soil, being the ratio of the volume of soil displaced by the sampler tube in proportion to the volume of the sample.

(3)P The minimum length of the tube where the area ratio shall be applied is two times the outside diameter of the tube or cutting shoe, see figure 12.1.

(4) Thin-walled samplers have a sample tube with the area ratio equal or less than 15 % and thick-walled open-tube more than 15 %.

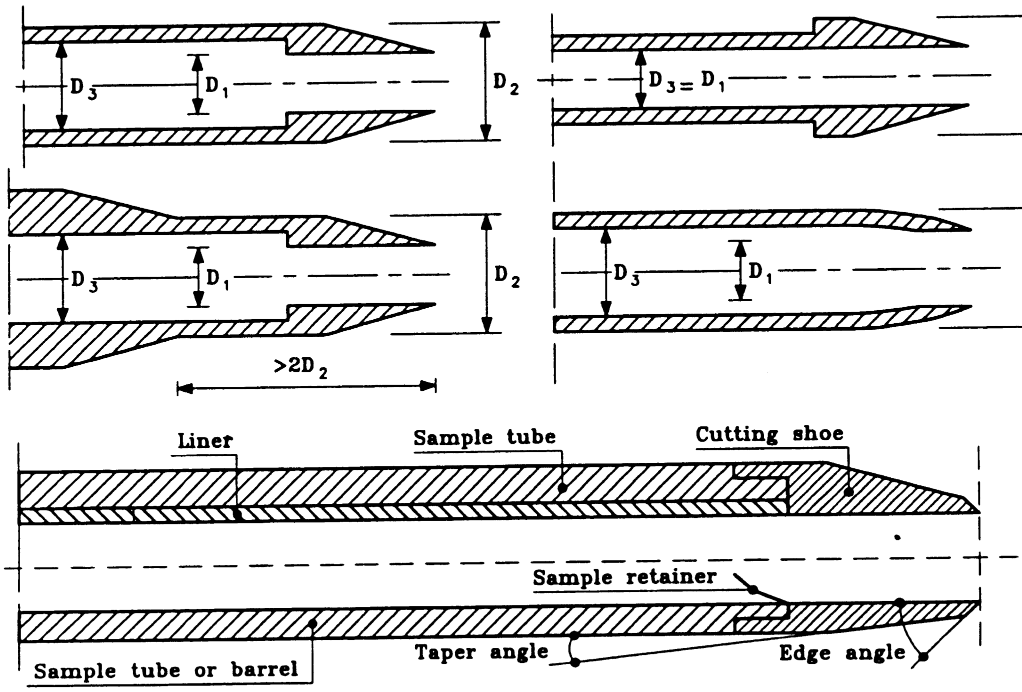


Figure 12.1: Definitions and measures of sample tubes

(5)P The inside clearance ratio C_i (%) of the sample tube is defined as:

$$C_i = \frac{D_3 - D_1}{D_1} \times 100$$

where:

D_1 is the inside diameter of the cutting shoe;
D_3 is the inside diameter of the sample tube or liner.

(6) The inside clearance ratio is one of the factors that determine the mechanical disturbance of the soil caused by the friction on the inside wall of sample tube or of the liner.

12.2.3 Techniques for sampling

(1) Techniques for obtaining soil samples can be divided in four groups defined as follows:

- drive sampling, in which a tube or a split-tube sampler having a sharp cutting edge at its lower end is forced into the ground either by a static thrust (by pushing), by a dynamic impact or by percussion. Drive samplers are usually open tube samplers or piston samplers. Drive samplers are mostly used as category A or B sampling methods;
- rotary core sampling, in which a tube with a cutter at its lower end is rotated into the ground, thereby producing a core sample. Rotary core samplers can have a single, double or triple tube rotary core barrel with or without a liner. Rotary core sampling is usually used as category B sampling methods, in some cases as category A sampling methods. Rotary core sampling can be replaced by vibrocoring;
- auger sampling with hand or mechanical augers. Augers are usually used as category C and rarely as category B sampling methods;
- block sampling made by hand cutting from a trial pit, shaft or heading or at depth using specially made block samplers with cutting procedure. Block sampling is mostly used as category A sampling methods.

12.2.4 Quality classes of soil samples related to sampling catagories

(1) According to 2.3 of ENV 1997-2 soil samples for laboratory tests are classified in five quality classes with respect to the soil properties that are assumed to remain unchanged during sampling and handling, and the catagory of sampling that may be used. The classes are described in table 12.1.

12.3 Equipment

12.3.1 Basic requirements for samplers

(1)P A sampler shall have the technical details in order to ensure the following:

- to get the soil into the sampler and keep it there during withdrawal of the sampler;
- to obtain the sample with a minimum of disturbance of the soil material during the actual sampling and before and after this operation.

(2)P A sampler shall be detailed such that it is possible to obtain samples of a required quality according to 2.3 of ENV 1997-2 from the soil in which the sampler will be used.

Table 12.1: Quality classes of soil samples for laboratory testing

Soil properties / quality class	1	2	3	4	5
Unchanged soil properties					
particle size	*	*	*	*	
water content	*	*	*		
density, density index, permeability	*	*			
compressibility, shear strength	*				
Properties that can be determined					
sequence of layers	*	*	*	*	*
boundaries of strata - broad	*	*	*	*	
boundaries of strata - fine	*	*			
Atterberg limits, particle density, organic content	*	*	*	*	
water content	*	*	*		
density, density index, porosity, permeability	*	*			
compressibility, shear strength	*				
Sampling category to be used	A	A	A	A	A
			B	B	B
					C

12.3.2 Requirements for drive samplers

12.3.2.1 General

(1)P Drive samplers used as category A sampling methods shall fulfil alll requirements presented in 12.3.2.2 to 12.3.2.7.

(2) Fulfilling these requirements does not guarantee that totally undisturbed samples will always be obtained, but it should at least minimize the disturbance of samples and thus the effect of such disturbance on the results of laboratory tests performed on these samples.

12.3.2.2 Diameter of the sample tube

(1)P For catagory A sampling methods, sample tubes with the inside diameter less than 70 mm, and in soft clays less than 50 mm shall not be used. The cross section of the sample tube shall be circular.

(2) For catagory A sampling methods, sample tubes having an inside diameter of 100 mm are recommended.

12.3.2.3 Length of the sample tube

(1)P For category A the maximum tube length for samplers used sampling methods shall be such that the inside wall friction and adhesion during the actual sampling do not have too much influence on the results of the laboratory test to be performed.

(2) A sample tube length of 2 to 6 times the inside diameter of the tube should be used for category A sampling methods. Sample tubes for cohesive soils may often be somewhat longer.

12.3.2.4 Wall thickness of the sample tube

(1)P The wall thickness of the sample tube shall be adequate to resist distortion while it is driven into the soil. A sample tube shall, however, be thin enough to minimize the disturbance of soil caused by displacement when the tube is inserted into the soil.

(2) In some samplers and soils liners inside the sample tube may be used.

(3)P For category A sampling methods the area ratio of the sample tube shall be kept as small as possible consistent with the strength requirements of the tube.

(4) For category A drive sampling methods in soft sensitive clays an area ratio of less than 15 % is recommended.

(5) For category A sampling methods in non-sensitive cohesive soils the area ratio of the sample tube should be less than 30 %.

12.3.2.5 Shape of the cutting edge

(1)P The cutting edge of the cutting shoe shall be sharp, and the taper angle as narrow as practicable.

(2) The taper angle should be specified in relation to the area ratio. For a sampler with an area ratio exceeding 15 % the taper angle should be less than 10° except close to the edge where the edge angle may be increased in order to avoid damaging the feather edge.

12.3.2.6 Inside clearance

(1)P The inside of the sample tube or liner shall be clean and smooth without any protruding edges or irregularities which may disturb the soil.

(2)P The inside clearance shall be sufficient to allow some lateral expansion of the sample to take place, but it shall not be so much that it permits excessive deformations and causes unnecessary disturbance of the sample.The inside clearance shall not be so much that it completely eliminates the inside wall friction thereby causing loss of the sample during withdrawal or loss of lateral support thereafter.

(3) For catagory A sampling methods it is recommended to have an inside clearance ratio of 0,5 % to 1,0 % for sample tubes used in cohesive soils and up to 3,0 % in other types of soils.

12.3.2.7 Sample retainer

(1) A sample retainer between the cutting edge and the sample tube may be used in soils that are difficult to sample, but preferably not for cohesive soils. The use of a sample retainer causes disturbance of the sample.

12.4 Sampling procedure

12.4.1 Selection of sampling method

(1)P The sampling category and sampling method shall be selected in advance in accordance with the quality of the sample required for classification of soils and for the laboratory test to be performed.

(2) The sample diameter for soils containing large particles should be chosen by the size of the largest particles of the sampled material.

(3) Different disturbance of sample may be expected when using various sampling methods. The quality of a sample taken with the same sampler may vary depending on e.g. the soil type to be sampled, the presence of groundwater and the sampling operation.

(4) The sampling, transportation and storage procedures should be taken into account when selecting samples for laboratory tests and interpreting the laboratory test results.

(5) Examples of some of the most common sampling methods and their suitability to different categories of sampling methods in different soils are listed in annex K.

12.4.2 Drive sampling operation

(1)P In order to obtain good quality soil samples in stiff soils or when using open-tube drive samplers, a method of preboring shall be used. A boring technique shall be selected which will minimize soil disturbance. Boreholes in unstable soils shall be stabilized with casing or using drilling fluid.

(2)P When using open-tube drive samplers as category A sampling methods, or in boreholes without casings in soft cohesive soils and in cohesionless soils below the groundwater table, the borehole shall be filled with water or drilling fluid. The level of the fluid shall be maintained at or slightly above the level of the groundwater table.

(3) When using category A sampling methods above the groundwater level the borehole should preferably be kept dry. The borehole may be filled with drilling fluid, but with water only in case of artesian groundwater.

(4)P In order to obtain samples from the bottom of a prebored borehole by category A sampling methods the hole shall be cleaned of disturbed soil and segregated material. When using open-tube drive samplers in a cased borehole the hole shall be cleaned to the edge of the casing.

(5) No cleaning is required when the borehole is uncased and the closed piston sampler is forced into undisturbed soil.

(6)P A piston sampler shall be forced through the zone of disturbed soil before the piston is released and the actual sampling is started.

(7)P When using category A sampling methods in soft to firm cohesive soils the drive sampler shall be forced into the soil to the predetermined depth by continuous pushing. There shall be no rotation of the sampler during the downward movement.

(8) In stiff cohesive soils the sampler may be forced into the soil by a dynamic impact which may, however, cause additional disturbance of the sample.

(9)P The sampler shall be withdrawn slowly and carefully and not rotated.

(10) It is advisable to withdraw the sampler after waiting a few minutes so that sufficient adhesion can develop between the sample and the sampling tube or liner.

(11)P In sampling category A after the sampling tube has been disconnected from the sampler head seriously disturbed material from the upper part of the sample shall be removed. The top of the sample shall be sealed and the open space at the tube top shall be filled out e.g. by remoulded material, foil and sealed.

(12)P Samples obtained by category A and B sampling methods shall be sealed immediately after retrieval to prevent change in water content.

12.4.3 Handling and storing of samples

(1)P Samples obtained by category A sampling methods shall be handled in such a way that no disturbance will occur after withdrawal of the sample or during transportation and storage.

(2) Samples obtained by category A sampling methods should effectively be protected from heat, frost, vibration and shock during shipment and storage. Great care should be taken not to remove the seals of the samples and not to deform the sampling tubes.

(3) Samples obtained by category A sampling methods should be stored during the transportation in a container and in the laboratory in a room where the temperature and humidity are kept constant and as close as possible to that of the in situ conditions.

12.5 Reporting on boring and sampling

12.5.1 Boring record

(1)P In addition to the requirements given in 2.6 the boring record shall contain the following information:

- date of boring;
- coordinates and number of boring;
- ground surface elevation at each boring or other reference level e.g. top of coating;
- the depth of the free groundwater level;
- the method of the preboring if used;
- the use of the casing and the depth of the casing edge of each sampling;
- the use of drilling fluid and the level of the drilling fluid in the borehole;
- the specifications and the type of sampler used, e.g. reference for standard if applicable;
- the diameter of each sample;
- the depth of the top and bottom of the sample;
- boring progress; any obstructions and difficulties encountered during the sampling operation.

12.5.2 Labelling of samples

(1)P All samples shall be numbered, recorded and labelled immediately after retrieval from a borehole or excavation and sealing.

(2)P The sample label shall show information about

- identification data of the project;
- date of the sampling;
- number of the bore hole;
- the sampling method and the type of the sampler used;
- elevation or depths of the top and the bottom of the sample.

(3)P Samples shall be properly marked so that there can be no doubt about the top and bottom of the sample.

(4) The soil type according to the visual examination should be given on the label, whenever possible.

12.5.3 Sampling report

(1)P The field report of sampling operations included in the site investigation report shall be clear and accurate and it shall contain not only the data required for determination of the soil profile and the location of the samples obtained but also any observations which will contribute to an estimate of the condition of the samples and of the physical properties of the soil in situ.

(2)P Unsuccessful sampling operations shall be reported.

13 ROCK SAMPLING

13.1 General

(1)P The aim of rock sampling is to obtain adequate samples for rock identification and for laboratory testing for reliable rock mechanical information on the stratum.

(2) Important rock properties needed in geotechnical design are structure, strength and deformation properties. These properties may be appropriately obtained by description of and laboratory testing on high quality samples representing the actual stratum.

13.2 Catagories and concepts

13.2.1 Sampling categories

(1)P Adequate samples shall contain all the mineral constituents of the strata from which they have been taken. Such samples shall not have been contaminated by any material from other strata nor from additives used during the sampling procedure which distinguished from the sample.

(2) There are three categories of sampling methods for description and laboratory testing:

- category A sampling methods;
- category B sampling methods;
- category C sampling methods.

(3) By using category A sampling methods it is intended to obtain samples in which no or only slight disturbance of the rock structure has occurred during the sampling procedure or in handling of the samples. The strength and deformation properties, water content, density, porosity and the permeability of the rock sample correspond to the in situ values. No change in constituents or in chemical composition of the rock mass has occurred.

(4) By using category B sampling methods the samples contain all the constituents of the in situ rock mass in their original proportions and the rock pieces have retained their strength and deformation properties, water content, density and porosity.

(5) By using category B sampling the general arrangement of discontinuities in the rock mass may be identified. The structure of the rock mass has been disturbed and thereby the strength and deformation properties, water content, density, porosity and permeability for the rock mass itself.

(6) By using category C sampling methods the structure of the rock mass and its discontinuities has been totally changed. The rock material may have been crushed. Some changes in constituents or in chemical composition of the rock material may occur. The rock type and its matrix, texture and fabric may be identified.

13.2.2 Visual rock identification in the field

(1)P Visual rock identification shall be based on examination of the rock masses and samples including all observations of decomposition and discontinuities.

(2)P Weathering classification shall be related to the geological processes and shall cover the grades between fresh rock and rock decomposed into soil.

(3) A simplified weathering classification subdivided in six grades is given in annex L.

(4)P Discontinuities are bedding planes, joints, fissures, cleavages and faults and shall be quantified with respect to pattern, spacing and inclination using unambiguous terms.

13.2.3 Rock recovery

(1)P Rock quality designation, RQD is the sum length of all core pieces that are 100 mm or longer, measured along the centre line of the core, expressed as a percentage of the length of the core run.

(2)P Solid core recovery, SCR is the length of core recovered as solid cylinders, expressed as a percentage of the length of the core run.

(3)P Total core recovery, TCR is the total length of core sample recovered, expressed as a percentage of the length of the core run.

13.2.4 Area ratio and inside clearance of the sample tube

(1)P The area ratio and inside clearance defined in 12.2.2 are determining the mechanical disturbance during sampling of rock strata in which a noticeable part is decomposed.

13.3 Equipment

13.3.1 Basic requirements for samplers

(1)P A sampler shall be adequatly technical detailed in order to:

- get the rock mass into the sampler and keep it there during breaking off the sample and retrieval of the sampler;
- obtain samples with the accepted degree of disturbance of the actual rock mass after the sampling operation.

13.3.2 Techniques for sampling

(1) The most suitable methods for obtaining rock mass samples is related to the structure and the decomposition grade of the rock mass and the required laboratory quality class:

- category A or B: rotary sampling in which a tube with a cutter at its lower end is rotated into the rock mass, thereby processing a core sample;
- category A or B: drive sampling in which a tube or a split-tube sampler having a sharp cutting edge at its lower end is forced into highly or completely weathered rock mass either by a static thrust or by a dynamic impact. Drive samplers are usually piston samplers or open tube samplers;
- category C: shell or auger sampling, where the sample is taken from the actual drilling tool;
- category C: cuttings sampling in which the rock mass, remoulded and crushed, by cable or rod handled percussion or cutting tools is brought to the surface by means of bailer or circulation of a transporting substance;
- category A: block sampling made by hand cutting from a trial pit, shaft or heading or using specially made block samplers.

(2)P Selection of the technique shall be made in accordance with the sample quality required for classification of the rock mass and for the laboratory test to be performed.

(3) For rock masses decomposed to completely weathered rock or residual soil all the sampling techniques described in 13.3.2 may be applied.

(4) Requirements and sampling procedure for the drive sampling technique for use in highly or completely weathered rock are similar to the requirements and procedures described for soil sampling in 12.3.2 and 12.4 to which reference is made.

(5) For fresh rock and masses decomposed less than to completely weathered rock or residual soil the following techniques may apply:

– rotary sampling;
– cuttings sampling;
– block sampling.

13.3.3 Requirements for rotary samplers

(1)P Rotary samplers for category A sampling in rock masses shall fulfil general requirements related to discontinuities in the rock mass and the grade of weathering as presented in (3) to (16).

(2) Fulfilling these requirements cannot guarantee that undisturbed samples will always be obtained. On the contrary it is normally not possible to core out undisturbed samples from rock masses decomposed more than slightly, especially when discontinuities are dominating the rock structure. However, fulfilling the requirements should at least minimize the disturbance of the samples.

(3) Single-tube core barrels rotate directly against the core and this sampling method may only be category A in case the rock mass is fresh and without discontinuities.

(4) Double-tube core barrels with ball bearing swivel between the outer tube and the inner tube which do not rotate against the core may, when used in fresh and slightly weathered rock masses, constitute a sampling method corresponding to category A and at least category B.

(5) Triple-tube core barrels containing a liner inside the inner tube remaining stationary will, when used in moderately and highly weathered rock masses with discontinuities, normally constitute a sampling method corresponding to category B. When the rock mass is fresh, slightly weathered or converted into a hard soil with apparent cohesion, this sampling method may be category A.

(6)P The minimum internal diameter of the core cutter (equal to the core sample diameter) shall be based on the rock mass structure.

(7) Rock masses with discontinuities should not be cored out using an internal diameter of the cutter smaller than 76 mm.

(8)P The minimum length of the inner tube or liner if used shall be such that one core run contains a quantity of rock mass sufficient for identification and performance of required laboratory tests.

(9)P The maximum length of the inner tube or the length of the actual core run shall be such that the inside wall friction does not have too great influence on the results of the laboratory tests to be performed.

(10) Normal lengths of inner tubes are 1, 1,5, 3 and 6 m.

(11)P The inside clearance shall be sufficient to allow for the lateral movement of the sample during the continued core run, but it shall not be so much that it permits excessive deformations and causes additional disturbance of the sample.

(12)P The inside wall of the inner tube or liner shall be clear and smooth without any protruding edges or irregularities which may disturb the sample.

(13) Friction at the inside wall of the inner tube or liner may be reduced by using a cutting edge with an internal diameter slightly less than the diameter of the tube or liner.

(14) When sampling highly weathered rock masses or masses with dominating discontinuities inner tubes or liners split longitudinally may be used to minimize the risk of blocking of the core sample.

(15) A core catcher placed in the sampling equipment just behind the cutter should normally keep a cored sample of fresh to moderately weathered rock inside the sampling equipment and mobilize the necessary resistance for breaking off the core from the rock stratum beneath the cutter face.

(16) For core sampling of highly or completely decomposed rock masses a core retainer such as a spring basket on the core catcher may be used.

13.4 Rotary sampling

13.4.1 Rotary sampling operation

(1)P Before commencement of rotary sampling category A or B in a predrilled borehole having an adequate diameter, the hole shall be cleaned out down to the initial sampling depth.

(2)P Before the core barrel is lowered down into the drillhole it shall be checked that

- no friction occurs between the inner tube and the outer tube;
- the distance between the tip of the inner tube and the core bit is according to specifications;
- the drilling fluid will pass the sample at the right place - in case of face discharge only through the water tracks in the face of the cutter;
- the pump pressure for circulating the drilling fluid is according to specifications.

(3)P During sampling the load on the cutter, the amount of circulated drilling fluid, and the rotation speed shall be checked complying with the specifications for the actual core barrel in use.

(4)P The penetration rate should be recorded based on time and core run progress.

(5)P The total penetration during one core run shall not exceed the net length of the inner tube or liner.

(6)P The core sample all be cut off from the stratum and the sampler retrieved without rotating the core barrel.

(7)P The sampler all be retrieved at uniform speed.

(8)P If using a rod handled core barrel, shocks on the sampling equipment, from disconnecting the rods, shall be avoided.

(9)P During withdrawal of the core from the barrel appropriate techniques to ensure that the core sample is not damaged or disturbed shall be applied.

(10)P After the sampling tube is disconnected from the core barrel the length of the recovered sample shall be measured and core losses, if any, determined.

13.4.2 Preservation, handling and storage of core samplers

(1)P After sampling and visual inspection has taken place cores from category A and B sampling methods shall be sealed immediately in case the natural water content is to be maintained.

(2) When using a liner the liner should be cut in a length suitable for placement of end caps with contact to the top and the bottom of the sample.

(3) When using core cases or pipes for transportation and storage the samples should be wrapped in a water tight film.

(4)P Samples shall be properly marked leaving no doubt about the top and bottom of the sample.

(5)P Cores from category A and B sampling methods shall be handled in such a way that no further disturbance will occur after withdrawal of the sample or during transportation and storage.

(6) Cores from category A and B sampling methods should be adequately protected from excessive heat, frost, vibration and shock during shipment and storage. Great care should be taken not to damage the seals of the samples and not to deform the packing.

(7) Cores from category A and B sampling methods should be stored during the shipment in a container and in the laboratory in a room where the temperature and humidity are kept constant.

13.5 Reporting of the core sampling operation

13.5.1 Labelling, preservation and handling of core samples

(1)P All samples shall be numbered, recorded and labelled immediately after being sealed.

(2)P The sample label shall show all necessary information about

- identification data of the actual project;
- date of the sampling;
- number of the borehole;
- sampling class and type of sampler used;
- elevation or depths of the top and the bottom of the sample;
- the length of the core run.

(3) The rock type, decomposition and possible discontinuities according to the visual examination should be given on the label, if possible.

13.5.2 Boring report

(1)P In addition to the requirements given in 2.6 the sampling report for boring in rock masses shall include the following essential data if applicable:

- date of sampling;
- position and elevation of boring location;
- borehole direction: inclination and orientation;
- whenever possible the depth of the free groundwater level;
- the method of the preboring if used;
- the use of casing and depth of the casing tip;
- the use of drilling fluid and the level of the drilling fluid in borehole;
- colour and colour shifts of drilling fluid;
- loss, if any, of drilling fluid;
- drilling fluid pressure and circulated volume;
- the specifications and the type of sampler used;
- the diameter or the size of the sample;
- the depth (top and bottom of the sample) and the length of the sample;
- the core run interval;
- pressure on the cutting edge;
- the rock mass type, discontinuities and grade of decomposition based on the visual examination of the sample by the field foreman and his judgement of the sampling category;
- any obstructions and difficulties encountered during the sampling operation.

(2)P The sampling report shall be clear and accurate, and it shall not only contain the data required for determination of the rock strata and the location (x,y,z) of the samples obtained but also of any observations which will contribute to an estimate of the condition of the samples and the physical properties of the rock mass in situ.

(3) All unsuccessful sampling operation should be recorded.

14 GROUNDWATER MEASUREMENTS IN SOILS AND ROCK

14.1 General

(1)P Groundwater measurements consist of determining the groundwater table or the pore pressure in soils and rock with different types of equipment for observation or monitoring purposes. The measurements are made by installing open pipes or pipes provided with filters and possibly transducers and loggers to record the pressure at the level of the tip.

(2)P Groundwater measurements shall be used to determine the water pressure or pore pressure and their variations at pertinent levels for design purposes.

(3)P Groundwater measurements shall also be used for monitoring purposes to control the pressure during certain activities or for risk evaluations.

(4)P This section handles the measurement of positive pore pressures related to the atmospheric pressure for building and civil engineering purposes. Thus measurements of negative pore pressures are not considered. Groundwater sampling and determination of groundwater quality are also outside the scope of this section.

(5)P Groundwater measuring for geotechnical purposes shall be carried out in accordance with the requirements given in this section.

(6)P Any deviations from the requirements given below shall be thoroughly reported.

(7) Additional guidelines for groundwater measurements, presentation of measurement and derivation of groundwater pressure can be found in annex L.

14.2 Definitions

(1)P closed systems: system where the groundwater in the equipment is prevented from contact with the atmosphere and the pressure is recorded by a transducer.

(2) The advantage of closed systems is that only a small amount of water has to flow into or out of the piezometer at fluctuations in the pressure. There are hydraulic, pneumatic and electric closed piezometer systems.

(3)P filter tip: tip for groundwater pipes or piezometers provided with a filter to prevent particles from entering the equipment.

(4)P galvanic cell: coupling of different metals or alloys in the ground resulting in a corrosion cell and an electric current and possibly pore pressure changes.

(5)P groundwater fluctuations: variations in the groundwater table or pore pressure distribution with time.

(6)P groundwater table: water level where the pore pressure is equal to the atmospheric pressure at that point and time.

(7)P high air entry value filter: filters with small pores so that, when saturated, they give a high resistance to air entry.

(8)P groundwater measuring station: place where one or more equipment for groundwater measurements are located.

(9)P open systems: systems where the groundwater in the equipment has a direct contact with the atmosphere and the measurements are made as water levels in the borehole, pipe or plastic hose.

(10) The open system in practice consists of three different types: an observation borehole with or without a casing, an open perforated pipe with a possible filter of coarse sand or geotextile and finally a pipe provided with a filter tip and possibly an inner plastic hose.

(11)P piezometers: equipment for groundwater measurements including both closed and open systems.

(12)P pore pressure: pressure of water (or gas) in voids and fissures in the ground at a certain point and time.

(13)P time lag: time elapsed between a change in atmospheric pressure and corresponding change in the pore pressure.

14.3 Equipment

14.3.1 General

(1)P The type of equipment to be used for groundwater measurements shall be chosen in advance with respect to the actual type of ground, the purpose of the measurements and the response time of the equipment and the soil system.

(2) At the planning of groundwater measurements for determining the groundwater table or the pore pressure, the soil and rock conditions ought to be considered especially where there are great variations in the permeability. At the selection of the measuring system it is also important to consider the purpose of the measurements:

- to have a single observation of the groundwater table or the pore pressure profile;
- to determine the fluctuations in the groundwater level or pore pressure for a certain period;
- to have a monitoring system.

(3) There are two main methods for measuring the groundwater pressure, open systems and closed systems:

- open systems in the form of observation boreholes and open perforated pipes can only be used in homogeneous soils and rock with a high permeability, e.g. sand, gravel or very fissured rock, where there is no risk for soil particles to enter the borehole or pipe;
- open systems as open pipes with filter tips and an inner plastic hose can also be used in soils with low permeability e.g. glacial deposits, silts and clay if the response time is short enough for the purpose of the measurements;
- closed systems should be used in low permeable soils and when continuous recording is required or short term variations should be monitored e.g. for flow-net derivations. Also in case of high artesian water pressure closed systems are recommended.

(4) Guidance for the choice of measuring system depending on the purpose of the measurements and the type of ground at the measuring level can be found in table 14.1.

Table 14.1: Suitability of different groundwater measuring systems depending on their response time and the purpose of the measurements

Ground conditions	Gravel, coarse sand	Fine sand, coarse silt	Fine silt, glacial deposits, clay
Purpose of measurements			
measurements of groundwater levels or pore pressure profiles and their fluctuations	observation borehole open pipe filter tip	open pipe filter tip piezometer (hydraulic, pneumatic, electric)	filter tip piezometer (hydraulic, pneumatic, electric)
measurements of variations in pore pressures due to fluctuations, pumping, excavations, loading or unloading, effects of pile driving or for monitoring of e.g. slopes etc.	Filter tip piezometer (hydraulic, pneumatic, electric)	filter tip piezometer (hydraulic, pneumatic, electric)	piezometer (hydraulic, pneumatic, electric)

(5)P In cases where lakes, rivers or creeks are situated within or close to the actual investigation area the water level in those shall be measured as references to other measurements. Also the water level in wells, the occurrence of springs and artesian water shall be noted.

14.3.2 Basic requirements

(1)P The equipment shall measure the pore pressure in relation to the actual atmospheric pressure or shall be capable to measure both the total pressure and the atmospheric pressure.

(2)P Metallic parts of the equipment in electric connection with each other shall be manufactured from the same type of alloy in order to avoid galvanic cells in the ground.

(3) In case of risk of freezing the measuring system should be filled with an antifreeze medium with a density equal to that of water or replaced by a closed system if applicable.

(4)P In cases where the pore pressure is to be measured at a certain level or in a certain layer, special equipment with packers shall be used, or special installation measures shall be taken to ensure that connections with other layers are blocked.

(5)P The equipment selected shall be adequate to provide reliable data during the whole observation period.

(6)P Open pipes with filter tips provided with an inner plastic hose shall allow gas bubbles to pass through the hose.

(7)P Closed hydraulic piezometers shall allow flushing of deaired water through the system in order to release possible air bubbles.

(8) Piezometers of the closed type should be calibrated before each installation or reinstallation.

(9) Closed systems should preferably allow for so called zero-checks to be performed during the measuring period especially for long term measurements.

(10) Pneumatic piezometers should be calibrated with the actual length of circuit and the actual measuring unit.

(11)P The required precision of the measurements for a certain project, shall be decided in advance so that a proper equipment for the project can be chosen.

(12) Taking into account all possible sources of error and the compensation for the atmospheric pressure, the precision of the measurements should normally not be worse than 1 kPa in the range 1 kPa to 100 kPa and 2 kPa for values greater than 100 kPa.

14.4 Test procedure

14.4.1 Installation

(1)P The location and depth of each individual piezometer or groundwater pipe shall be chosen considering the purpose of the measurements, the topography, the stratigraphy and the soil conditions, especially the permeability of the ground or identified aquifers.

(2)P The number and location of measuring stations shall be chosen in such a way that an acceptable interpolation of the groundwater situation can be made for the actual purpose.

(3)P For monitoring projects e.g. groundwater lowering, excavations, fillings and tunnels, the location shall in addition be chosen with respect to expected changes to be monitored.

(4) The natural fluctuations in the groundwater pressure should also be measured in a reference measuring station outside but close to and in the same layers as the influenced zone also before and after the actual activities.

(5)P In order to obtain measurements reflecting the pore pressure in the intended point, soil or rock layer, provisions shall be made to ensure the required filter and sealing off towards other layers or aquifers and the function of the measuring equipment.

(6)P Installation of piezometers shall be made either by pushing or by preboring.

(7) During the installation the following should be checked, depending on the type of measuring equipment:

- the function of filters before and after installation;
- the saturation of filters and other water filled parts of the piezometer until inserted below water;
- that air bubbles in circuits are flushed away;
- that the generated excess pore pressure does not exceed the capacity of the equipment;
- that anti-freeze medium is added if applicable;
- that cables or pipes extended above ground surface are protected against filling or damage;
- that surface water cannot flow into the borehole;
- that elevation or depth of filter or stand pipe tip has been taken;
- that the screening to separate different aquifers is sufficient.

14.4.2 Measurements

(1)P The number and frequency of readings and the length of the measuring period shall be settled in advance in a preliminary plan for each commission considering the purpose of the measurements and the stabilizing period.

(2) All measurements should be related to the actual elevation system at the site.

(3) In order to find long term fluctuations regular measurements at reasonable frequent intervals over a long time span should be chosen.

(4) For short term measurements, e.g. for monitoring effects of pile driving, groundwater lowering or excavations, also automatic recording systems should be considered.

(5) In order to check that reliable results have been obtained at a certain occasion, the measurements taken should be recorded and checked on site by e.g. comparisons with previous and neighbouring measurements.

14.5 Interpretation of groundwater measurements

(1)P The results of a groundwater measurement shall be expressed in one of the following ways:

- water pressure, related to end of open pipe or the average filter level, in m. water collumn;
- water table, elevation, in m;
- pore pressure at a certain depth or elevation, in kPa.

(2)P The interpretation of the groundwater measurements shall be made with respect to actual measurements, the design of the piezometer and the atmospheric pressure if applicable.

(3) For observational boreholes and open pipes with or without a filter, the groundwater table is normally related to the upper end of the pipe or the ground surface which should be recorded.

(4) For hydraulic piezometers the interpretation should be based on the measured pressure and the difference in elevation between the measuring unit and the middle of the filter.

(5) For pneumatic or electrical equipment where the membrane is located above the filter the interpretation should be based on the measured pressure and the difference in elevation between the middle of the filter and the membrane.

(6)P For pore pressure transducers measuring the total pressure correction shall be made for the actual atmospheric pressure.

(7) The possible time lag between the changes in atmospheric pressure and corresponding changes in the pore pressure should be considered.

14.6 Reporting of groundwater measurements

(1)P Records from a groundwater measurement project shall in addition to general information required in 2.6 contain the following information whenever applicable. Records shall be filed according to commission and presented in the ground investigation report.

- date of installation;
- type of measurement made, related to a certain elevation or atmospheric pressure;
- type of equipment e.g. by reference to published standard;
- atmospheric pressure while setting the zero reading for closed system transducers;
- ground level at the piezometer location;
- elevation of tip of pipe or filter;
- elevation of upper end of pipe;
- date of each reading;
- interpreted pore pressure or water table elevation;
- signatures at installation and each reading;
- other measures taken to investigate the groundwater flow or quality.

(2) The presentation of the results should consider the purpose of the measurements, the number of piezometers and the length of the measuring period. Some examples are presented in annex L.2.

(3)P The ground investigation report shall contain an evaluation of the groundwater measurements with respect to the accuracy or the necessity of making further installations or measurements.

(4) Inaccuracy in the measurements may occur for many reasons such as:

- gas or air bubbles in the system;
- clogging of the filter;
- insufficient tightening around the pipes;
- temperature fluctuations and freezing;
- changes in calibration factors;
- corrosion due to e.g. a galvanic cell;
- human activities, e.g. pumping in neighbouring wells.

(5) A demand for further installations or measurements may arise where the soil profile is more complicated and unexpected groundwater pressure profiles are obtained.

14.7 Derived values of groundwater or pore pressures

(1)P Values of groundwater pressures shall be derived from groundwater measurements considering the natural fluctuations in ground, the season of the measurements and the actual meteorological conditions.

(2)P If applicable both upper and lower values for extreme and normal circumstances shall be derived. See ENV 1997-1, subclause 2.4.1, paragraph (10)P and (11).

(3) Upper and lower values for extreme circumstances can be derived from the measured values by adding or subtracting part of the expected fluctuations at that typical site considering the ground conditions. Further information can be found in annex M.3.

(4) Upper and lower values for normal circumstances can be derived from the measured values by adding or subtracting part of the expected fluctuations at the actual site. The reduction depends on the design situation, e.g. the time of consolidation for a soil layer.

ANNEX A
(Informative)

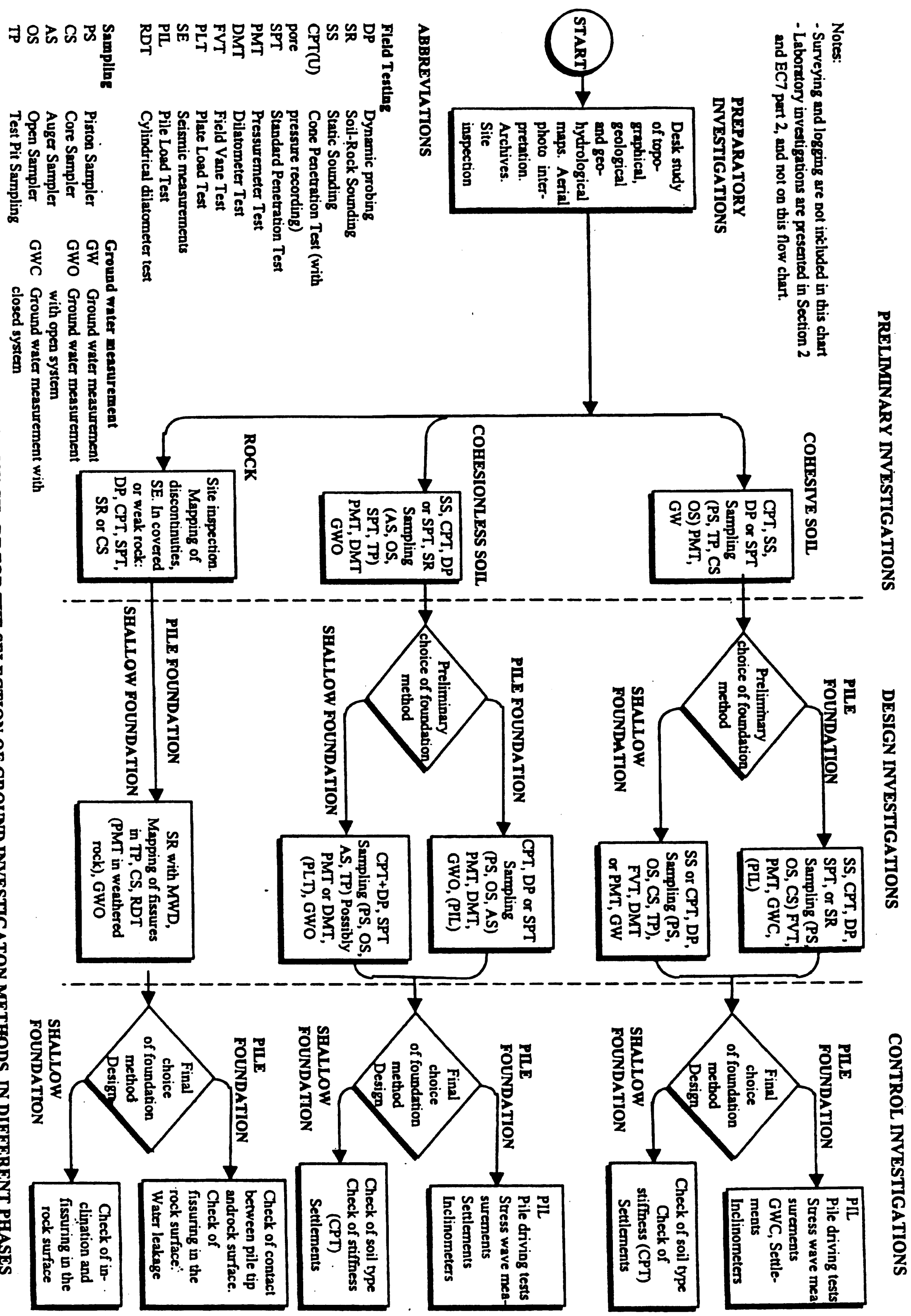


FIGURE A.1 EXAMPLE OF A FLOW CHART FOR THE SELECTION OF GROUND INVESTIGATON METHODS IN DIFFERENT PHASES

ANNEX B.1
(informative)

Cone Penetration Test (CPT)

(1) Table B.1 is an example of derived values, from the value of q_c, of the angle of shearing resistance N' and drained (long term) Young's modulus E_m for quartz and feldspar sands, for calculations of the bearing capacity and settlement of spread foundations.

(2) This example correlates the mean value of q_c in a layer to the mean values of N' and E_m (see subclause 1.3.2).

Table B.1: Derived values for the angle of shearing resistance N' and drained Young's modulus E_m for quartz and feldspar sands from cone resistance q_c (after Bergdahl et al. 1993)

Relative density	Cone resistance (q_c) [Mpa] (from CPT-test)	Angle of shearing resistance$^{1)}$ (N')	Drained Young's modulus$^{2)}$ (E_m) [Mpa]
very low	0,0 - 2,5	29 - 32	< 10
low	2,5 - 5,0	32 - 35	10 - 20
medium	5,0 - 10,0	35 - 37	20 - 30
high	10,0 - 20,0	37 - 40	30 - 60
very high	> 20,0	40 - 42	60 - 90

1) Values given are valid for sands. For silty soils a reduction of 3° should be made. For gravels 2° should be added.

2) Values given for the drained modulus correspond to settlements for 10 years. They are obtained assuming that the vertical stress distribution follows the 2:1 approximation. Furthermore, some investigations indicate that these values can be 50 % lower in silty soils and 50 % higher in gravelly soils. In overconsolidated cohesionless soils the modulus can be considerably higher. When calculating settlements for ground pressures greater than 2/3 of the design bearing pressure in ultimate limit state, the modulus should be set to half of the values given in this table.

For additional information and documents giving examples, see Annex M.

ANNEX B.2
(informative)

Cone Penetration Test (CPT)

(1) The following is an example of a semi-empirical method for calculating settlements of spread foundations in cohesionless soils (after Schmertmann, 1970).

(2) The settlement s of a foundation under load pressure q is expressed as:

$$s = C_1 \times C_2 \times \left(q - s'_{v0}\right) \times \int_0^{z1} \frac{I_z}{E_m} dz$$

where:

C_1 is 1 - 0,5 [σ'_{v0} / (q - σ'_{v0})];
C_2 is 1,2 + 0,2 . log t;
σ'_{v0} is the initial effective vertical stress at the level of the foundation;
t is the time, in years.

(3) Figure B.1 gives for axisymmetric (circular and square) spread foundations and for plane strain (strip spread foundations) the distribution of the vertical strain influence factor I_z. The derived value for Young's modulus E_m to be used in this method is:

- $E_m = 2.5 \times q_c$, for axisymmetry, and
- $E_m = 3.5 \times q_c$, for plane strain.

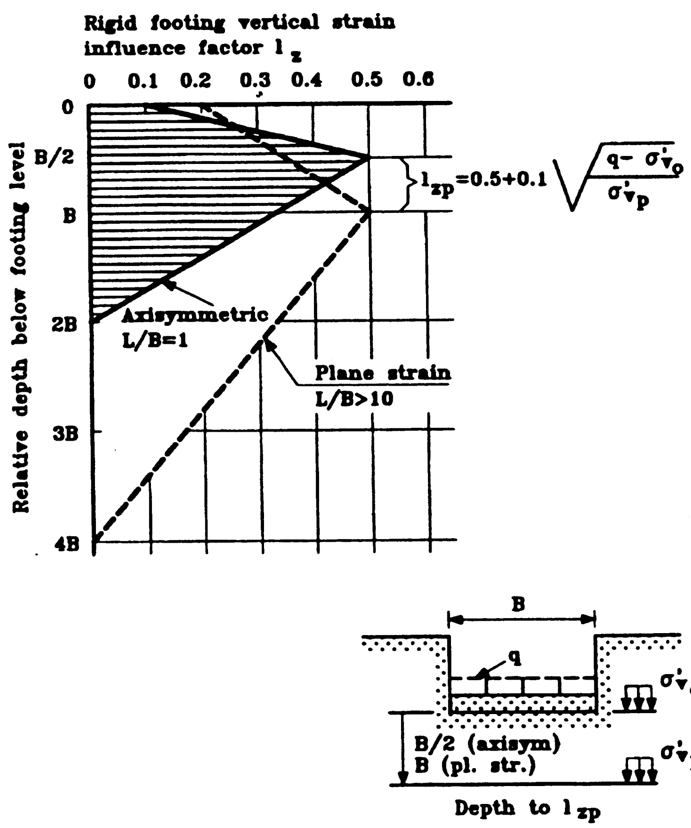


Figure B.1: Values for strain influence factor diagrams

For additional information and examples see Annex M.

ANNEX B.3
(informative)

Cone Penetration Test (CPT)

(1) The following is an example of derived values of " for various types of soils (after Sanglerat, 1972)

CL - low-plasticity clay:	$q_c \leq 0{,}7$ [Mpa]	$3 < \alpha < 8$
	$0{,}7 < q_c < 2$ [Mpa]	$2 < \alpha < 5$
	$q_c \geq 2$ [Mpa]	$1 < \alpha < 2{,}5$
ML - low-plasticity silt:	$q_c < 2$ [Mpa]	$3 < \alpha < 6$
	$q_c \geq 2$ [Mpa]	$1 < \alpha < 2$
CH - very plastic clay MH - very plastic silt:	$q_c < 2$ [Mpa]	$2 < \alpha < 6$
	$q_c > 2$ [Mpa]	$1 < \alpha < 2$
OL - very organic silt:	$q_c < 1{,}2$ [Mpa]	$2 < \alpha < 8$
T-OH-peat and very organic clay:	$q_c < 0{,}7$ [Mpa]	
	$50 < w \leq 100$	$1{,}5 < \alpha < 4$
	$100 < w \leq 200$	$1 < \alpha < 1{,}5$
	$w > 300$	$\alpha < 0{,}4$
Chalks:	$2 < q_c \leq 3$ [Mpa]	$2 < \alpha < 4$
	$q_c > 3$ [Mpa]	$1{,}5 < \alpha < 3$
Sands:	$q_c < 5$ [Mpa]	$\alpha = 2$
	$q_c > 10$ [Mpa]	$\alpha = 1{,}5$

For additional information and examples see Annex M.

ANNEX B.4
(informative)

Cone Penetration Test (CPT)

(1) The following is an example of the determination of the maximum bearing resistance of a single pile on the basis of the q_c-values from a CPT.

(2) The maximum bearing resistance of a pile follows from:

$$F_{max} = F_{max;base} + F_{max;shaft}$$

where:

$$F_{max;base} = A_{base} \times p_{max;base}$$

and

$$F_{max;shaft} = O_p \int_0^{\Delta L} p_{max;shaft} \, dz$$

where:

F_{max}	is the maximum bearing resistance of the pile, in MN;
$F_{max;base}$	is the maximum base resistance, in MN;
$F_{max;shaft}$	is the maximum shaft resistance, in MN;
A_{base}	is the cross sectional area of the base, in m^2;
D_{eq}	is the equivalent diameter of the base, in m;
	$D_{eq} = 1,13\ a\ /(b/a)$
	where:
	a is the length of the smallest side of the base area, in m;
	b is the largest side, in m;
$p_{max;base}$	is the maximum unit base resistance, in MN/m^2;
	O_p is the circumference of the part of the pile shaft in the layer in which the base of the pile is placed, in m;
	ΔL is the distance from the base of the pile to the bottom of the first soil layer above the base with $q_c < 2$ MN/m^2; moreover $\Delta L \leq$ the length of the enlarged part of the pile point if applied, in m;
$p_{max;shaft}$	is the maximum unit shaft resistance, in MN/m^2;
z	is the depth or vertical direction (positive downwards).

(3) The maximum base resistance $p_{max;base}$ can be derived from the following equation:

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

and

$$p_{max;base} \leq 15 \text{ MN/m}^2$$

where:

$q_{c;I;mean}$ is the the mean of the $q_{c;I}$-values over the depth running from the pile base level to a level which is at least 0,7 times and at most 4 times the equivalent pile base diameter D_{eq} deeper (see figure B.2);

$$q_{c;\,I;\,mean} = \frac{1}{d_{crit}} \int_0^{d_{crit}} q_{c;\,I}\, dz$$

$0{,}8\ D_{eq} < d_{crit} < 4\ D_{eq}$

At the critical depth the calculated value of $p_{max;base}$ becomes a minimum;

$q_{c;II;mean}$ is the mean of the lowest $q_{c;II}$-values over the depth going upwards from the critical depth to the pile base (see figure B.2);

$$q_{c;II;mean} = \frac{1}{d_{crit}} \int_{d_{crit}}^{0} q_{c;II}\ dz$$

$q_{c;III;mean}$ is the mean value of of the $q_{c;III}$ -values over a depth interval running from pile base level to a level of 8 times the pile base diameter higher. This procedure starts with the lowest $q_{c;II}$-value used for the computation of $q_{c;II;mean}$ (see figure B.2);

$$q_{c;\,III;\,mean} = \frac{1}{8\,D_{eq}} \int_0^{-8\,D} q_{c;\,III}\, dz$$

For continuous flight auger piles $q_{c;III;mean}$ may not exceed 2 MN/m^2, unless the results of CPT's which were performed at a distance from the pile > 1 m after pile fabrication are used for the calculation of the bearing resistance;

α_p is the pile class factor given in table B.2;
β is the factor which takes account of the shape of the pile point as shown in figure B.3;
s is the factor which accounts for the shape of the pile base as shown in figure B.4;
OCR is the Over Consolidation Ratio.

(4) The maximum shaft resistance $p_{max;shaft}$ should be determined as follows:

$$p_{max;shaft} = \alpha_s\, q_{c;z;a}$$

where:

α_s is the factor according to table B.2 and B.3;
$q_{c;z;a}$ is q_c at depth z, in MN/m^2.
If $q_{c;z;} \geq 15$ MN/m^2 over a continuous depth interval of 1 m or more, $q_{c;z;a} \leq 15$ MN/m^2 over this interval.

When the depth interval with $q_{c;z;a} > 12$ MN/m^2 is less than 1 m thick, $q_c \leq 12$ MN/m^2 over this interval.

Table B.2: Maximum values of α_p and α_s for sands and gravely sands

Pile class or type	α_p	α_s 1)
Soil displacement type piles, diameter > 150 mm – driven prefabricated piles, – cast in place piles made by driving a steel tube with closed end. The steel pipe is reclaimed during concreting.	1,0 1,0	0,010 0,014
Soil replacement type piles, diameter > 150 mm – flight auger piles, – bored piles (with drilling mud).	0,8 0,6	0,006 2) 0,005
1) Values valid for fine to coarse sands. For very coarse sands a reduction factor of 0,75 is necessary; for gravel this reduction factor is 0,5. 2) This value is used in the case of applying the results of CPT s which were carried out before pile installation. When CPT s are used that have been carried out in the vicinity of the flight auger piles, α_s may be raised to 0,01.		

Table B.3: Maximum α_s-values for clay, silt and peat

Soil type	relative depth z/d_{eq}	α_s
clay/silt ($q_c \leq 1$ MN/m^2)	$5 < z/d_{eq} < 20$	0,025
clay/silt ($q_c \geq 1$ MN/m^2)	$z/d_{eq} \geq 20$	0,055
clay/silt ($q_c > 1$ MN/m^2)	not applicable	0,035
peat	not applicable	0
d_{eq} equivalent pile shaft diameter		

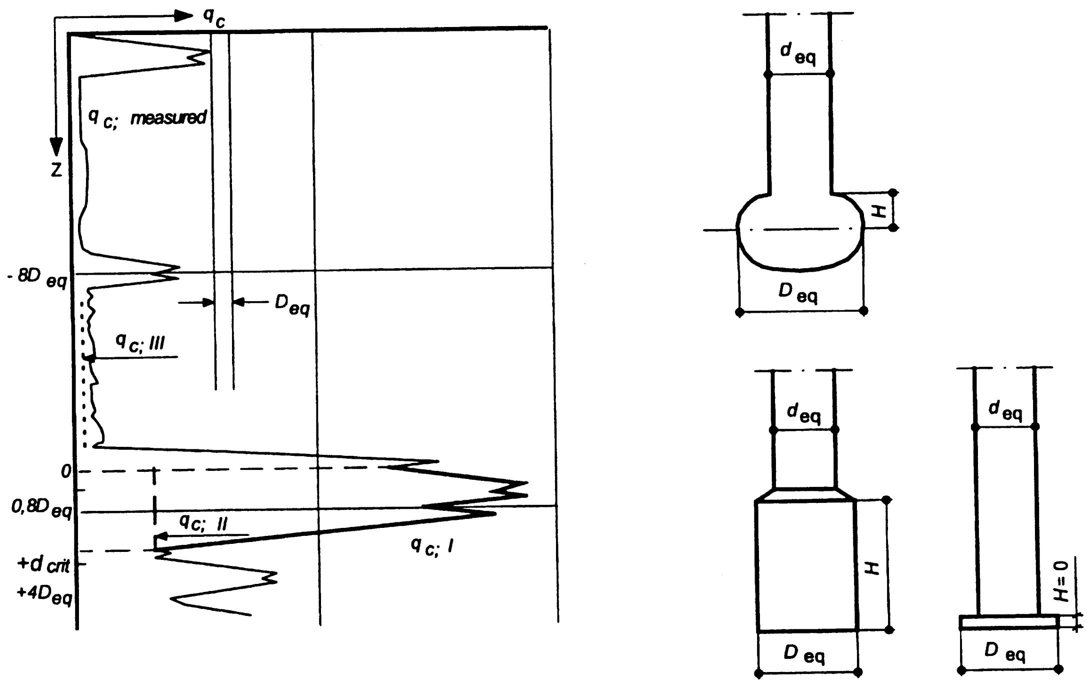


Figure B.2: Explanation of $q_{c;I}$, $q_{c;II}$ and $q_{c;III}$

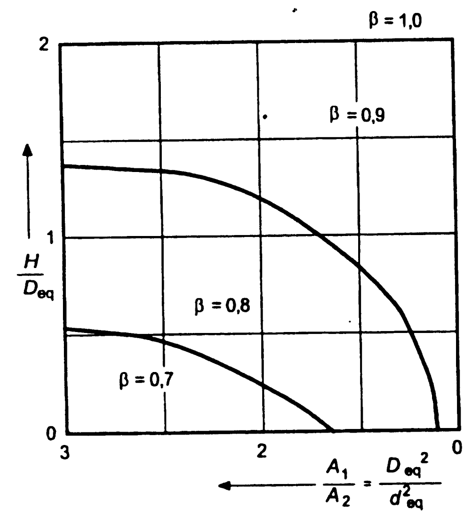


Figure B.3: Pile point shape factor

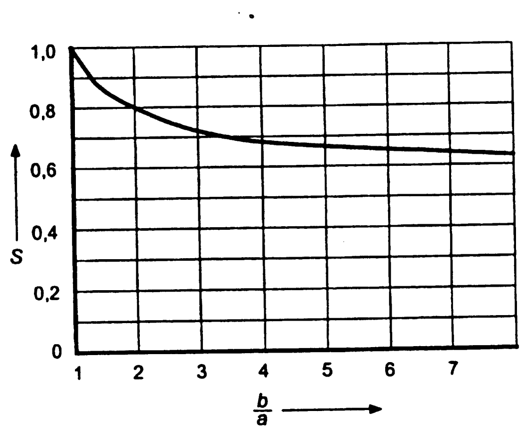


Figure B.4: Values of *s*

ANNEX C.1
(informative)

Pressuremeter Test (PMT)

(1) The following is an example of the calculation of the bearing resistance of spread foundations using a semi-empirical method and the results of an MPM test.

(2) The bearing resistance is calculated from

$$\frac{R}{A'} = \sigma_{vo} + k\ (p_{LM} - p_o)$$

where:

R is the resistance of the foundation against normal loads;
A' is the effective base area as defined in ENV 1997-1;
σ_{vo} is the total (initial) vertical stress at the level of the foundation base;
p_{LM} representative value of the Ménard limit pressures at the base of the spread foundation;
p_o is $[K_o\ (\sigma_v - u) + u]$ with K_o conventionally equal to 0,5, and F_v is the total vertical stress at the test level and u is the pore pressure at the test level;
k is a bearing resistance factor given in table C.1;
B is the width of the foundation;
L is the length of the foundation;
D_e is the equivalent depth of foundation.

Table C.1: Derived values for the bearing resistance factor, *k*, for spread foundations

Soil Category		p_{LM} [Mpa]	k
clay and silt	A	< 0,7	$0{,}8\times[1 + 0{,}25\ (0{,}6 + 0{,}4\ B/L)\times D_e/B]$
	B	1,2 - 2,0	$0{,}8\times[1 + 0{,}35\ (0{,}6 + 0{,}4\ B/L)\times D_e/B]$
	C	> 2,5	$0{,}8\times[1 + 0{,}50\ (0{,}6 + 0{,}4\ B/L)\times D_e/B]$
sand and gravel	A	< 0,5	$[1 + 0{,}35\ (0{,}6 + 0{,}4\ B/L)\times D_e/B]$
	B	1,0 - 2,0	$[1 + 0{,}50\ (0{,}6 + 0{,}4\ B/L)\times D_e/B]$
	C	> 2,5	$[1 + 0{,}80\ (0{,}6 + 0{,}4\ B/L)\times D_e/B]$
chalk			$1{,}3\times[1 + 0{,}27\ (0{,}6 + 0{,}4\ B/L)\times D_e/B]$
marl and weathered rock			$[1 + 0{,}27\ (0{,}6 + 0{,}4\ B/L)\times D_e/B]$

For additional information and examples, see Annex M

ANNEX C.2
(informative)

Pressuremeter Test (PMT)

(1) The following is an example of the calculation of the settlement, s, of spread foundations using a semi-empirical method developed for MPM tests.

$$s = (q - \sigma_{vo}) \left(\frac{2\,B_o}{9\,E_d} \times \left(\frac{\lambda_d \;\; B}{B_o} \right)^{\alpha} + \frac{\alpha \;\; \lambda_c \;\; B}{9\,E_c} \right)$$

where:

B_o	is a reference width of 0,6 m;
B	is the width of the foundation;
λ_d and λ_c	are shape factors given in table C.2;
α	is a rheological factor given in table C.3;
E_c	is the weighted value of E_M immediately below the foundation;
E_d	is the harmonic mean of E_M in all layers up to $8 \times B$ below the foundation;
σ_{vo}	is the total (initial) vertical stress at the level of the foundation base;
q	is the design normal pressure applied on the foundation.

Table C.2: The shape coefficients, λ_c, λ_d, for settlement of spread foundations

L/B	circle	square	2	3	5	20
λ_d	1	1,12	1,53	1,78	2,14	2.65
λ_c	1	1,1	1,2	1,3	1.4	1,5

Table C.3: Derived values for the coefficient α for spread foundations

Type of ground	Description	E_M/p_{LM}	α
peat			1
clay	overconsolidated normally consolidated remoulded	< 16 9 - 16 7 - 9	1 0,67 0,5
silt	overconsolidated normally consolidated	> 14 5 - 14	0,67 0,5
sand		> 12 5 - 12	0,5 0,33
sand and gravel		> 10 6 - 10	0,33 0,25
rock	extensively fractured unaltered weathered		0,33 0.5 0,67

For additional information and examples, see Annex M.

ANNEX C.3
(informative)

Pressuremeter Test (PMT)

(1) The following is an example of the calculation of the ultimate bearing resistance, Q, of piles from the MPM test, using

$$Q = A \ k \ [p_{LM} - p_o] + P \ \Sigma[\ q_{si} \ . \ z_i]$$

where:

A is the base area of the pile which is equal to the actual area for close ended piles and part of that area for open ended piles;

p_{LM} is the representative value of the limit pressure at the base of the pile corrected for any weak layers below;

p_o is $[K_o (\sigma_v - u) + u]$ with K_o conventionally equal to 0,5, and σ_v is the total vertical overburden pressure at the test level and u is the pore pressure at the test level;

k is a bearing resistance factor given in table C.4;

P is the pile perimeter;

q_{si} is the unit shaft resistance for soil layer i given by figure C.1 read in conjunction with table C.5;

z_i is the thickness of soil layer i.

Table C.4: Derived values of the bearing resistance factor, *k*, for axially loaded piles

Soil category		p_{LM} [Mpa]	Bored piles and small displacement piles	Full displacement piles
clay and silt	A	< 0,7	1,1	1,4
	B	1,2 - 2,0	1,2	1,5
	C	> 2,5	1,3	1,6
sand and gravel	A	< 0,5	1,0	4,2
	B	1,0 - 2,0	1,1	3,7
	C	> 2,5	1,2	3,2
chalk	A	< 0,7	1,1	1,6
	B	1,0 - 2,5	1,4	2,2
	C	> 3,0	1,8	2,6
marl	A	1,5 - 4,0	1,8	2,6
	B	> 4,5	1,8	2,6
weathered rock	A	2,5 - 4,0	(i)	(i)
	B	> 4,5		
(i) Choose *k* for the closest soil category.				

Table C.5: The selection of design curves for unit shaft resistance

soil category		clay and silt			sand and gravel			chalk			marl		rock
pile type		A	B	C	A	B	C	A	B	C	A	B	
bored piles and caissons	no support	1	1/2	2/3	-		-	1	3	4/5	3	4/5	6
	mud support	1	1/2	1/2	1	1/2	2/3	1	3	4/5	3	4/5	6
	temp casing	1	1/2	1/2	1	1/2	2/3	1	2	3/4	3	4	-
	perm casing	1	1	1	1	1	2				2	3	-
hand dug caisson		1	2	3	-	-	-	1	2	3	4	5	6
displace-ment piles	closed end	1	2	2	2	2	3				3	4	4
	prefab concrete	1	2	2	3	3	3				3	4	4
	cast in situ	1	2	2	2	2	3	1	2	3	3	4	-
	coated shaft	1	2	2	3	3	4				3	4	-
grouted piles	low pressure	1	2	2	3	3	3	2	3	4	5	5	-
	high pressure	1	4	5	5	5	6	-	5	6	6	6	7

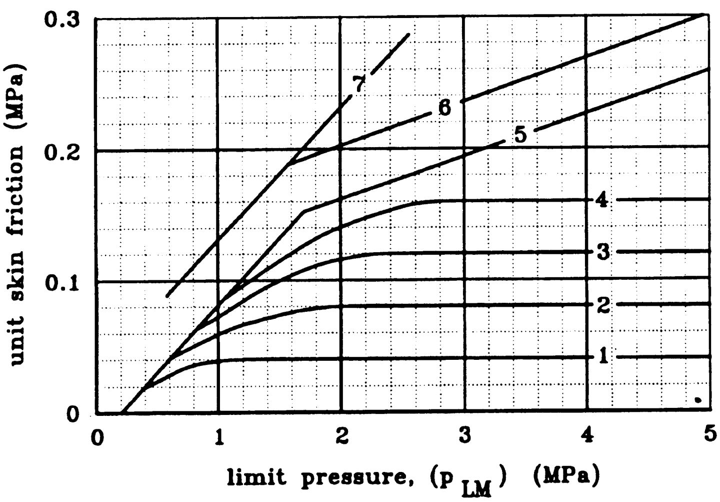


Figure C.1: Unit shaft resistance for axially loaded piles

For additional information and examples see Annex M.

ANNEX D.1
(informative)

Standard Penetration Test (SPT)

Table D.1: Values of the energy ratios ER_r of the common equipment used in various countries and the correction factors to apply for normalizing to ER_r = 60 %

Country	Hammer	Release	ER_r (%)	$ER_r/60$
North and South America	Donut Safety Automatic	2 turns of rope 2 turns of rope Trip	45 55 55 to 83	0,75 0,92 0,92 to 1,38
Japan	Donut Donut	2 turns of rope Auto-Trigger	65 78	1,08 1,3
China	Donut Automatic	2 turns of rope Trip	50 60	0,83 1,0
United Kingdom	Safety Automatic	2 turns of rope Trip	50 60	0,83 1,0
Italy	Donut	Trip	65	1,08

For additonal information and examples see Annex M

ANNEX D.2
(informative)

Standard Penetration Test (SPT)

(1) Below examples of correlations of blow counts and density indices (Skempton, 1986) are given.

(2) The relationship between the blow count N_{60}, density index $I_D = (e_{max} - e) / (e_{max} - e_{min})$ and the effective overburden pressure σ_v' (kPa $\times$ 10^{-2}) in a given sand can be represented by the expression:

$$\frac{N_{60}}{I_D^2} = a + b\sigma_{v'}$$

The parameters a and b in normally consolidated sands are nearly constant for $0{,}35 < I_D < 0{,}85$ and $0{,}5 < \sigma_v' < 2{,}5$, in kPa x 10^{-2}.

(3) For normally consolidated natural sand deposits the correlation shown in table D.2 has been established between I_D and the normalized blow count $(N_1)_{60}$:

Table D.2: Correlation between the density index I_D and the normalized blow count $(N_1)_{60}$

I_D =	0 %	15 %	35 %	65 %	85 %	100 %
	Very loose	Loose	Medium	Dense	Very dense	
$(N_1)_{60}$ =	0	3	8	25	42	58

For $I_D > 0{,}35$ it corresponds to $(N_1)_{60}/I_D^2 \cong 60$

(4) For fine sands the N-values should be reduced in the ratio 55/60 and for coarse sands increased in the ratio 65/60.

(5) The resistance of sand to deformation is greater the longer the period of consolidation. This "ageing" effect is reflected in higher blow counts, and appears to cause an increase in the parameter *a*.

Typical results for normally consolidated fine sands are given in table D.3.

Table D.3: Effect of ageing in normally consolidated fine sands

	Age [years]	$(N_1)_{60}/I_D^2$
Laboratory tests	10^{-2}	35
Recent fills	10	40
Natural deposits	$> 10^2$	55

(6) Overconsolidation increases the coefficient b by the factor

$$\frac{1 + 2 \times K_0}{1 + 2 \times K_{0NC}}$$

where:

K_0 and K_{0NC} are the in situ stress ratios between horizontal and vertical effective stresses for the overconsolidated and normally consolidated sand respectively.

(7) All the above mentioned correlations have been established for predominantly silica sands. Their use in more crushable and compressible sands like calcareous sands or even silica sands containing a non-negligible amount of fines, may lead to an underestimation of I_D.

For additional information and examples see Annex M.

ANNEX D.3
(informative)

Standard Penetration Test (SPT)

(1) This is an example of derivation of the angle of shearing resistance of silica sands, *N*', from the density index I_D. The values of *N*' are also influenced by the angularity of the particles and the stress level.

Table D.4: Angle of shearing resistance of silica sands, *N*'

Desity index I_D	Fine grained		Medium grained		Coarse grained	
[%]	Uniform	Well graded	Uniform	Well graded	Uniform	Well graded
40	34	36	36	38	38	41
60	36	38	38	41	41	43
80	39	41	41	43	43	44
100	42	43	43	44	44	46

For additinal information and examples see Annex M

ANNEX D.4
(informative)

Standard Penetration Test (SPT)

(1) This is an example of an empirical direct method for the calculation of settlements in granular soils of spread foundations proposed by Burland and Burbidge (1985).

(2) The settlement for stresses below the overconsolidation pressure is assumed to be 1/3 of that corresponding to the normally consolidated sand. The immediate settlement, s_i, in mm, of a square footing of width B, in m, is then given by:

$$s_i = \sigma'_{v0} \times B^{0,7} \times \frac{I_c}{3} + (q' - \sigma'_{v0}) \times B^{0,7} \times I_c$$

where:

σ'_{v0} is maximum previous overburden pressure, in kPa;
q' is average effective foundation pressure, in kPa;
I_c is $a_f/B^{0,7}$;
a_f is the foundation subgrade compressibility, $\Delta s_i/\Delta q'$, in mm/kPa.

(3) Through a regression analysis of settlement records the value of I_c is obtained through the expression:

$$I_c = 1{,}71 / \overline{N}^{1,4}$$

where $\overline{N}$ is the average SPT blow count over the depth of influence. The standard error of a_f varies from about 1,5 for $\overline{N}$ greater than 25 to 1,8 for $\overline{N}$ less than about 10.

(4) The N-values for this particular empirical method should not be corrected for the overburden pressure. No mention is made of the energy ratio (ER_r) corresponding to the N-values. The effect of the water table is supposed to be already reflected in the measured blow count, but the correction $N' = 15 + ½ \times (N\text{-}15)$ for submerged fine or silty sands should be applied for $N > 15$.
In cases involving gravels or sandy gravels, the SPT blow count should be increased by a factor of about 1,25.

(5) The value of $\overline{N}$ is given by the arithmetic mean of the measured N-values over the depth of influence, $z_I = B^{0,75}$, within which 75 % of the settlement takes place, for cases where N increases or is constant with depth. Where N shows a consistent decrease with depth, the depth of influence is taken as $2B$ or the bottom of the soft layer whichever is the lesser.

(6) A correction factor f_s for the length-to-width ratio (L/B) of the foundation

$$f_s = \left[\frac{1{,}25 \times L/B}{(L/B) + 0{,}25}\right]^2$$

should be applied.The value of f_s tends to 1,56 as L/B tends to infinity. No depth (D) correction factor has to be applied for $D/B < 3$.

(7) Foundations in sands and gravels exhibit time-dependent settlements. A correction factor, f_t, should be applied to the immediate settlement given by:

$$f_t = (1 + R_3 + R_t \log t/3)$$

where f_t is the correction factor for time $t \geq 3$ years, R_3 is the time-dependant factor for the settlement that takes place during the first 3 years after construction and R_t is the time-dependent factor for the settlement that takes place each log cycle of time after 3 years.

(8) For static loads conservative values of R_3 and R_t are 0,3 and 0,2 respectively. Thus at $t = 30$ years, $f_t = 1{,}5$. For fluctuating loads (tall chimneys, bridges, silos, turbines etc.) values of R_3 and R_t are 0,7 and 0,8 respectively so that at $t = 30$ years, $f_t = 2{,}5$.

For additioal information and examples, see Annex M.

ANNEX E.1
(informative)

Dynamic Probing (DP)

(1) This is an example of derived values for the density index I_D from the dynamic probing DP test, for different values of the uniformity coefficient C_u (range of validity $3 \leq N_{10} \leq 50$):

a) poorly graded sand ($C_{\text{ã}} \leq 3$) above groundwater

I_D = 0,15 + 0,260 log N_{10} (DPL)

I_D = 0,10 + 0,435 log N_{10} (DPH)

b) poorly graded sand ($C_{\text{ã}} \leq 3$) below groundwater

I_D = 0,21 + 0,230 log N_{10} (DPL)

I_D = 0,23 + 0,380 log N_{10} (DPH)

c) well graded sand-gravel ($C_u \geq 6$) above groundwater

I_D = - 0,14 + 0,550 log N_{10} (DPH).

For additional information and examples see, Annex M

ANNEX E.2
(informative)

Dynamic Probing

(1) This is an example of deriving the effective angle of shearing resistance ϕ' from the density index I_D, for bearing capacity calculations of cohesionless soils.

Table E.1: Conservative estimates of derived values for the effective angle of shearing resistance N' of cohesionless soils from the density index I_D for different values of the uniformity coefficient U

Soil type	**Grading**	**Range of I_D, [%]**	**Angle of shearing resistance ϕ'**
slightly fine-grained sand, sand, sand-gravel	poorly graded, ($U < 6$)	15 - 35 (loose) 35 - 65 (medium dense) > 65 (dense)	30 32,5 35
sand, sand-gravel, gravel	well graded, ($6 \leq U \leq 15$)	15 - 35 (loose) 35 - 65 (medium dense) > 65 (dense)	30 34 38

For additional information and examples, see Annex M

ANNEX E.3
(informative)

Dynamic Probing (DP)

(1) This is an example of the derivation of the vertical stress dependent oedometer settlement modulus E_{oed}, frequently recommended for settlement calculation of spread foundations, defined as follows:

$$E_{oed} = v \times P_a \left(\frac{\sigma'_v + 0{,}5\ \sigma'_p}{P_a} \right)^w$$

where:

v is the stiffness coefficient;
w is the stiffness exponent; for sands with a uniformity coefficient $U \leq 3$: $w = 0{,}5$; for clays of low plasticity ($I_p \leq 10$; $w_L \leq 35$): w=0,6;
σ'_v is the effective vertical stress at the base of the foundation or at any depth below it due to overburden of the soil;
σ'_p is the effective vertical stress caused by the structure at the base of the foundation or at any depth below it;
p_a is the atmospheric pressure
I_p is the plasticity index
w_L is the liquid limit.

(2) Values for the stiffness coefficient v can be derived from DP tests using for example the following equations, depending on the soil type:

a) closely graded sands ($U \leq 3$) above groundwater

$v = 214 \log N_{10} + 71$ (DPL; range of validity: $4 \leq N_{10} \leq 50$)

$v = 249 \log N_{10} + 161$ (DPH; range of validity: $3 \leq N_{10}$ (10))

b) low plasticity clays of at least stiff consistency ($0{,}75 \leq I_c \leq 1{,}30$) and above groundwater (I_c is the consistency index)

$v = 4\ N_{10} + 30$ (DPL; range of validity: $6 \leq N_{10} \leq 19$)

$v = 6\ N_{10} + 50$ (DPH; range of validity: $3 \leq N_{10} \leq 13$).

For additional information and examples see, Annex M

ANNEX F
(informative)

Weight Sounding Test (WST)

Table F.1: An example of derived values of angle of shearing resistance ϕ' and drained Young's modulus of elasticity E_m for naturally deposited quartz and feldspar sands estimated from Weight Sounding resistance in Sweden

Relative density	Weight sounding resistance[1], halfturns / 0,2 m	Angle of shearing resistance[2] [N']	Drained Young's modulus[3], E_m [Mpa]
very low	0 - 10	29 - 32	< 10
low	10 - 30	32 - 35	10 - 20
medium	20 - 50	35 - 37	20 - 30
high	40 - 90	37 - 40	30 - 60
very high	> 80	40 - 42	60 - 90

[1] Before determination of the relative density the weight sounding resistance in silty soils should be divided by a factor of 1,3.

[2] Values given are valid for sands. For silty soils a reduction of 3° should be made. For gravels 2° may be added.

[3] Values given for the drained modulus correspond to settlements after 10 years. They are obtained assuming that the vertical stress distribution follows the 2:1 approximation. Furthermore some investigations indicate that these values can be 50 % lower in silty soils and 50 % higher in gravelly soils. In overconsolidated cohesionless soils the modulus can be considerably higher. When calculating settlements for ground pressure greater than 2/3 of the design pressure in ultimate limit state the modulus should be set to half the values given in this table.

(1) Table F.1 gives an example of the derived values of the angle of shearing resistance ϕ and drained Young's modulus of elasticity E_m, estimated from weight sounding resistance. This example correlates the mean value of weight sounding resistance in a layer to the mean values of N' and E_m.

(2) If results of weight sounding tests only are available the lower value in each interval for the angle of shearing resistance and Young's modulus in table F.1 should be selected.

(3) When evaluating weight sounding resistance diagrams for application in table F.1, peak values caused e.g. by stones or pebbles should not be accounted for. Such peak values are common in weight sounding tests in gravel.

For additional information and examples, see Annex M.

ANNEX G
(informative)

Field Vane Test (FVT)

(1) Examples of correction factors to obtain the undrained shear strength from the measured value are shown in figures G.1 and G.2, based on local experience and back calculations of slope failures.

(2) Correction factors based on figure G.1 may be used in soft normally consolidated clays.

(3) Correction factors based on figure G.2 may be used in overconsolidated clays.

(4) If more than one method correcting the measured value is used, the value of the correction factor which will give the lowest value of the undrained shear strength should be applied.

(5) A greater correction factor than 1,2 should not be used without support from supplementary investigations.

(6) In fissured clays a correction factor as low as 0,3 can be necessary.

For additional information and examples, see Annex M

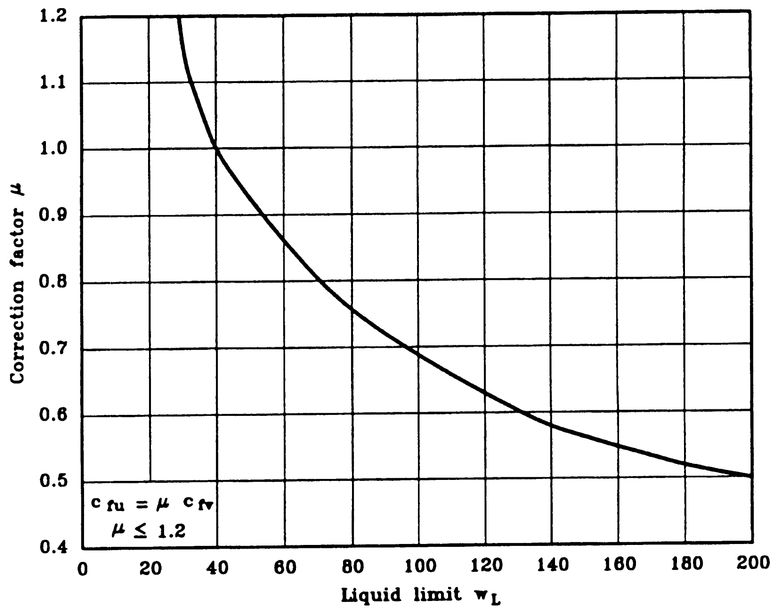


Figure G.1: An example of correction factors for c_{fv} based on liquid limit for normally consolidated clays

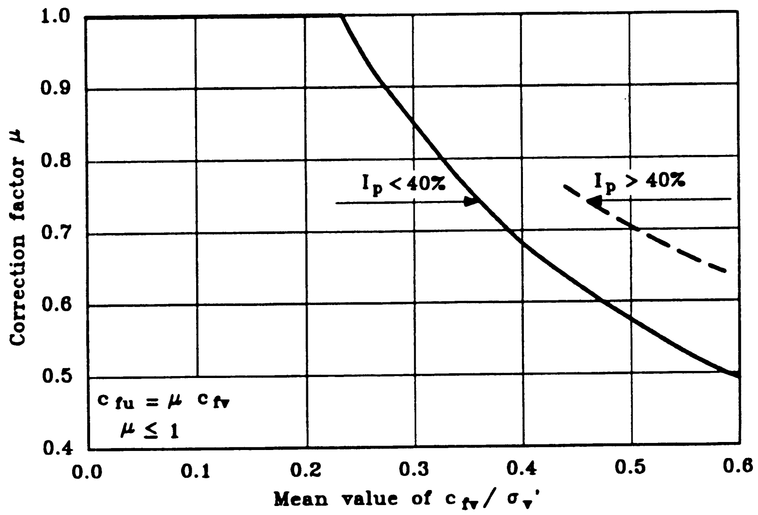


Figure G.2: An example of correction factors for c_{fv} based on plasticity index and effective vertical stress σ_v' for overconsolidated clays

ANNEX H
(informative)

Flat Dilatometer Test (DMT)

(1) The following is an example of correlations that may be used to determine the value of the one-dimensional tangent modulus $E_{oed} = d\sigma'/d\varepsilon$ from results of DMT tests:

$$E_{oed} = R_M \times E_{DMT}$$

in which R_M is estimated either on the basis of local experience or using the following relationships:

- if $I_{DMT} \leq 0,6$; then $R_M = 0,14 + 2,36 \log K_{DMT}$
- if $I_{DMT} \geq 3,0$; then $R_M = 0,5 + 2 \log K_{DMT}$
- if $0,6 < I_{DMT} < 3,0$; then $R_M = R_{M0} + (2,5 - R_{M0}) \log K_{DMT}$, in which $R_{M0} = 0,14 + 0,15 (I_{DMT} - 0,6)$
- if $K_{DMT} > 10$; then $R_M = 0,32 + 2,18 \log K_{DMT}$

if values of $R_M < 0,85$ are obtained in the above relationships, R_M is taken equal to 0,85.

For additional information and examples, see Annex M.

ANNEX I.1
(informative)

Plate Loading Test (PLT)

(1) This is an example of deriving the undrained shear strength c_u; c_u can be obtained by using the following equation:

$$c_u = \frac{P_u - \gamma . z}{N_c}$$

where:

p_u is the ultimate contact pressure from the PLT results;

$\gamma . z$ is the total stress (density times depth) at test level included when the test is conducted in a borehole with a diameter smaller than three times the diameter or width of the plate;

N_c is the bearing capacity factor; for circular plates is:

- $N_c =$ 6 for PLT on the subsoil surface;
- $N_c =$ 9 for PLT in boreholes of depths greater than four times the diameter or width of the plate.

For additional information and examples, see Annex M

ANNEX I.2
(informative)

Plate Loading Test (PLT)

(1) This is an example of deriving the plate settlement modulus E_{PLT} (secant modulus).

(2) For loading tests made at the ground level or in an excavation where the bottom is at least five times the plate diameter, the plate settlement modulus E_{PLT} may be calculated from the general equation:

$$E_{PLT} = \frac{\Delta p}{\Delta s} . \frac{\pi \times b}{4} (1 - \nu^2)$$

where:

Δp is the selected range of applied contact pressure considered;
Δs is the change in total settlement for the corresponding change in the applied contact pressure Δp including creep settlements;
b is the diameter of the plate;
is Poisson's ratio for the conditions of the test.

(3) If not determined in other ways, is equal to 0,5 for undrained conditions of cohesive soils and 0,3 for non-cohesive soils.

(4) If the test is made at the base of a borehole the value of E_{PLT} may be calculated from the equation:

$$E_{PLT} = \frac{\Delta p}{\Delta s} . \frac{\pi \times b}{4} (1 - \nu^2) \times c_z$$

where:

c_z is a depth correction factor; an example for suggested values is given in figure I.1.

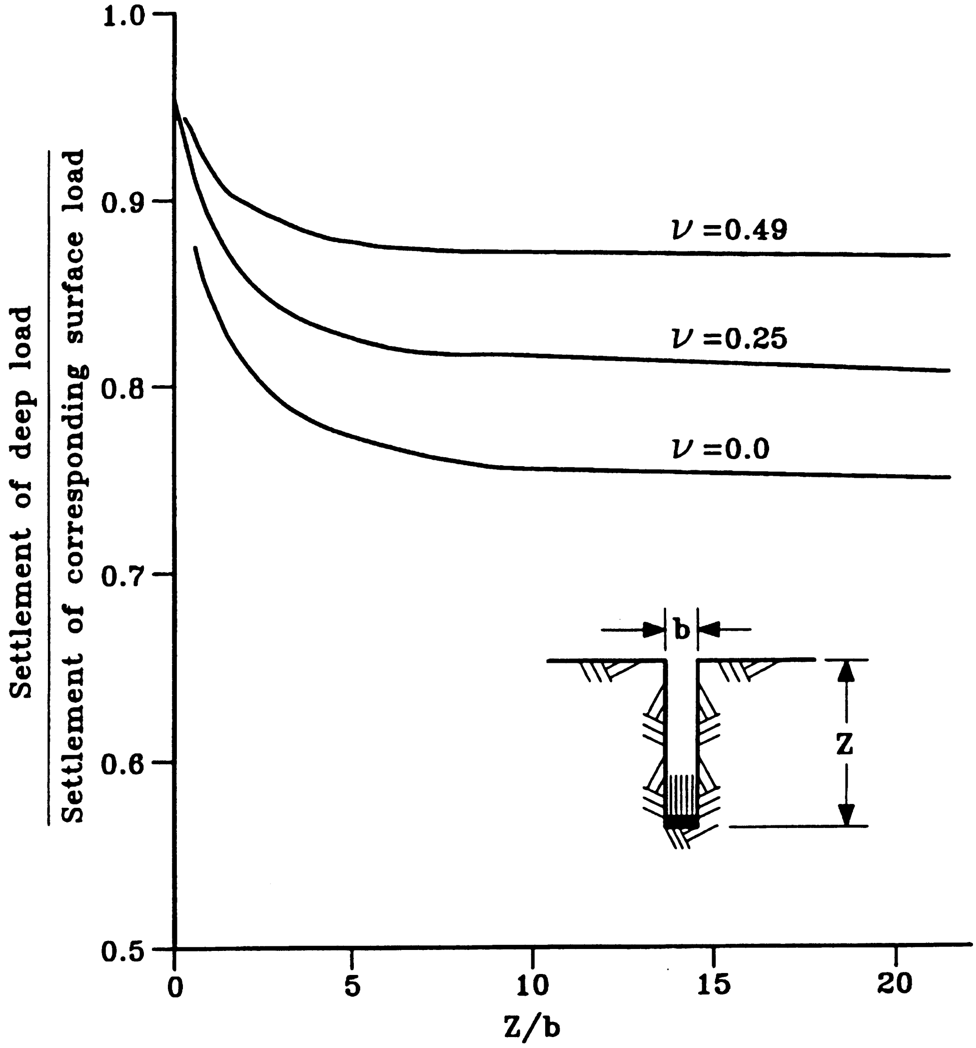


Figure I.1: Depth factor C_z as function of plate diameter b and depth z for PLT results obtained with a uniform circular load at the base of an unlined shaft (after Burland 1969)

For additional information and examples, see Annex M.

ANNEX I.3
(informative)

Plate Loading Test (PLT)

(1) This is an example of deriving the coefficient of subgrade reaction k_s; k_s may be calculated from the equation:

$$k_s = \frac{\Delta p}{\Delta s}$$

where:

Δp is the selected range of applied contact pressure considered;
Δs is the change in settlement for the corresponding change in applied contact pressure)p including creep settlements.

(2) The dimensions of the loading plate should be stated when calculating values of k_s.

ANNEX I.4
(informative)

Plate Loading Test (PLT)

(1) This is an example of deriving settlements directly. The settlements of the footing in sand may be derived empirically according to the relations given in figure I.3, if the ground beneath the footing to a depth larger than two times the width is the same as the ground beneath the plate (see figure I.2).

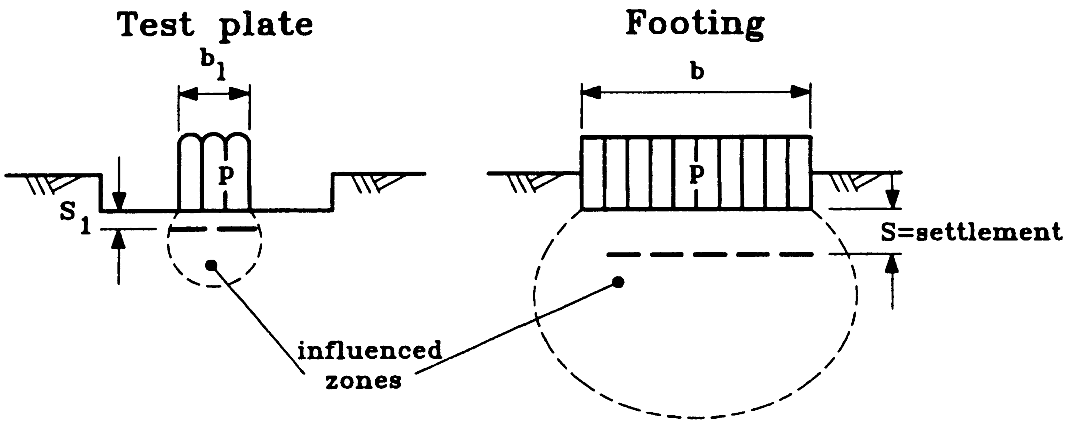


Figure I.2: Influenced area beneath a test plate and a footing

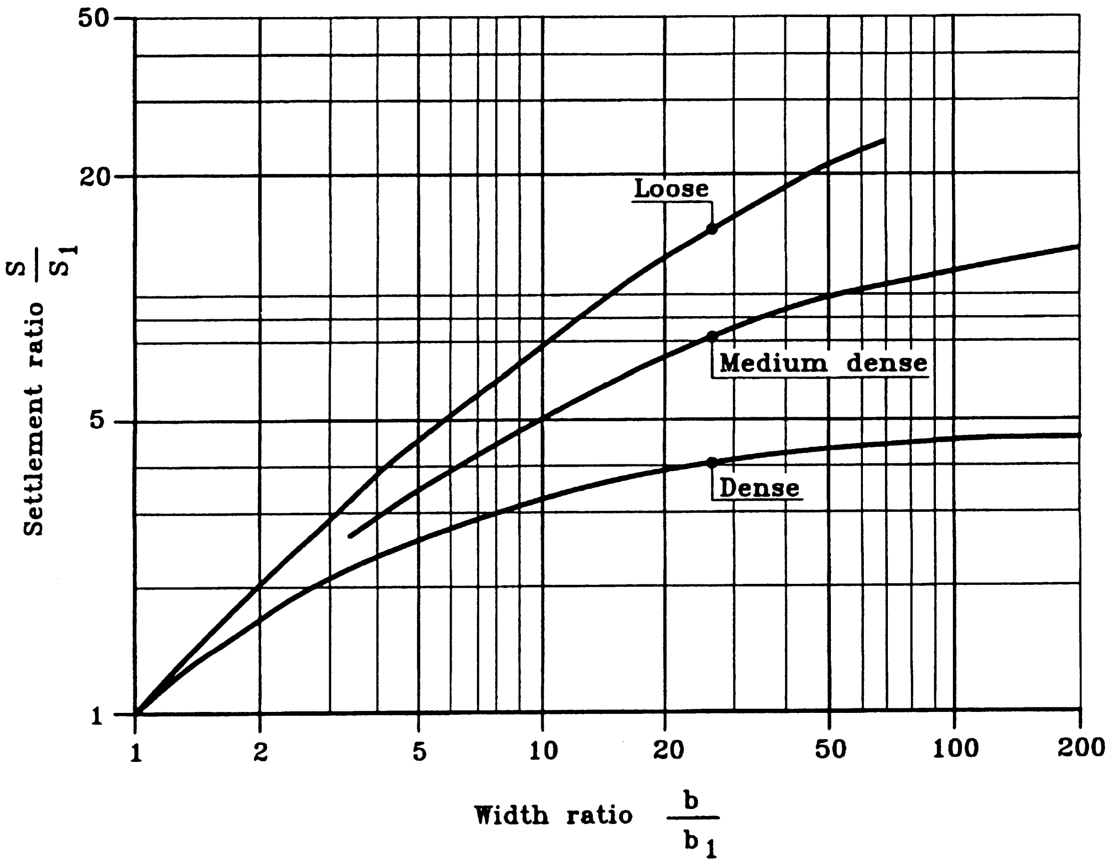


Figure I.3: Graph for calculations of settlement based on plate loading tests

For additional information and examples, see Annex M

ANNEX J
(informative)

Selection of the sampling method

Table J.1: Examples on sampling methods with respect to the sampling category in different soils; the abbreviations of the sampling methods are listed in table J.2

Soil type	**Suitability depends on e.g.**	**Category A**	**Category B**	**Category C**
		Sampling method		
Clay	–stiffness or strength –sensitivity	PS-PU OS-TNW-PU OS-TNW-PE*) OS-TCW-PE*) CS-DT BS-TP BS-FD	OS-TNW-PE OS-TCW-PE CS-ST AS*)	AS
Silt	–stiffness or strength –sensitivity –groundwater table	PS-PU OS-TNW-PU BS-TP	CS-DT OS-TCW-PE	AS CS-ST
Sand	–sizes of the particles –density –groundwater table	BS-TP OS-TNW-PU*) OS-TCW-PE*)	OS-TCW-PE CS-DT	AS CS-ST
Gravel	–size of the particles –density –groundwater table	BS-TP	OS-TCW-PE*) CS-DT*)	AS
Peat	–state of decay	PS-PU OS-TNW-PU BS-TP	CS-ST AS*)	AS
*) can be used only on very favourable conditions.				

(2) The quality class of the sample for laboratory tests obtained by a certain sampling method also depends very strongly on:

– details of the sampler;

– carefulness of the sampling procedure.

Table J.2: Sampling methods and their abbreviations used in table J.1

OS-TNW-PU	open-tube samplers, thin-walled/pushed
OS-TNW-PE	open-tube samplers, thin-walled/percussion
OS-TCW-PE	open-tube samplers, thick-walled/percussion
PS-PU	Piston samplers/pushed
CS-ST	rotary core drilling, single tube, vibrocoring
CS-DT	rotary core drilling, double or triple tube
AS	Augering
BS-TP	block samplers/test pit
BS-FD	block samplers/from depth

For additional information and examples, see Annex M

ANNEX K
(informative)

This is an example of simplified weathering classification in six grades.

Grade	Degree of decomposition	Diagnostic features in samples and cores
I	Fresh rock	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.
II	Slightly weathered rock	Discoloration indicates weathering of rock material and discontinuity surfaces.
III	Moderately weathered rock	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.
IV	Highly weathered rock	More than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
V	Completely weathered rock	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
VI	Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

For additional information and examples, see Annex M

ANNEX L
(informative)

Sample guidelines for groundwater measurements

L.1 General

(1) The groundwater level and the pore water pressure in soil and rock have a great influence on many geotechnical questions: slope stability, bearing capacity, settlements, drainage, erosion and frost actions. Also the interpretation of other site investigation results are influenced by the groundwater situation. Therefore groundwater measurements are important parts of site investigations and can also be a part of a monitoring system in e.g. slopes, dams and groundwater lowering projects.
In some cases it has also been found that the groundwater quality can have an important role for the design of different constructions e.g. durability of steel piles.

(2) Insufficient groundwater measurements may lead to misjudgement of the groundwater conditions and the following consequences:

– an underestimate of the groundwater level or the permeability can give e.g. unstable constructions or increased settlements;
– an overestimate can e.g. give increased construction costs.

(3) There are many factors influencing the groundwater situation: geology, stratigraphy, climate, the atmospheric pressure, and also human activities. In figure L.1 the natural fluctuations in different kinds of ground are presented compared to the actual precipitation. Better correlations may sometimes be obtained by comparing the groundwater fluctuations with the net precipitation or an averaging of the precipitation for a certain period.

(4) Normally, the groundwater pressure is not hydrostatic due to layers of various permeability and groundwater flow which should be considered.

(5) In this annex some additional information about groundwater measurements is presented in order to facilitate the fulfilment of the requirements in section 14.

(6) Guidance for the arrangements of some groundwater measuring systems with respect to different aquifer systems can also be found in the examples mentioned in Annex M.

(7) When locating the piezometers or stand pipes for a certain project it is necessary to know the purpose of the measurements and the soil layer sequence. In complex soil layers and hilly terrain a number of measuring points can be required to model the groundwater conditions. This means that in hilly terrain measuring stations should be located where major changes in slope inclination occur.

(8) In each measuring station the elevation for each piezometer should be chosen with respect to the purpose of the measurements and the permeability variations of the ground.

(9) When the groundwater table should be observed in soils with a high and rather constant permeability, only one pipe penetrating the water table is normally required.

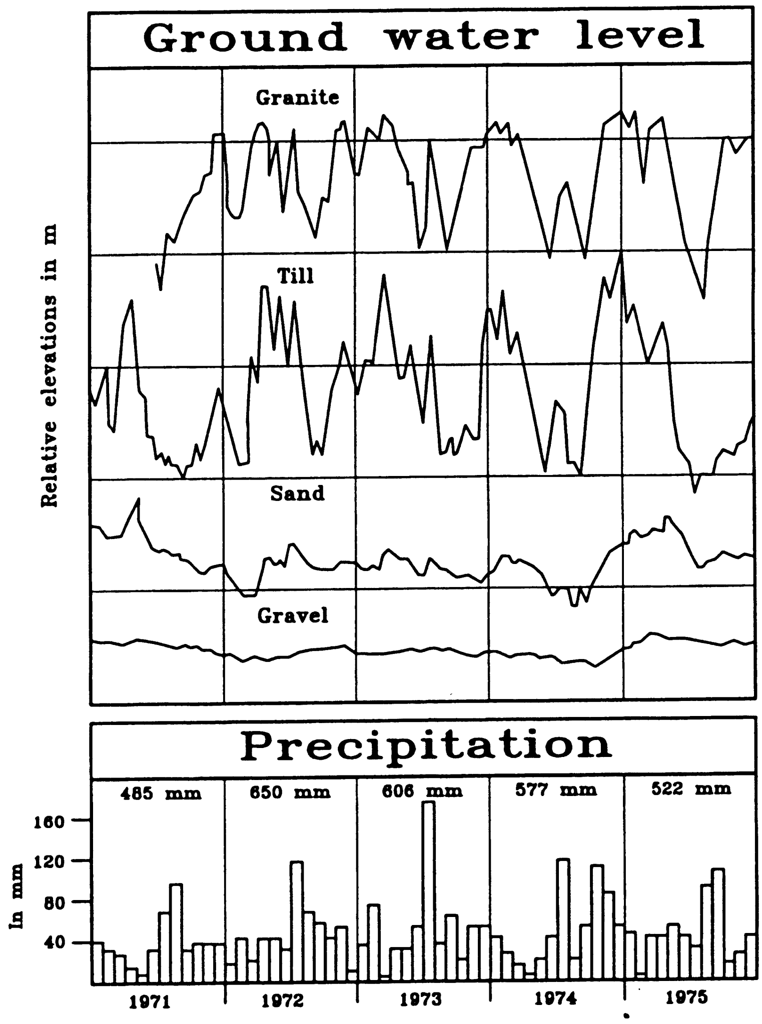


Figure L.1: Groundwater fluctuations in different kinds of ground compared to the precipitation

(10) In clays or in soils with a variable permeability with depth a number of (at least three) piezometers are normally required to obtain a groundwater pressure profile.

(11) Installation of the groundwater pipes or piezometers may be performed by pushing or by preboring.

(12) In soft clay it is normally sufficient to push the penetrometer carefully into the ground. However, preboring to the groundwater table is required to keep the piezometer tip or filter water saturated during installation. The remaining annular spacing between the pipe and the soil can be filled with e.g. bentonite to prevent surface water inflow.

(13) When pushing the piezometer into clay, an excess pore water pressure up to about 15 times the undrained shear strength of the clay can be generated. This excess pore pressure will dissipate during the stabilizing period. The stabilizing period might be between 1 day and 20 days depending on the permeability of the soil.

(14) In order to prevent the piezometer from overloading, the penetration rate should be kept sufficiently low or a piezometer with sufficiently high upper pressure limit shall be chosen, provided the precision is satisfactory.

(15) Drilling water additives should be avoided when preboring for groundwater pipes or piezometers.

(16) When installing in prebored holes a filter should be arranged around the filter tip. Above that filter zone a seal should be made with a permeability lower than that of the surrounding ground.

L.2 Sample guidelines for presentation of groundwater measurements

(1) The presentation of the groundwater measurements can be made in different ways depending on the purpose of the measurements and the presentation of other ground investigations. As the groundwater conditions normally influence the interpretation of the total ground situation the results should be presented in both plans and profiles. In addition it can be necessary to present long term measurements and pore pressure profiles in special diagrams.

(2) In a plan the elevation of the groundwater table can be indicated at its actual position as follows:

o No. 16
GW +8.30 82-03-15
GL +9.20

where:

o	is the location of the borehole;
No. 16	is the borehole No.;
GW +8.30	is groundwater level + 8.30;
82-03-15	is date of measurement;
GL +9.20	is elevation of ground surface.

(3) The results of groundwater observations in boreholes, pipes or piezometers can be presented in a simplified way in sections according to figure L.2.

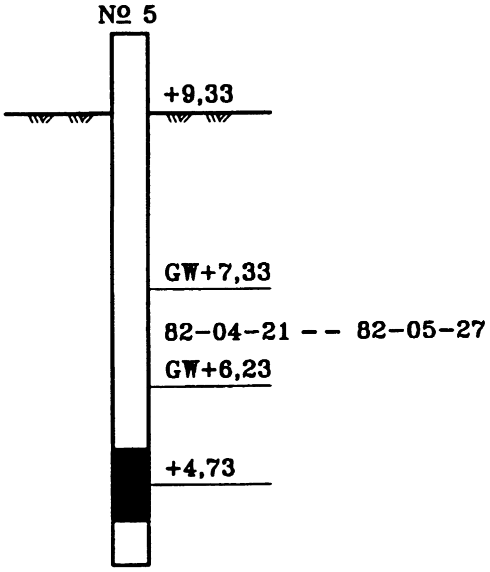


Figure L.2: Presentation of groundwater measurements for a certain period in sections

Figure L.2 indicates a measurement made in a piezometer where the ground surface elevation was +9,33 m and the centre of the filter was located at elevation +4,73. During the measuring period 1982-04-21 to 1982-05-27 the highest observed level was +7,33 m and the lowest +6,23 m. The borehole No. 5 is indicated above.

(4) The results from a number of piezometers at different depths in one measuring station can be compiled like the examples in figure L.3, where also a reference hydrostatic line is indicated. Figure a) indicates the pore pressure measured at each level at three occasions. Figure b) indicates the maximum and minimum pressures measured during the measuring period 1989-04-10 to 1989-10-12 and the mean values from each elevation.

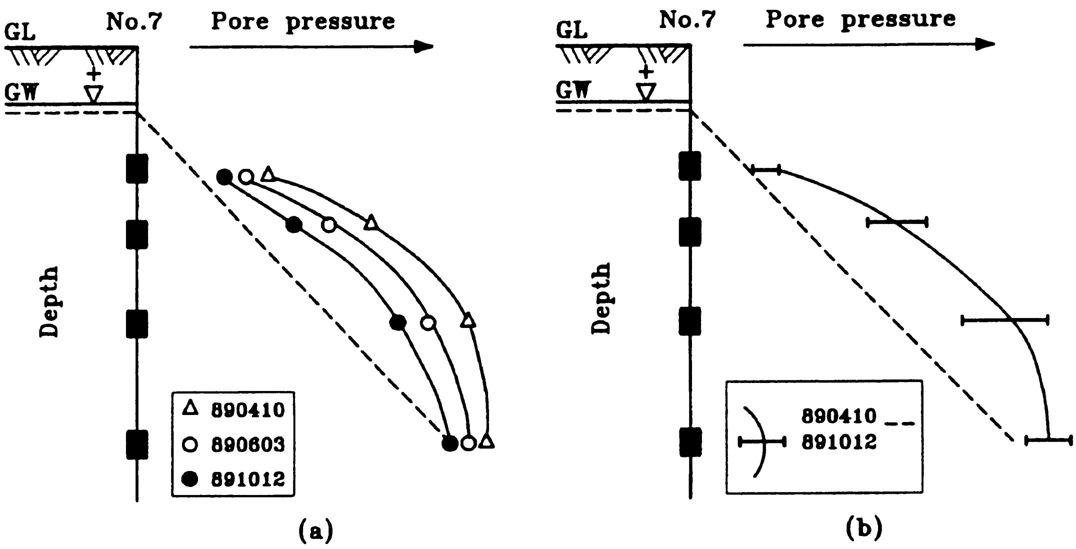


Figure L.3: Presentation of results from a number of piezometers in one measuring station designated No. 7

(5) In case of presentation of long term measurements the example in figure L.4 can be followed. Alternatively the groundwater level can be expressed in depth below ground surface in m or as a pore pressure in kPa.

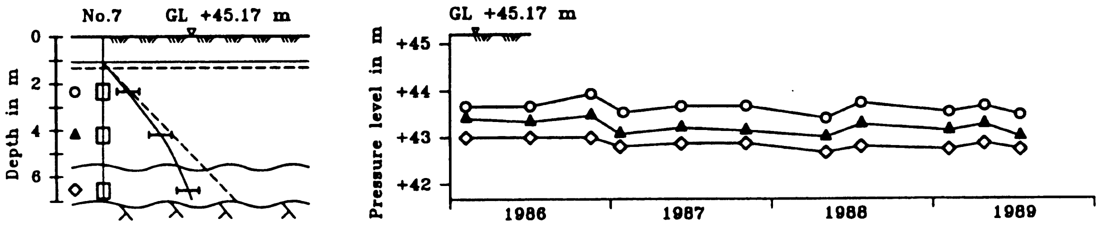


Figure L.4: Presentation of long term measurements from three piezometers in one measuring station

L.3 Sample guidelines for derivation of groundwater pressure

(1) The natural groundwater pressure is part of the hydrological cycle being influenced by the precipitation, evapotranspiration, snow melting, surface run off etc.

(2) In order to establish a model of the groundwater situation for a building or a civil engineering project site and the surrounding area, available hydrogeological information should be compiled and compared to the actual groundwater measurements. Such information could be:

– water level fluctuations;

– geohydrological maps;

– previous measurements in the surroundings;

– typical water levels of surface water or in wells;

– long term measurements in similar aquifers.

(3) The groundwater measurements for a project normally only contain a short series of measurements. In these cases it is important to make a prediction of the expected groundwater pressure for the actual design situation. Such a prediction can be based on the model mentioned above and on long term measurements of the groundwater in a similar aquifer in the same region as the project in combination with a short term measurement on the site.

(4) Using statistical methods it has been possible to predict the groundwater pressure within a few decimeters based on 15 to 20 years measurements in a reference system and a 3 month measuring period on the actual site, see figure L.5.

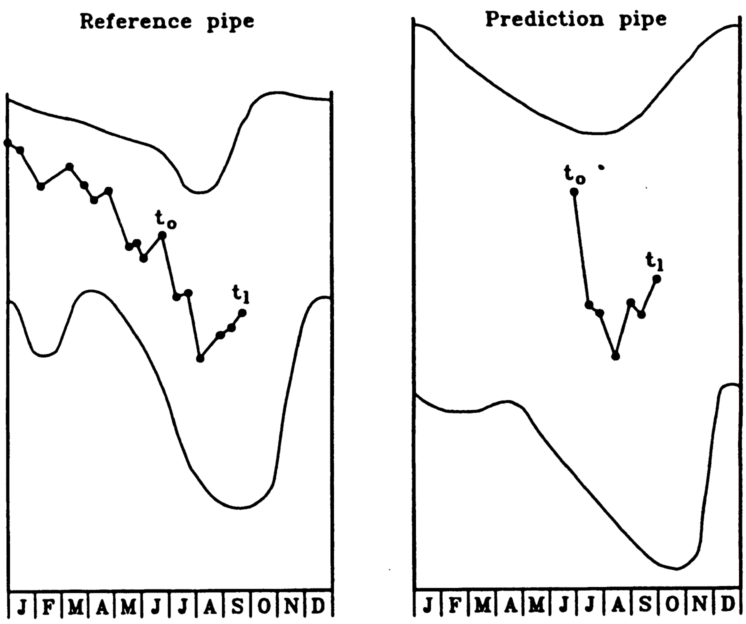


Figure L.5: Left: the maximum and minimum groundwater level for a reference pipe; Right: the actual measurements together with the predicted maximum/minimum groundwater levels.

(5) A conceptual model can also simulate the groundwater fluctuations by a conceptual model. Precipitation and air temperature can be used as input in the model. The groundwater response is calibrated against long term measured groundwater fluctuations in the region.

For additional information and examples, see Annex M.

Annex M
(Informative)

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The listing here after contains documents that give additional information and examples on the following tests.

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