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# Determination of representative values from derived values for verification with limit states with EN 1997

*Guidelines for the application of the 2nd generation of Eurocode 7: Geotechnical design*

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# Contents

## Abstract

This document summarizes the global process of designing geotechnical structures, as given in the 2nd Generation EN 1997, and highlights the step where the designer must determine representative values for limit state verifications based on ground investigation data.

This guideline explains the different concepts related to the ground property values, including measured, derived, representative, nominal, indicative, characteristic, design and best estimate values.

The text describes procedures for determining representative values in special cases, which are common in practice. These situations include dealing with limited samples, with depth-dependent properties, and data from different sources.

Five examples are presented and solved, covering various aspects such as different geotechnical structures and limit states, different number of derived values, different distribution function, and independent and depth- dependent ground properties.

This guideline is an essential tool for designers to understand how to determine representative values for the design and verification of geotechnical structures, in accordance with the procedures outlined in the Second Generation of Eurocode 7.

## Foreword

### *JRC Foreword*

The construction ecosystem is of strategic importance to the European Union (EU), as it delivers the buildings and infrastructures needed by the rest of the economy and society, having a direct impact on the safety of persons and the quality of citizens' life. The construction ecosystem includes activities carried out during the whole lifecycle of buildings and infrastructures, namely design, construction, maintenance, refurbishment and demolition. The industrial construction ecosystem employs around 25 million people in the EU and provides an added value of EUR 1 158 billion (9.6% of the EU total)[1,2,3].

The construction ecosystem is a key element for the implementation of the European Single Market and many other important EU strategies and initiatives. The European Green Deal (COM(2019) 640 final) aims to achieve climate neutrality for Europe by 2050, and relies on numerous initiatives, noteworthy:

- the New Circular Economy Action Plan (COM(2020) 98 final) and the New Industrial Strategy for Europe (COM(2020) 102 final) intending to accelerate the transition of the EU industry to a sustainable model based on the principles of circular economy;
- the revision (COM(2022) 144 final) of the Construction Products Regulation (Regulation (EU) No 305/2011) aiming to enable the construction ecosystem's contribution to meeting climate and sustainability goals and embrace the digital transformation of the built environment;
- the New EU Strategy on Adaptation to Climate Change (COM (2021) 82 final) supported by the recent Commission Communication on managing climate risks (COM(2024) 91 final) that reinforces the need to address climate change concerns to guarantee resilience and sustainability of built structures and infrastructures and to ensure regular science-based risk assessments;
- the first European Climate Risk Assessment (EUCRA) report which highlights the importance of EU policies for the built environment, including updating construction standards and related European datasets.

Furthermore and recognizing that the EU's ambitions towards a climate neutral, resilient and circular economy cannot be delivered without leveraging the European standardization system, the European Commission presented a new Standardization Strategy (COM(2022) 31 final). The strategy spots standards as "*the silent foundation of the EU Single Market and global competitiveness*".

The EU has put in place a comprehensive legislative and regulatory framework for the construction sector, including European standards (EN). Within this framework, the Eurocodes are a series of 10 European standards, EN 1990 to EN 1999, providing common technical rules for the design of buildings and other civil engineering works. In fact, the Commission Communication on managing

[1] Commission staff working document: Scenarios for a transition pathway for a resilient, greener and more digital construction ecosystem (https://ec.europa.eu/docsroom/documents/47996)

[2] Council of the EU, Press release 30 June 2023, https://www.consilium.europa.eu/en/press/press-releases/2023/06/30/council-adopts-position-on-the-construction-products-regulation/

[3] Transition Pathway For Construction, European Commission, DG GROW, https://ec.europa.eu/docsroom/documents/53854

climate risks directly mentions the Eurocodes, highlighting the role of building and infrastructure standards in integrating climate adaptation and resilience.

The Commission Recommendation 2003/887/EC on the implementation and use of the Eurocodes for construction works and structural construction products recommends undertaking research to facilitate the integration into the Eurocodes of the latest developments in scientific and technological knowledge. In this context, the so-called second generation of the Eurocodes is under development under Mandate M/515 and expected to be available by 2026. The second generation Eurocodes incorporates improvements to the existing standards and extends their scope by embracing new methods, new materials, and new regulatory and market requirements, including considerations for climate change impact on structural design.

In order to support the implementation of the second generation EN 1997 “Geotechnical Design”, CEN Technical Committee 250/Sub-Committee 7 (TC 250/SC 7) produced a series of guidelines addressing the most important new aspects in the standard. The series of guidelines contains the following documents:

Guideline C1 – “Determination of representative values from derived values for verification of limit states with EN 1997”

Guideline C2 – “Assembling the Ground model and the derived values”

Guideline C3 – “Reliability-based verification of limit states for geotechnical structures”

Guideline C4 – “Implementation of design in execution and service life”

Within the framework of Administrative Arrangements between the European Commission’s Joint Research Centre (JRC) and DG GROW on support to policies and standards for the construction ecosystem, JRC is engaged in activities facilitating the implementation and practical use of the second generation Eurocodes. In this context, the guidelines by TC 250/SC 7 are published as JRC technical reports, part of the series “*Support to the implementation, harmonization and further development of the Eurocodes*”.

We hope that this report will provide a sound and helpful basis for the implementation and use of the second generation EN 1997 “Geotechnical Design” and contribute to training and education of the professionals engaged in geotechnical design, supporting further skills development for individuals' careers and also EU's competitiveness. The report is available to download from the “Eurocodes: Building the future” website (http://eurocodes.jrc.ec.europa.eu).

The authors have sought to present useful and consistent information in this report. However, users of the information contained in this report must assess if such information is suitable for their purposes.

Ispra, September 2024

François Augendre, Head of Unit

Georgios Tsionis, Deputy Head of Unit

E. 3 Built Environment Unit
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Joint Research Centre (JRC)
European Commission

***CEN/TC 250/SC 7 Foreword***

With the adoption of the 2nd Generation of Eurocode 7 – Geotechnical design - Member States will need to implement new procedures in many different topics, not considered in the first generation of the Code such as the assessment of Representative values, the Ground Model, the use of Reliability methods and the implementation of design in the execution phase. To facilitate the implementation of Eurocode 7, CEN Technical Committee 250/Sub-Committee 7 (TC 250/SC 7) therefore decided to produce a suite of Guidelines, one for each of the most relevant new aspects, to help ensure that the objectives of the code writers are reached in practice.

According to Clause 4 of Eurocode 7, Basis of design, the fundamental step to ensure that the prescribed reliability in geotechnical design is reached, is the development of a representative Geotechnical Design Model being the combination of the Ground Model and the set of design values of relevant geotechnical properties needed for verifications.

The first Guideline of the suite addresses the fundamental process of determining design values of geotechnical properties from derived values, obtained from a variety of activities of the ground investigation. Once the representative values of properties, either characteristic (through statistical evaluation) or nominal (cautious estimate), are determined, design values are obtained by applying the partial factors for a design situation.

The process of assembling the Ground Model is addressed in the second Guideline, where the importance of the progressive upgrading of the Model with an increase of knowledge of the ground within the Zone of Influence of the specific structure is highlighted. Note that the concept of the Zone of Influence in the second generation of Eurocode 7 is sensibly widened after environmental and seismic aspects have become central in planning the Ground Investigation and processing the results.

A significant novel aspect is the use of Eurocode 7 in combination with reliability-based methods. This is likely to lead to a very important evolution of safety assessments in geotechnical design in the coming years. Eurocode 7 like other Eurocodes is fundamentally reliability-based although safety verifications are tied to the application of partial factors. However, in the second generation of Eurocode 7, Clause 4 explicitly states that reliability-based design is only one of the options to verify limit states in geotechnical design. As these methods are not usually addressed in most of the teaching programs, it has been decided to dedicate a specific guideline to reliability-based verification of limit states coherently with the safety concepts of the Eurocodes. Moreover, the objective of this third Guideline is to provide information for code developers to perform reliability calibration of partial factors given the preparation of National Annexes.

It is well known by geotechnical designers that a great contribution to the reliability of a geotechnical construction relies upon its execution and its real performance. This is why Eurocode 7 now specifically dedicates a full clause in EN 1997-1 to the implementation of design during execution and service life. The fourth Guideline presents measures to ensure that the design is correctly implemented in the different construction phases and how to document the activities carried out to this scope on the construction site. After a general description of the suggested rules and methods, the guideline describes good practice for establishing a Supervision Plan, Inspection Plan, Monitoring Plan and Maintenance Plan, how to establish acceptance criteria and limit values and gives contingency measures that might be utilized when an acceptance criterion/limit value is reached. This is implemented for typical geotechnical constructions such as embankments, bored piles, rigid inclusions, and groundwater control.

The upgrading of Eurocode 7 to the methods of modern Geotechnics was a very difficult task that has been achieved thanks to the strong involvement of many dedicated people from all European countries. However, this process cannot be considered concluded with the publication of the new Eurocode documents alone and there will be a long route to implementing Eurocode 7 into the engineering practices of the many countries involved.

The involvement of the many members of the Task Group C1 to C4 of SC7, who prepared these Guidelines is very gratefully acknowledged. These Task Groups have performed tremendous work over almost 4 years to compile knowledge and experience in European geotechnical engineering to draft these Guidelines.

It is therefore strongly believed that for the transition from the 1st to the 2nd Generation of Eurocode 7, these Guidelines will be very helpful in clarifying the new concepts and methods. The Guidelines will also provide didactic background material that could not be presented in the Code.

Giuseppe Scarpelli, Coordinator Task Groups C1 to C4

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### *Editor*

Jose Estaire

# 1 Introduction

## 1.1 Context

In modern design codes, geotechnical design has evolved from experience and judgment, based on overall factor of safety approaches, to semi-probabilistic design methods, with partial factors applied to actions and to strength properties and resistances, as well as to probabilistic design methods.

EN 1997-1, 4.2.1 (2) states that there are five methods to verify limit states of geotechnical structures: partial factor method, other reliability-based methods, prescriptive rules, testing, and the Observational Method, as shown in Figure 1. They are equally valid, but, since it is the only treated specifically in EN 1990-1[4], the partial factor method can be considered the main method to verify limit states under the Eurocode system. In the context of the partial factor method, one of the key issues in the verification of the limit states is the determination of the representative and design values of the ground and geometrical properties and of the actions that should be used in such verifications.

The present document serves as a guideline for the determination of representative and design values of ground properties from the derived values obtained in the different activities of the ground investigation. Representative values of geometrical properties and of actions are also mentioned, but with less extension.

[4] Eurocode EN 1990-1: Basis of structural and geotechnical design-Part 1: New structures. EN 1990-1 establishes the principles and requirements for the safety, serviceability, robustness and durability of structures, including geotechnical structures, that are common to all Eurocodes parts, appropriate to the consequences of failure, describes the basis for structural and geotechnical design and verification and gives guidelines for related aspects of structural reliability. More information: https://eurocodes.jrc.ec.europa.eu/EN-Eurocodes/eurocode-basis-structural-design

**Figure 1.** Principles of limit state design (EN 1997-1, section 4.2).

**4.2 Principles of limit state design**

**4.2.1 General**

(1) <REQ> EN 1990-1, 5, shall apply.

(2) <REQ> Limit states for geotechnical structures shall be verified by one or more of the following methods:

– partial factor method (4.4) or other reliability-based methods;
– prescriptive rules (4.5);
– testing (4.6);
– the Observational Method (4.7).

(3) <REQ> In addition to (2), verification of limit states shall cover the potential for:

– failure in the ground;
– failure of structural elements in contact with the ground;
– combined failure of the ground and structural elements.

(4) <RCM> When the uncertainty in ground properties is too large to ensure the level of reliability required by EN 1990-1, limit states for geotechnical structures should not be verified using the partial factor method.

(5) <REQ> The verification of limit states by any of the methods given in (2) shall provide a level of reliability no less than that required by EN 1990-1.

*Source: EN 1997-1/4.2.*

## 1.2 Objectives

The main objectives of the document are:

— to explain the principles and the system presented in EN 1997 that describes the route a value should follow after it is obtained in a ground investigation activity to when it is used in the verification of a limit state, and

— to give practical procedures for designers to determine the representative and design values to be used in such verifications of limit states.

To achieve these objectives, the document covers the following topics:

— Different input data (desk study, site inspection, in-situ and laboratory tests, results from monitoring and testing of geotechnical elements).

— Data related to soil, rock and groundwater.

— Ground behaviour related to different geotechnical structures.

— Representative values determined either as nominal or characteristic values.

— Usage of the Ground Investigation Report.

— Analysis of different examples.

## 1.3 Target audience

The target audience of this Guideline is the following:

- Practitioners – Competent engineers
- Practitioners – Graduates
- Expert specialists
- Software developers
- Educators
- National regulators
- Degree students

This target audience list is part of the whole range of Eurocode users that, according to Denton and Angelino (2022) can be divided in the categories shown in Table 1. The implication of addressing this target audience is that this Guideline must focus on practical guidance rather than extensive theoretical backgrounds.

**Table 1**. Categories of Eurocode users.

| **Category of Eurocode user** | **CEN/TC 250 statement to intent to meet users´ needs** |
|---|---|
| Practitioners – Competent engineers | We will aim to produce Standards that are suitable and clear for all commom design cases without disproportionate levels of effort to aply them |
| Practitioners – Graduates | We will aim to produce Eurocodes that can be used by Graduates where necessary supllemneted by suitable guidance documents and textbooks and under supervision of an experienced practitiones when appropriate |
| Expert specialist | We will aim not to restrict innovation by providing freedom to experts to apply their specialist knowledge and expertise |
| Software developers | We will aim to provide unambiguous and complete design procedures. Accompanying formulae will be provided for charts and tables where possible |
| Educators | We will aim to use consistent underlying technical principles irrespective of the intended use of a structure (e.g. bridge, building) and that facilitate the linkage between physical behaviour and design rules |
| National regulator | We will endeavour to produce standards thar can be referenced or quoted by National Regulations |
| Private sector businesses | We will continue to promote thechnical harmonisation across Europena markets in prder to reduce barries to trade |
| Clients | We will produce Eurocodes that enable the design of safe, serviceablem robust and durable structures, aiming ro promote cost-effectiveness throughout their whole lyfecycle, includong design, construction and maintenance |
| Other CEN/TCs | We will engage proactively to promote effective collaboration with those other CEN/TCs that have shared interests |

*Source: Denton and Angelino (2022).*

## 1.4 Scope

The present document leads the target audience through the process to determine the representative and design values to be used in the verifications of limit states for geotechnical structures.

The text explains and develops the ideas behind EN 1997-1/4.3.2 'Material and product properties', EN 1997-1/4.3.3 'Geometrical properties' and EN 1997-1/Annex A.

Based on the objectives and target audience given above, the contents of this document aim at practical application. Generally speaking, textbook material has been avoided, except when deemed necessary for coherence and readability. As much as possible, the approaches in this document have a proven track record in practical application. Ongoing developments in academia to improve the state-of-the-art of geotechnical reliability analysis may be mentioned in references but are not extensively discussed. These may be included in future updates of this document when deemed sufficiently robust for practical use.

## 1.5 Application of this Guideline

As noted in Section 1.1 of this document, the use of the 'partial factor method' to verify the safety of geotechnical structures with respect to the exceedance of limit states is the design method in EN 1997. In this context, the appropriate determination of the representative and design values of the ground properties to be used in calculations for the partial factor method is an essential task for the success of the design process.

This document should be used in the first steps of such a design process when the designers must determine the values of the ground properties they will use in the calculations, based on the values obtained from the ground investigation and compiled in the Ground Investigation Report (GIR) or, in some countries, in the Geotechnical Design Report (GDR).

## 1.6 Outline

This Guideline starts by providing a general introduction followed by the following chapters:

— Chapter 2 presents a summary of the tasks involved in the global process for the design of geotechnical structures, as given in EN 1997. This process provides the frame for the steps the designer must take to determine the representative and design values to be used in the limit state verifications, based on the type, quantity and quality of the ground property values obtained from the ground investigation.

— Chapter 3 highlights the fact that the values involved in the limit state verifications are not only values related to ground properties, but also, on one hand, to actions and, on other hand, to geometrical properties, which include, among others, some of the groundwater related levels, the boundaries between geotechnical units and the geometrical description of rock discontinuities.

— Chapter 4 is the core of the document as it explains the different concepts related to the ground property values that are named as: measured, derived, nominal, characteristic, representative, design, and also indicative and best estimate values and how these values are determined and used.

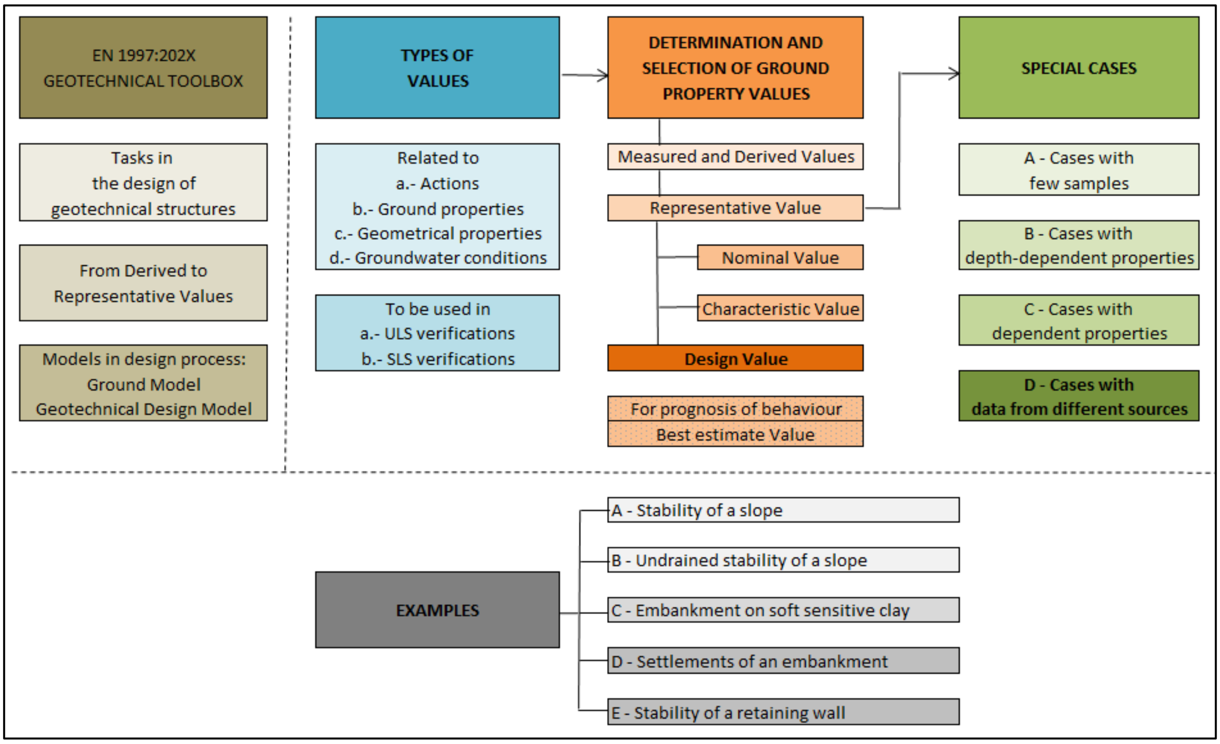
— Chapter 5 describes procedures to determine the representative values in special cases, often very common in practice, such as when there are very few samples, when the properties are depth-dependent and when there are data from different sources.

— Chapter 6 summarises five examples that cover different aspects of the selection and determination of ground property values for design situations. These are: different geotechnical structures and limit states, different number of derived values, different ground property distribution function, and independent and depth- dependent ground properties.

— Chapter 7 makes a summary of the most relevant topics developed in the document: the description of the two possible ways to determine the representative value of a ground property, as established in EN 1997-1, the introduction of a new type of estimate in the determination of the representative vale, not included in EN 1997-1 and the description of procedures to determine representative values in special cases, although very common in usual practice.

Figure 2 shows a visual outline of the contents of this Guideline.

**Figure 2.** Visual outline of this guideline.




*Source: Developed by the authors.*

# 2 Second Generation EN 1997 – Geotechnical Toolbox

## 2.1 Introduction

The aim of this chapter is to give an appropriate context of the task of determining the representative values, the main objective of this document, in the global process for the design and verification of geotechnical structures.

## 2.2 Tasks in the design of geotechnical structures

### 2.2.1 Global overview

The design of a geotechnical structure, according to EN 1997, comprises five major tasks, as shown in Figure 3:

— *Geotechnical reliability assessment*: defining the procedure to place the geotechnical structure into a Geotechnical Category (GC), based on the combination of the consequence of failure (by the Consequence Class, CC) and on the geotechnical complexity of the site (by the Geotechnical Complexity Class, GCC). The GC is used to specify the extent and amount of several measures that must be taken, in order to achieve the appropriate level of reliability required.

— *Ground investigation*: whose main outputs, as stated in the Note to EN 1997-25/4.1(4), are the Ground Model that 'comprises the geological, hydrogeological, and geotechnical conditions at the site' and the Ground Investigation Report that compiles the results of ground investigation (See 'JRC Report - Assembling the Ground Model and derived values - Second Guideline of this series' for more information on the Ground Model).

— *Design verification*: covering all the procedures to verify that no limit states are exceeded in any design situations that the structure may encounter during its service life.

— *Design implementation during execution*: ensuring that the structure is constructed while meeting the design assumptions and other detailed plans developed during the design phase. (See European Commission: Joint Research Centre, Bogusz, Caplane et al., 2024 for more information).

— *Reporting*: all the work carried out during the design and execution of the geotechnical structure must be documented by carrying out the following reports:

- Ground Investigation Report (GIR),
- Geotechnical Design Report (GDR) and
- Geotechnical Construction Record (GCR).

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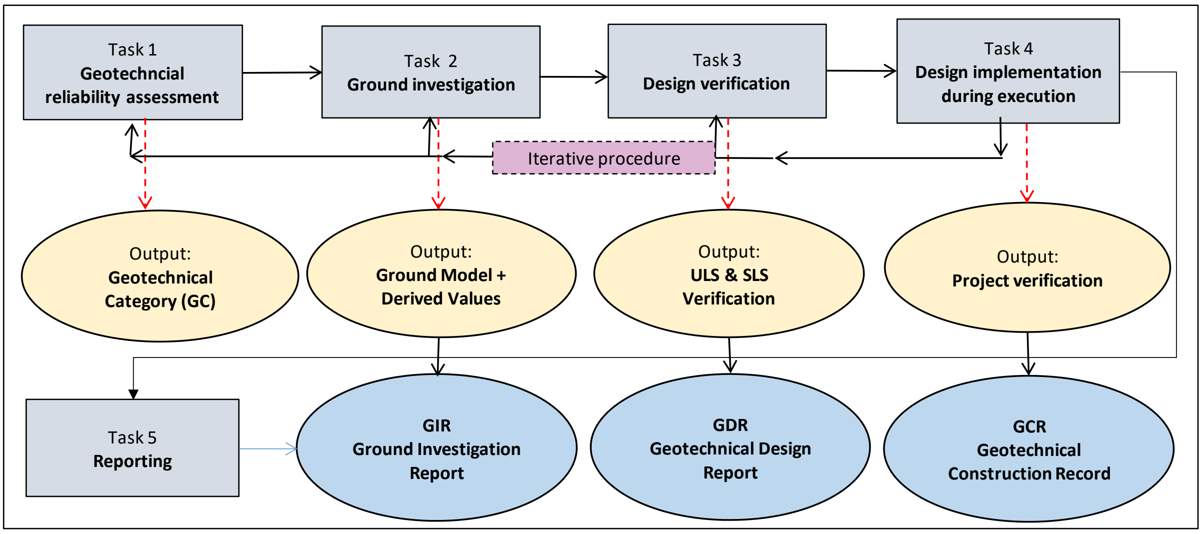
[5] Eurocode EN 1997-2: Geotechnical design - Part 2: Ground properties. EN 1997-2 establishes rules for obtaining information about the ground at a site, as needed for the design and execution of geotechnical structures. This document covers guidance for planning ground investigations, collecting information about ground properties and groundwater conditions, and preparation of the Ground Model. More information: https://eurocodes.jrc.ec.europa.eu/EN-Eurocodes/eurocode-basis-structural-design

**Figure 3.** Tasks in the design and execution of a geotechnical structure.




*Source: Developed by the authors.*

### 2.2.2 Ground investigation and derived values

EN 1997-2/4.2(1) states that the *'Derived values of the properties of a geotechnical unit shall be established from data gathered during the desk study, site inspection, preliminary and design investigations, and monitoring of the ground and structures'*. Furthermore, all the derived values, relative to different ground properties for each of the various geotechnical units identified during the ground investigation, must be referenced in the Ground Model. In this respect, a geotechnical unit is a '*volume of ground that is identified as a single material'*. All this information should be documented in the Ground Investigation Report (or in the Geotechnical Design Report, according to the national practice).

For the sake of explanation in this document, the values obtained in the different activities of the ground investigation (desk study, site inspection, preliminary and design investigation, and monitoring of the ground and structures) can be classified according to the terminology for the values given in Table 2 and shown in the upper part of Figure 7 (shown in section 2.3 of this document). On other hand, Table 3 collates some examples of values obtained during the different activities of the ground investigation for some geotechnical structures.

**Table 2.** Values obtained during the ground investigation activities.

| **Activity** | **Value** (1) | **Definition** |
|---|---|---|
| **Desk study** | Historical values (2) | Values of ground properties obtained during the desk study from information collected by previous investigations on the specific site, its surrounding or comparable cases. |
| **Site inspection** | Assessed values | Values of ground properties obtained during the site inspection mainly by visual observation. In many cases, these 'assessed values' are categorical values (3). |
| **Field and laboratory ground investigation** | Measured values | Values of ground properties obtained by measurements performed during field or laboratory tests. |

| Activity | Value (1) | Definition |
|---|---|---|
| **Monitoring or Observation of the ground or the structure** | Monitored values | Value of ground properties obtained by monitoring or observation of the ground or the structure, during its execution, service life or after failure. |

1 Note: these terms for values are not used in EN 1997-1.
2 Note: historical values can be regarded as derived, nominal, characteristic, representative or design values (see Figure 7).
3 Note: categorical values are values that can be divided into groups or categories, which can be labelled with names or numerical values. These categories are based on qualitative characteristics. For instance, rock mass quality or zones of degradation, mentioned in EN 1997-2/C4 (7), can be termed as 'categorical values'.

*Source: Developed by the authors*

**Table 3.** Examples of values obtained during the ground investigation for specific limit states relative to some geotechnical structures.

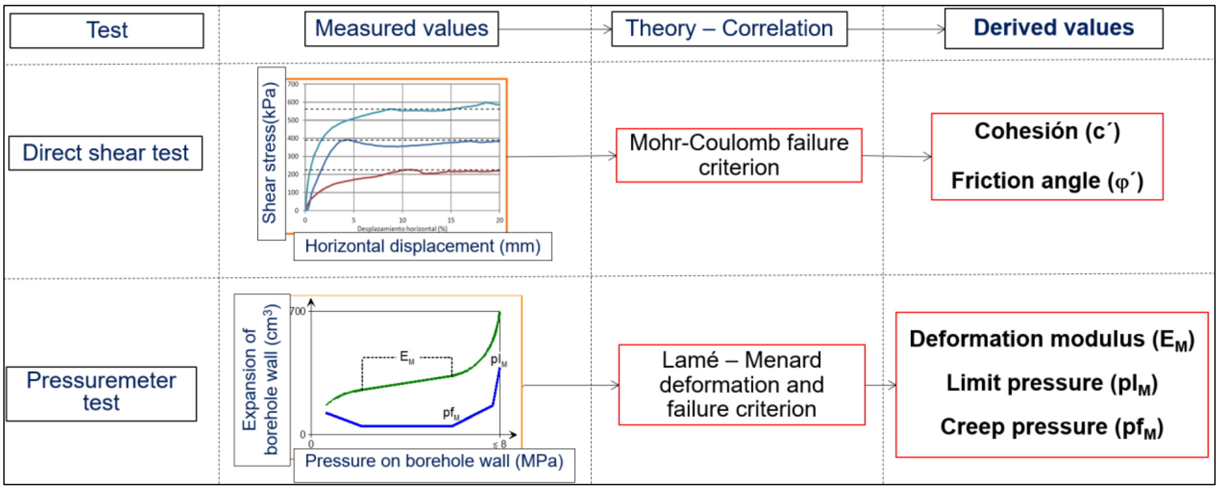
| **Type of value** | **Geotechnical structures and Limit State considered** | | |
|---|---|---|---|
| | **Embankment on soft soils** ***SLS of settlements*** | **Shallow foundation** ***ULS of bearing capacity*** | **Rock slope** ***ULS of stability*** |
| | Ground properties of the soft layer: *Ground deformability and consolidation coefficients* | Ground properties of a particular geotechnical unit in the zone of influence: $c_u$, $c'$ and $\varphi'$ | Ground properties of the joints: $c'$, $\varphi'$, *dip and dip direction* |
| **Historical values** | Test results obtained in previous ground investigations and values used in the design of comparable neighbouring cases | Values used in the design of comparable neighbouring cases | Data of historical failures and inventory of near slopes |
| **Assessed values** | Observations to detect the presence of soft silty soils and peat | Observations to provide information on groundwater levels. | Determination of rock mass quality, rock mass outcrops, zones of degradation, and discontinuities |
| **Measured values** | Results of measurements taken in oedometer, pressuremeter, dilatometer or CPTU (1) tests | Results of measurements taken in triaxial, direct shear or pressuremeter tests | Results from joint shear test, tilt test and point load test |
| **Monitored values** | Results from a trial embankment | Large-scale shallow foundation load tests | Piezometric level taken during service life or joint strength from slope failure |

1 Piezocone penetration tests (CPTU).

*Source: Developed by the authors*

The last step of Task 2 (ground investigation) related to the scope of this document is the use of soil or rock mechanic theories, correlations, or empiricism to obtain derived values from the different values (historical, assessed, measured, and monitored values) obtained in the ground investigation. Using empiricism to obtain derived values means using comparable experience to obtain them, i.e. using experience of similar structures on similar ground subjected to similar loading condition. Figure 4 shows some examples of measured values, and the procedures used to determine the derived values from theses measured values.

**Figure 4.** Examples of measured values in different tests and the procedures used to determine the derived values.

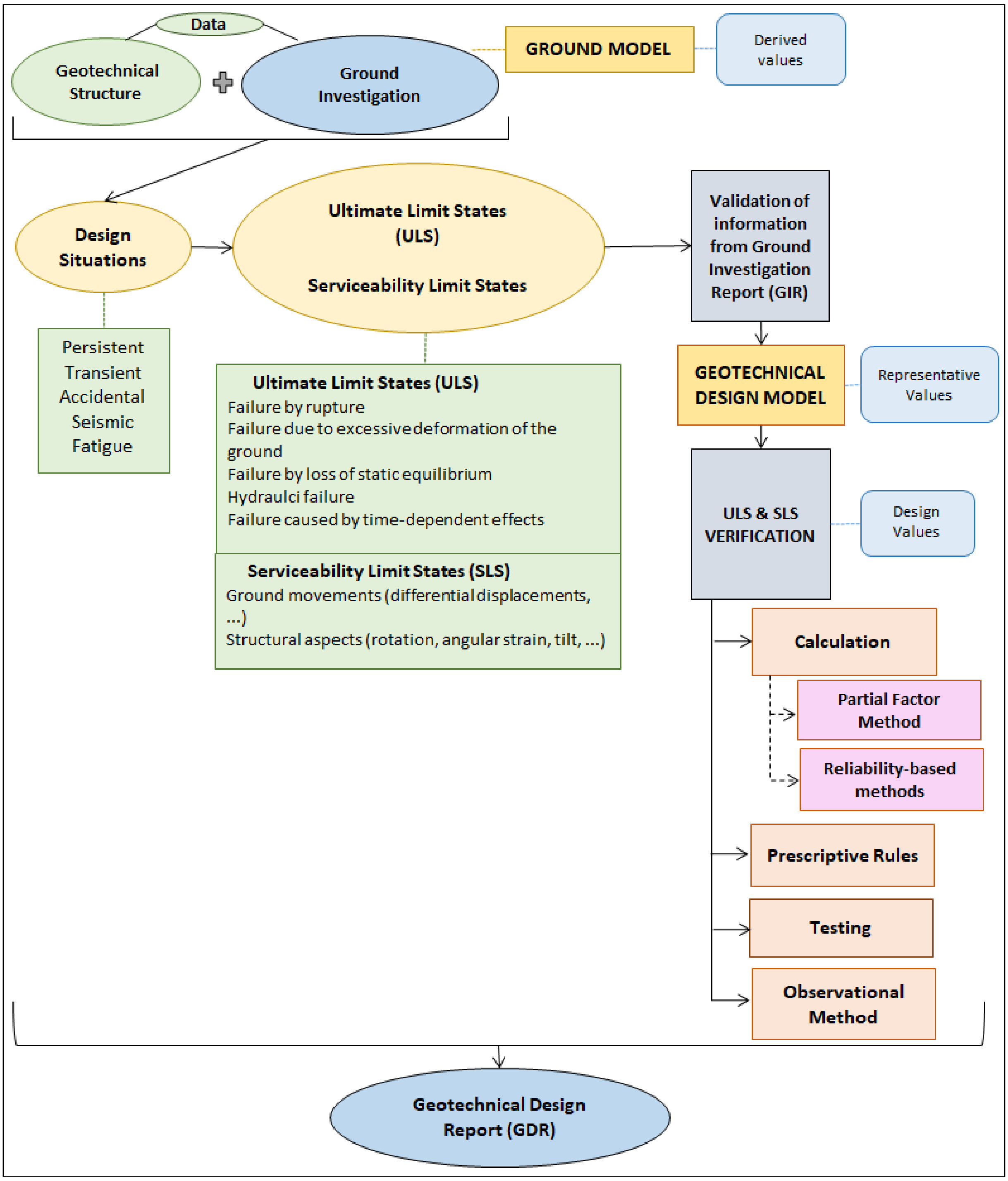



*Source: Developed by the authors*

### 2.2.3 Design verification

The next task in the design process is the Design Verification that should begin with an analysis of the Ground Model and the conditions under which the structure must meet its design requirements, as shown in Figure 5.

**Figure 5.** Design verification flow chart




*Source: Developed by the authors*

The aims of this task are:

— to identify the design situations that, according to EN 1990-1/3.1.2.2, are '*the physical conditions expected to occur during a certain time period ...*' in the structure and to define their associated relevant limit states;

— to validate the information gathered in the Ground Investigation Report, according to the measures given in EN 1997-1/4.2.4 and shown in Figure 6, to check its appropriateness for the considered design situations and associated limit states, and

— to develop the Geotechnical Design Model(s) that, according to EN 1997-1/3.1.6.4 is a '*conceptual representation of the site derived from the Ground Model for the verification of each appropriate design situation and limit state*' performed by the designer for engineering design purposes. To do so, the designer must identify the geotechnical units *('volume of ground that is identified as a single material')* and their ground properties relevant to the limit states under consideration.

**Figure 6.** Measures to validate the information obtained from the GIR (EN 1997-1, Table 4.7).

**Table 4.7 (NDP) — Measures to validate the information obtained from the GIR**

| Geotechnical Category | Measures |
|---|---|
| GC3 | All measures given below for GC2 and GC1 and, in addition:<br>— determine relevant quality[a] parameters based on the available data;<br>— confirm that areas with low confidence in the determined geological, hydrogeological and geotechnical conditions do not have a significant influence on the design and verification of the limit state. |
| GC2 | All measures given below for GC1 and, in addition:<br>— compare the consistency of boundaries of the geotechnical units from different sources of information, to confirm that the used methods of interpolation have sufficiently captured the variations;<br>— compare ground description, classification, and strength index test results to identify inconsistencies;<br>— evaluate the performed testing to ensure that the test results are appropriate for the design situation considered, with respect to e.g. loading rate, strain level, stress path and boundary conditions;<br>— confirm that, for the design situation considered, the derived values have been appropriately determined and correlations used within their respective limitations;<br>— confirm that suitable in-situ techniques and laboratory tests have been used in relation to the design situation, ground property and sample quality class (see FprEN 1997-2:2024, 5.4.5). |
| GC1 | All measures given below:<br>— if field investigation and laboratory testing is performed, confirm that appropriate testing standards have been used;<br>— compare derived values from different sources to identify inconsistencies and anomalies;<br>— confirm that anomalies have been identified in the GIR, and<br>— confirm that non-relevant information is not included in the GIR. |
| [a] Quality parameters include for example sample disturbance, zero-drift for CPT, measurement accuracy. | |

*Source: EN 1997-1/Table 4.7.*

The design situations, which are classified, in EN 1990-1/5.2 (3), as persistent, transient, accidental, seismic, and fatigue, are associated with several relevant Ultimate Limit States (ULSs) and Serviceability Limit States (SLSs) that must be verified.

Verification that limit states are not exceeded by geotechnical structures may be achieve by one or more of the following methods:

— by calculation, applying the partial factor method or any other reliability-based method;

— by using prescriptive rules;

— by testing;

— by the application of the Observational Method.

When checking Ultimate Limit States (ULSs) for a geotechnical structure by the partial factor method, the inequality $E_d \leq R_d$ must be satisfied, where $E_d$ is the design value of the effect of actions and $R_d$ is the design value of the corresponding resistance. For each ULS, the representative and design values of actions, material properties, and ground resistances must be identified and determined.

Design values of geotechnical resistances $R_d$ should be determined using calculation models and applying one of the two factor approaches defined in EN 1990-1:

— the 'Material Factor Approach' (MFA), which applies partial factors to material properties, as shown in Equation 1, or

— the 'Resistance Factor Approach' (RFA), which applies partial factors to resistances, as shown in Equation 2.

**Equation 1.** Design values of geotechnical resistances $R_d$, for MFA calculations [Taken from EN 1990-1/Formula (8.19)].

$$R_d = R\{X_d; a_d; \Sigma F_{Ed}\}$$

**Equation 2.** Design values of geotechnical resistances $R_d$, for RFA calculations [Taken from EN 1990-1/Formula (8.20)].

$$R_d = \frac{R\{X_{rep}; a_d; \Sigma F_{Ed}\}}{\gamma_R}$$

where:

$X_d$ denotes the design values of material properties and equals $X_{rep}/\gamma_M$;

$a_d$ denotes the design values of geometrical properties;

$F_{ED}$ denotes design values of actions used in the assessment of both $E_d$ and $R_d$;

$X_{rep}$ denotes the representative value of a material property;

$\gamma_M$ is a material partial factor, whose values can be found in EN 1997-1/Table 4.8 (Eurocode 7: Geotechnical design - Part 1: General rules) and may be adjusted according to the consequences of failure by the consequence factors $K_M$.

$\gamma_R$ is a resistance partial factor, whose values can be found in EN 1997-3[6], for the different geotechnical structures, and may be adjusted according to the consequences of failure by the consequence factors $K_R$.

Equation 1 and Equation 2 show that, when using the partial factor method, the determination of the representative value of a material property for each geotechnical unit is the first objective of this design stage, since that is the value needed in the calculation models used to verify the different limit states for each of the relevant design situations of the geotechnical structure.

### 2.2.4 Design implementation during execution

The design, execution, and maintenance of the geotechnical structure during its subsequent service life must be part of the same process so they require continuity in its development, as indicated by EN 1997-1. To assure this, the project itself must contain the following plans: 'Supervision', 'Inspection', 'Monitoring and Maintenance Plans', which must be complied with during the execution of the works.

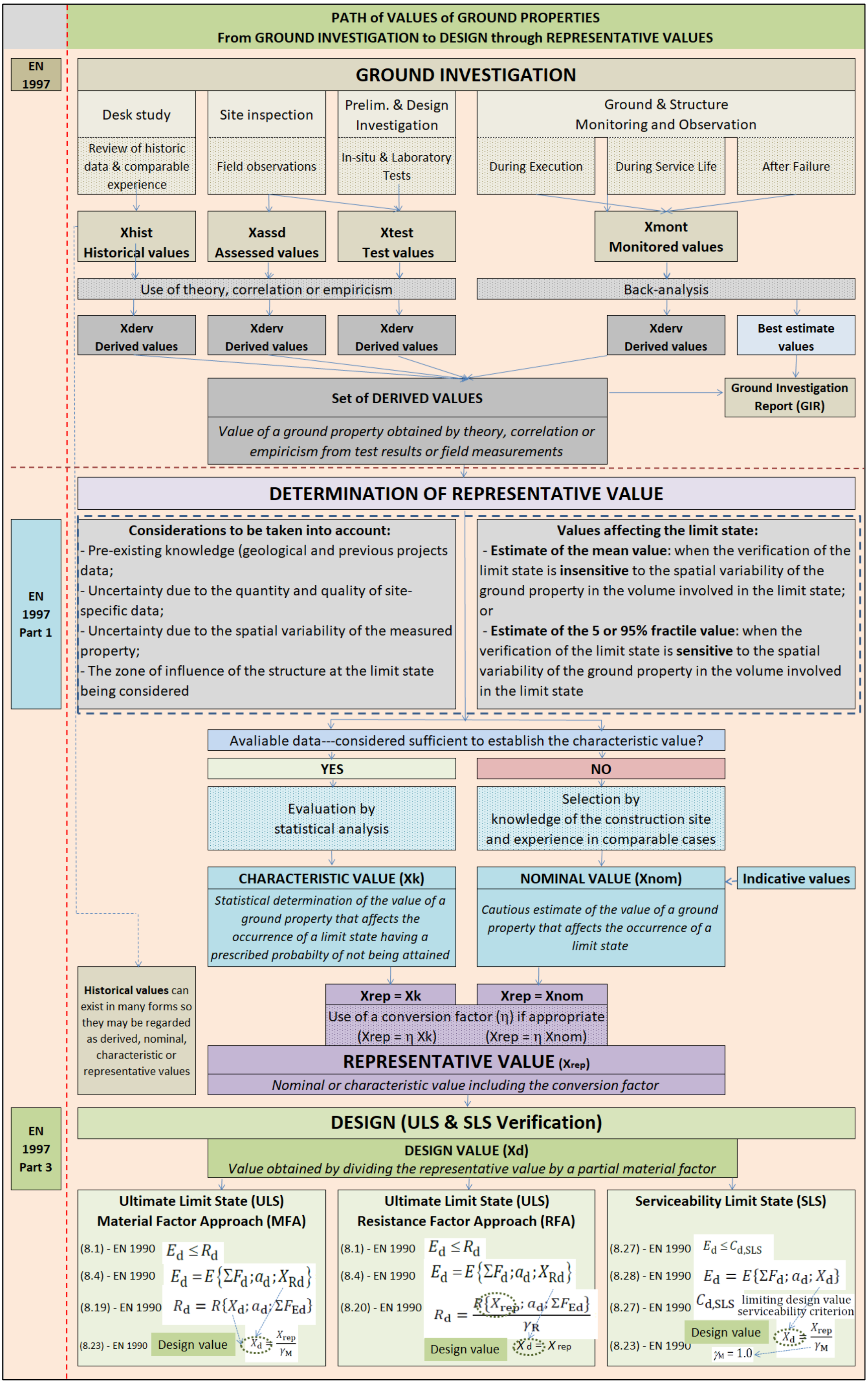
It should be noted that the level of complexity and detail of these plans is related to the Geotechnical Category (GC) of the structure. This level of complexity necessitates a certain number of supervision and inspection actions; a certain amount of in-situ measurements and tests to be carried out; and the specification of a set of required maintenance tasks.

## 2.3 From derived to representative values

Figure 7 shows the path that a value must follow from its collection during the ground investigation to its use in a calculation model to verify a limit state, either ULS or SLS.

[6] Eurocode EN 1997-3: Geotechnical design - Part 3: Geotechnical structures. EN 1997-3 establishes principles and requirements for the design and verification of the following geotechnical structures: slopes, cuttings, embankments, shallow foundations, piled foundations, retaining structures, reinforced fill structures, soil nailed structures and ground improvement and the following supporting elements: anchors, reinforcing elements in reinforced fill structures, soil nails, rock bolts and rock surface support, and ground improvement. More information: https://eurocodes.jrc.ec.europa.eu/EN-Eurocodes/eurocode-basis-structural-design

**Figure 7.** Path of values: From Ground to Geotechnical Design Model.




*Source: Developed by the authors.*

Figure 7 shows that there are two possible ways to determine the representative value of a ground property ($X_{\text{rep}}$):

— the 'nominal value path': in which the designer selects the value by judgement and experience gained in similar cases; or

— the 'characteristic value path': in which the designer uses statistical procedures as a tool to determine the value.

In both ways, in order to fulfil the reliability requirements relating to the determination of the representative value, the designer must take into account the following aspects:

— pre-existing knowledge including geological information and data from previous projects that is gathered in the desk study,

— uncertainty due to the quantity and quality of site-specific data,

— uncertainty due to the spatial variability of the measured property, and

— the zone of influence of the structure at the limit state being considered.

When selecting the value by judgement and experience in similar cases, the designer will make *'a cautious estimate of the value of the ground property'*. When doing so, the value obtained must be termed a 'nominal value'. That cautious estimate should be made considering the aspects mentioned above.

On the other hand, when evaluating the value by statistical procedures, as explained in Chapter 4 of this document, the value must be termed a 'characteristic value', aligned with the definition given in EN 1990-1/3.1.4.1 that relates the characteristic value of a material property with a value '*having a prescribed probability of not being attained in a hypothetical test series. This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product…')*.

The last step of this procedure is to obtain the representative value from either the characteristic value or the nominal value, according to one of the Equation 3 or Equation 4:

**Equation 3.** Determination of representative values as characteristic value [Taken from EN 1997-1/Formula (4.1)].

$$X_{rep} = X_k$$

**Equation 4.** Determination of representative values as nominal value [Taken from EN 1997-1/Formula (4.2)].

$$X_{\text{rep}} = X_{\text{nom}}$$

When the designer needs to take into account effects, among others, of scale, moisture, temperature, ageing of materials, anisotropy, stress path or strain level, a conversion factor ($\eta$) may be used to obtain the representative value, according to one of the following Equation 5 or Equation 6:

**Equation 5.** Determination of representative values as characteristic value, including the conversion factor [Taken from EN 1997-1/Formula (4.3)]

$$X_{\text{rep}} = \eta X_{\text{k}}$$

**Equation 6.** Determination of representative values as nominal value, including the conversion factor [Taken from EN 1997-1/Formula (4.4)]

$$X_{\text{rep}} = \eta X_{\text{nom}}$$

This procedure to determine the representative values of ground properties is valid for verifications of both ULS and SLS.

## 2.4 From Ground Model to Geotechnical Design Model

### 2.4.1 General

Section 2.2 explained the path that a set of derived values, obtained in the ground investigation, must follow to become a representative value to be used in a calculation model to verify a limit state, either ULS or SLS.

As said in EN 1997-2/4.1 (5) '*the Ground Model shall reference the derived values …*', while '*the Geotechnical Design Model should include representative values* …', as stated in EN 1997-1/12.3.2 (2).

So, taking into account the relation between, on one hand, the derived values and the Ground Model (GM) and, on the other hand, the representative values and the Geotechnical Design Model (GDR), this section gives some ideas about the transition from GM to GDM.

### 2.4.2 Ground Model

According to EN 1997-2/4.1(1), '*A Ground Model shall comprise the geological, hydrogeological, and geotechnical conditions at the site, based on the ground investigation results*'. Furthermore, the '*Ground Model and the Ground Investigation Report are the main output of the Ground Investigation and …'* (EN 1997-2/4.1(4) Note) so it '*should be documented in the Ground Investigation Report (GIR)*', but also '*may be documented in the Geotechnical Design Report*'. See European Commission: Joint Research Centre, Garin, Baldwin et al., 2024 for more guidance on this topic.

In the context of this document, it is important to highlight that the geotechnical conditions, mentioned above, refer mainly to the '*mechanical behaviour of the ground described by the properties of the geotechnical units'* (EN1997-2/4.1(1) Note 3), quantified by the derived values *'established from data gathered during the desk study, site inspection, preliminary and design investigations, and monitoring of the ground and structures'* (EN1997-2/4.2(1)).

### 2.4.3 The transition from the Ground Model to the Geotechnical Design Model

The Ground Investigation Report (GIR) can be considered as an instrument to transmit the information obtained during the ground investigation to the designer of the geotechnical structure. The validation measures shown in Figure 6 are guidelines to validate the information presented in the GIR and they may also be used for appropriately preparing the GIR. This means the GIR is a key part of the design process. This validation procedure provides a link between the preparation of the GIR and the design of the geotechnical structure.

### 2.4.4 The Geotechnical Design Model

According to EN 1997-2/4.1 (4) Note, *'the Ground Model and the GIR…form the basis for the development of the Geotechnical Design Model (GDM)'.* The GDM is developed for the verification of each relevant geotechnical design situation, with corresponding combinations of actions and associated relevant limit states.

The development of the GDM can begin once the designers of the geotechnical structure have validated the information obtained from the GIR. In the context of this document, the first stage is to identify the various geotechnical units involved in the verification of the limit state. Note that a geotechnical unit is defined as a '*volume of ground that is defined as a single material*' (EN 1997-1/3.1.6.5); this implies that, in the following step of the design process, the designer must determine the representative values of the ground properties for each geotechnical unit that is relevant to the design situation and limit state under verification. All the representative values must be included in the Geotechnical Design Report.

As a summary, Table 4 shows the main differences between the Ground Model and the Geotechnical Design Model regarding different topics.

**Table 4.** Differences between the Ground Model and the Geotechnical Design Model.

| **Topic** | **Ground Model** | **Geotechnical Design Model** |
|---|---|---|
| **Developed in EN 1997** | Part 2 | Part 1 |
| **Data to support the model** | Results from ground investigation | Data in Ground Investigation Report (GIR) and Ground Model |
| **Property values included** | Derived values | Representative and design values |
| **Reported in** | Ground Investigation Report (GIR) or Geotechnical Design Report (GDR) [in EN 1997-2/4.1 (6 & 7)] | Geotechnical Design Report (GDR) [in EN 1997-1/12.3.2(2)] |

*Source: Developed by the authors*

Finally, as previously stated, to assure the success of the design it is very important that good communication exists between those involved in each stage of the design process. Communication can be supported by verbal or other means but at the completion of a stage, it should be in formal report form that can be passed on. Reports need to be clear identifying what are facts, opinions, and uncertainties. The greater the number of parties involved from stage to stage (e.g. preparation of: preliminary desk study; ground investigation specification; ground investigation factual data; GIR; GDR; execution documentation; works method statements etc.) the greater the risk of missed or misunderstood communication and hence the requirement that, at all stages, documentation is clear. The individuals who plan, execute, and evaluate the ground investigation perform a key function in the design process and should have professional design expertise appropriate to the geotechnical complexity class, just as those who carry out the subsequent geotechnical design (focused, in this stage, on limit state verification). On the other way round, the individuals who design and verify geotechnical structures should also have ground modelling expertise.

# 3 Types of values

## 3.1 Introduction

The aim of this chapter is to highlight the various types of values that must be described and determined prior to performing a geotechnical design:

- Values related to actions,
- Values related to ground properties, and
- Values related to geometrical properties.

Additionally, due to its relevance in geotechnical design, another section is devoted to the groundwater conditions that need to be determined. It is anticipated that some of those groundwater conditions can be considered as actions and others, as material or geometrical properties.

## 3.2 Values related to actions

According to EN 1990-1/6.1.1, actions must be classified by their variation in time, as follows:

- Permanent ($G$), or
- Variable ($Q$), or
- Accidental ($A$)or
- Seismic ($A_E$).

Additionally, actions may be also classified by their origin (as direct or indirect), spatial variation (as fixed or free) and the structural response (as static or dynamic).

In the geotechnical field, according to EN 1997-1/4.3.1.2, the following potential permanent and variable actions should be included in relevant geotechnical design situations:

a) the weight of the ground and groundwater,

b) in situ ground stresses and pressures,

c) groundwater pressures,

d) ground movements and pressures arising from loads or through other structural elements,

e) ground movements and pressures caused by pre-existing stresses in the ground or changes thereof,

f) ground movements and pressures caused by environmental influences,

g) stress and stress changes due to construction and operation, and

h) anticipated future structures.

Actions must be represented by their representative values. According to EN 1990-1/6.1.2, '*the principal representative value of an action ($F_{rep}$) should be its characteristic value ($F_k$)*', that must be chosen according to the methods given in EN 1991 and the relevant part of EN 1997. This characteristic value can be a mean value, an upper or lower value, or a nominal value.

Specifically for permanent actions, as stated in EN 1990-1/6.1.2.2, the representative value of a permanent action ($G_{rep}$) shall be taken as its characteristic value ($G_k$), that may be taken either as the mean value of G, if its coefficient of variation is small (5% or 10%, according to the limit state involved), or, otherwise, as upper ($G_{k,sup}$) and lower ($G_{k,inf}$) characteristic values.

On other hand, as stated in EN 1990-1/6.1.2.3, the representative value of a variable action ($Q_{rep}$) shall be taken as one of the following, depending on the limit state being verified:

— Characteristic Value ($Q_k$): that must correspond to one of the following:

  - An upper value with an intended probability of not being exceeded during a specific period,
  - A lower value with an intended probability of being exceeded during a specific reference period,
  - A nominal value when the statistical distribution of variable action is not known.

— Combination value ($Q_{comb}$),

— Frequent value ($Q_{freq}$) or

— Quasi-permanent value ($Q_{qper}$).

The combination, frequent and quasi-permanent values should be determined by multiplying the characteristic values ($Q_k$) by combination factors ($\psi$) which are given by EN 1990-1/Annex A.

Lastly, the design value of a permanent action $G_d$ that produces an unfavourable effect should be calculated using Equation 7, while if it produces a favourable effect, its design value should be calculated using Equation 8.

**Equation 7.** Permanent action $G_d$ that produces an unfavourable effect [Taken from EN 1990-1/Formula (8.6)]

$$G_d = \gamma_G G_k$$

**Equation 8.** Permanent action $G_d$ that produces a favourable effect [Taken from EN 1990-1/Formula (8.7)]

$$G_{d,fav} = \gamma_{G,fav} G_k$$

where $\gamma_G$ is a partial factor for permanent actions (specified in EN 1990-1/Annex A), $G_k$ is the characteristic value of a permanent action and $\gamma_{G,fav}$ is a partial factor specified in EN 1990-1/Annex A.

The design value of a variable action $Q_d$ that produces an unfavourable effect should be calculated using Equation 9.

**Equation 9.** Variable action $G_d$ that produces an unfavourable effect [Taken from EN 1990-1/Formula (8.10)]

$$Q_d = \gamma_Q G_{rep}$$

where $\gamma_Q$ is a partial factor for variable actions (specified in EN 1990-1/Annex A) and $G_{rep}$ is the representative value of a variable action.

## 3.3 Values related to ground properties

Ground properties are described in EN 1997-2, as follows:

- State, physical and chemical properties, in Clause 7;
- Strength properties, in Clause 8;
- Stiffness, compressibility and consolidation properties, in Clause 9;
- Cyclic, dynamic and seismic properties, in Clause 10;
- Groundwater and geohydraulic properties, in Clause 11;
- Thermal properties of the ground, in Clause 12.

Ground properties are characterized by different distributions and variability. The most common distributions are Gaussian normal and lognormal. Uncertainties are generally indicated by means of the coefficient of variation (V). The coefficient of variation is a statistical measure of the dispersion of data points in a data series around the mean. The coefficient of variation represents the ratio of the standard deviation ($\sigma$) to the mean value ($X_{mean}$) by the expression: $V = \sigma/X_{mean}$. In the literature, V is often referred to as COV (e.g. Phoon and Kulhawy 1999).

Both global (worldwide) and local (national or regional) databases of ground properties are available in the literature. Those include measured properties from laboratory or field tests and transformation models defining relationships between properties (e.g. Ching and Phoon, 2014). Additionally, site-specific databases are available that may be associated with specific projects or test sites. However, uncertainties of ground properties observed from these databases can vary significantly. This can be due to the following factors:

- Geographical coverage (global vs local);
- Geological conditions, such as depositional and stress histories;
- Test types;
- Specific ground property (e.g. friction angle vs undrained shear strength)
- Sample size;
- Sample quality;
- Effects of scale, moisture, temperature, material ageing, anisotropy, stress path;
- Strain level;
- Measurement error;
- Reliability of the correlation used;
- Consequence class.

Chapters 4 and 5 of this document provide some guidance on how to take account of uncertainties due to these factors when determining and selecting representative values of ground properties for use in design calculations.

## 3.4 Values related to geometrical properties

From a geotechnical point of view, as stated in EN 1997-1/4.3.3 (2), the following items must be regarded as geometrical properties:

— Ground surface

— Surface water level

— Groundwater levels

— Boundaries between geotechnical units

— Dimensions of geotechnical structures

Ground surface and boundaries between geotechnical units are dealt with in the second Guideline of this series (European Commission: Joint Research Centre, Garin, Baldwin et al., 2024) while representative values of groundwater and surface levels are dealt in Section 3.5 of this document. The dimensions of geotechnical structures are set in the projects, and they are out of the scope of this document.

The key role of discontinuities in geotechnical design should be noted. For this specific issue, according to EN 1997-1/4.3.3 (3), '*the geometrical properties of discontinuities in the ground shall include information on location, orientation, spacing, extent, voids or openings, and surface roughness*'. Furthermore, geometrical properties of discontinuities within a geotechnical unit may be considered either as properties of discretely defined discontinuities within the unit or as equivalent ground properties of the unit when modelled as a continuum.

On other hand, according to EN 1990-1/8.3.7 (2), 'when the design of the structure is not significantly sensitive to deviations in a geometrical property, the design value of a geometrical parameter $a_d$ may be calculated' as the nominal value. However, when the design of the structure is sensitive to deviations in a geometrical property, the design value of the geometrical property, $a_d$ should be calculated as the sum of the nominal value ($a_{nom}$) and the deviation ($\Delta a$) in the geometrical property.

It is worth noting that, according to EN 1990-1/6.3 (3), 'when there is sufficient data, the characteristic value of a geometrical property may be determined from its statistical distribution and used instead of a nominal value'. Besides that, according to EN 1997, 4.3.3.(6) 'the nominal value of geometrical properties for ground discontinuities may be determined by sensitivity analysis using a probabilistic approach'.

## 3.5 Values related to hydrogeological and groundwater conditions

### 3.5.1 General

Due to its relevance in geotechnical design, this section describes the values related to the hydrogeological and groundwater conditions. In this respect, according to EN 1997-2/5.2.5, and including the absolute permeability of the ground and the groundwater density, groundwater conditions that need to be investigated and considered in the design are shown in Table 5.

**Table 5.** Groundwater design conditions.

| Design topic | Type of value |
|---|---|
| Depth, thickness, and extent of water-bearing geotechnical units | Geometrical property |
| Surface, groundwater levels and their variation over time | Geometrical property |
| Piezometric levels and their variation over time | Geometrical property or action (see 3.5.4) |
| Groundwater pressure (u) distribution | Action |
| Hydraulic conductivity (K) of geotechnical units and its possible anisotropy | Ground property |
| Absolute permeability of ground (k) | Ground property |
| Water weight density ($\gamma_w$) | Groundwater property |
| Chemical composition and temperature of groundwater | Groundwater property |

*Source: Developed by the authors.*

### 3.5.2 Groundwater actions: groundwater pressures

In the same way as other actions are handled, the characterization of water actions depends, according to EN 1990-1/6.1.3.2, on whether they are classified as permanent, variable, or accidental.

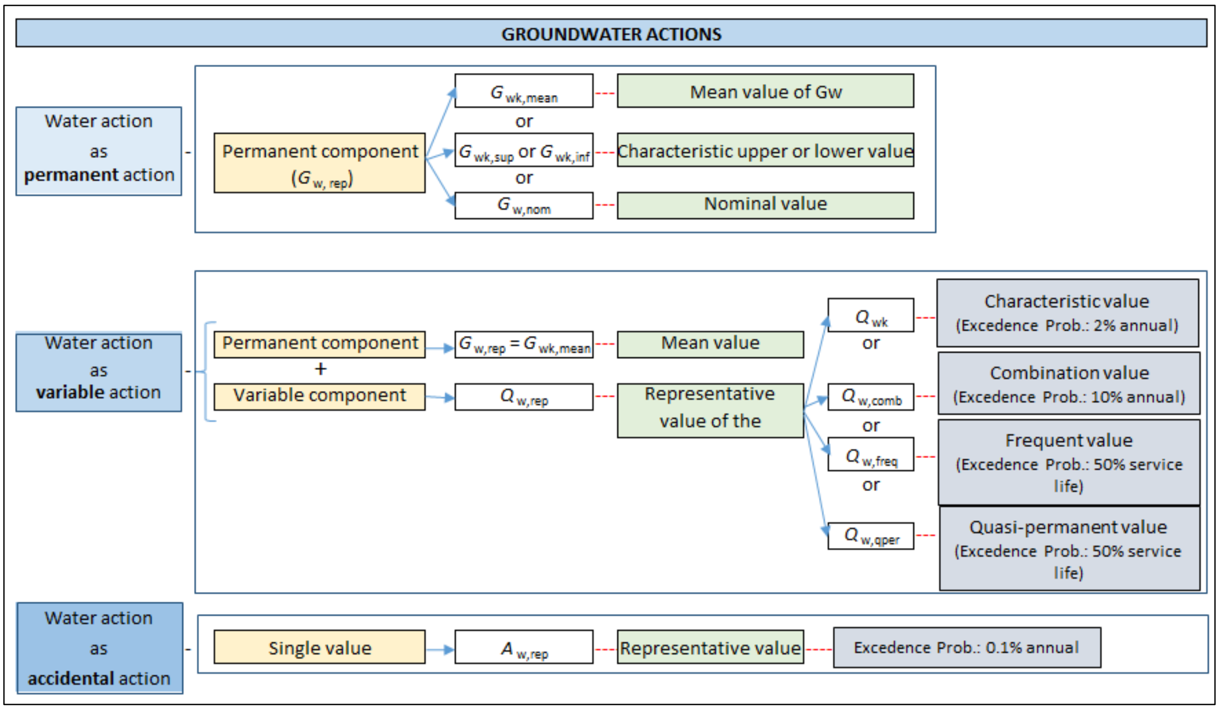
When the variation in magnitude of the water action is small or monotonic throughout the design service life, it should be classified as permanent. Its representative value ($G_{w,rep}$) is then given by one of three possible values, as illustrated in the top part of Figure 8:

— a single characteristic value $G_{wk}$ equal to the mean value of $G_w$ ($G_{w,rep} = G_{wk,mean}$)

— either the upper or lower characteristic value, $G_{wk,sup}$ or $G_{wk,inf}$, whichever is more onerous

— a nominal value $G_{w,nom}$: this is often used in geotechnical design when there is sparse information about groundwater levels (and hence pressures).

**Figure 8.** Determination of the representative value of water actions.




*Source: Developed by the authors.*

Alternatively, when the variation in magnitude of the water action is neither negligible nor monotonic, it should be classified as a variable action whose representative value $F_{w,rep}$ is made up of two components, as shown in the bottom part of Figure 8:

— a permanent component $G_{w,rep}$ taken as the mean value of $G_w$ ($G_{wk,mean}$), i.e. the first option above

— a variable component $Q_{w,rep}$ that represents the variation in water action from the mean.

The magnitude of the variable component $Q_{w,rep}$ depends on which combination of actions is appropriate for the design situation being considered and can be any of the alternatives given in Table 6. On other hand, when the water action is of significant magnitude and typically of short duration but unlikely to occur during the design service life, it should be classified as an accidental action whose representative value $F_{w,rep}$ (denoted $A_{w,wep}$) should have a probability of exceedance of 0.1% per annum (return period 1 000 years).

**Table 6.** Specification of water actions, according to EN 1990-1.

| **Variable or accidental water action** | **Symbol** | **Probability of exceedance (in EN 1990-1,6.1.3.2)** |
|---|---|---|
| Characteristic | $Q_{wk}$ | 2% per annum (return period 50 years) |
| Combination | $Q_{w,comb}$ | 10% per annum (return period 10 years) |
| Frequent | $Q_{w,freq}$ | Fraction of time exceeded = 1% |
| Quasi-permanent | $Q_{w,qper}$ | Fraction of time exceeded = 50% |
| Accidental | $A_{w,rep}$ | 0.1% per annum (return period 1 000 years) |

*Source: Developed by the authors, based on data from EN 1990-1.*

EN 1997-1/6.4 outlines the procedure to select representative values of groundwater pressures, while EN 1997-1/6.5.1, states that '*design values of groundwater pressures in ultimate limit states shall be determined by one of the following methods:*

*i) direct assessment or*
*ii) applying a deviation to the representative piezometric level or to the representative groundwater pressure or*
*iii) applying a partial factor to the representative groundwater pressures or to their action effects.*

### 3.5.3 Groundwater properties

Hydraulic conductivity of ground is a property with significant amplitude of variation (i.e. several orders of magnitude). When dealing with relatively small ground models, so that the zone of influence of the limit state has a very limited extent, (e.g. a construction site as opposed to a city-wide model) the size of the model may be influenced by extreme local variations. The influence of these variations is called the '*nugget effect'* and reflects the variability seen between closely spaced samples. Therefore, to reduce the uncertainties related to that effect and ground anisotropy when assessing the hydraulic conductivity, in situ testing is preferred to laboratory testing for better representativeness of the tested ground volume.

Groundwater chemistry may present variations that usually are the result of the types of anthropic activities carried out in the vicinity of the study area.

### 3.5.4 Geometrical properties related to groundwater

As noted before, surface water and groundwater levels are considered geometric properties whose determination is dealt with in EN 1997-1, Clause 6 'Groundwater'. However, it is not clearly stated in EN 1997-1 or EN 1997-2 if the piezometric level can be considered as a geometrical property and to discuss this issue is out of the scope of this document. The definitions included in EN 1997-1/3.1.8.2, 3 & 4 does not give any valuable information in this respect: groundwater level ('*level of the water surface in the ground*'); piezometric level ('*level to which water would rise in a standpipe designed to detect the pressure of water at a point beneath the ground surface*') and surface level ('*level of water above the ground surface*').

Nevertheless, it is clearly stated that groundwater pressure (*u*), considered as an action, must be determined from the piezometric level using EN 1997-1/Formula 6.1 (see below), so groundwater actions and piezometric levels are mathematically connected, as shown in Equation 10.

**Equation 10.** Groundwater pressure (u) determined from the piezometric level [Taken from EN 1997-1/Formula (6.1)]

$$u = \gamma_w (h_{wz} - z)$$

where $\gamma_w$ is the weight density of the pore water, $h_{wz}$ is the piezometric level, i.e. water pressure head, at elevation *z*, i.e. depth, and *z* is the elevation where u is measured (positive upwards).

## 3.6 Values to be used in Limit State verifications

The design values for ground properties for Ultimate Limit States are to be obtained by applying partial factors, $\gamma_M$, to representative values, when using the partial factor method. The representative value is usually divided by the partial factor, however multiplication may be used instead, when a superior value of ground property is unfavourable. The values of the partial factors depend on the ground property, on the timescale of the design situation (i.e. persistent, transient, and accidental design situations) and on the consequence class ($K_M$).

When verifying Serviceability Limit States, the design values of ground properties are to be determined using a partial factor $\gamma_M$=1.0.

# 4 Determination and selection of ground property values

## 4.1 Overview of ground property values

This chapter provides guidance on the determination and selection of the ground property values involved in a geotechnical design calculation. An important feature of ground, as an engineering material, which affects geotechnical design and causes EN 1997 to differ from the Eurocodes for other materials involved in structural design, is that ground is a natural material, unlike the materials used in structural design, such as concrete and steel, which are manufactured. One consequence of this is that the values of the properties of the ground need to be determined as part of the geotechnical design process rather than being specified by the designer, as in the case of manufactured materials in structural design.

For this reason, EN 1997-1: *General rules* provides definitions for the different ground property values used in geotechnical design and the principles for obtaining representative and design values of ground properties from derived values via nominal and characteristic values.

Correspondingly, EN 1997-2: *Ground properties* gives guidance on the ground investigation activities to obtain derived values of ground properties, with special focus on the selection of field investigation and laboratory test methods. EN 1997-3: *Geotechnical structures* specifies how the representative and design ground property values shall be used in geotechnical design verifications. The path of different ground property values involved in going from derived values to design values for use in geotechnical design calculations is given in Section 2.1 of this document and shown in Figure 7.

Looking to the past, it should be highlighted that the inclusion in EN 1997-1:2004 of a definition for the value of a ground property value, i.e. the representative or characteristic value, to be used in a geotechnical design verification was an important and innovative feature. Such a definition for the value of ground property was not given in geotechnical design codes before the first generation of Eurocode 7 (EN 1997-1:2004). The choice of the ground property value to be used in a geotechnical design verification was left to the designer to assess, based on the available geotechnical information, the design situation and their experience.

## 4.2 Measured and derived values

### 4.2.1 Measured values

The measured value of a ground property is defined in EN 1997-2/3.1.2.2 as '*the value of a ground property recorded during a test*'. Guidance on the determination of measured ground property values from field and laboratory tests is given in European Commission: Joint Research Centre, Garin, Baldwin et al. 2024.

### 4.2.2 Derived values

The derived value of a ground property is defined in EN 1997-1/3.1.3.2. This value is '*obtained by theory, correlation or empiricism from test results or field measurements*', as illustrated in Figure 8. Derived values are used to establish the Ground Model, which, according to EN 1997-2/4.1(1), '*shall comprise the geological, hydrogeological, and geotechnical conditions at the site, based on the ground investigation results*' while 4.1(5) requires that '*The Ground Model shall reference the* *derived values of ground properties for encountered geotechnical units*'. According to EN 1997-1/3.1.6.5, *a geotechnical unit is 'the volume of ground that is defined as a single material'.*

In this respect, the key words in the definition are 'single material'. The interpretation of such words should be that a 'single material' is a material whose properties can be defined with a 'single' set of values. These values can be a number (for instance, the friction angle) or a trend line or a trend curve (for instance, a trend line with depth), including also anisotropy whether it be in strength, stiffness or permeability.

The procedure for the determination of derived values from test results is described in the second Guideline of this series (European Commission: Joint Research Centre, Garin, Baldwin et al., 2024). The subsequent assessment of the derived values is an iterative process that should involve three steps:

- The first step is to eliminate all derived values that are recognised and identified as being gross errors generated by measurement or other errors. This information should be found in the Ground Investigation Report.
- The second step is to identify the geotechnical units in the investigation site in the zone of influence relevant to the design situation.
- The third step is to apply adjustments or corrections to modify the derived values to account for differences between the conditions pertaining during the determination of the derived values and the design situation, for example to account for scale effects as noted in EN 1997-2/4.2(6), which recommends that '*Derived values of ground mass properties that are determined from test results on samples should be adjusted for scale effects*'.

As noted in first bullet point, the first step is more relevant to the person preparing the GIR. It is appropriate that the second two steps are carried out by the person selecting the representative values and preparing the GDR as they involve knowing what the design situation is, which information the person preparing the GIR may not have.

Further information about the determination of derived ground property values is given in European Commission: Joint Research Centre, Garin, Baldwin et al., 2024. It is important to note that a derived property value represents the actual value of a ground property at a specific location in the ground and within a particular geotechnical unit. This value is obtained from a measured valued in a particular test, establishing a direct relationship between the measured and derived values.

## 4.3 Representative value

### 4.3.1 Concept of a representative value

The representative value of a ground property ($X_{rep}$) is defined in EN 1997-1/3.1.3.5 as a '*nominal or characteristic value including the conversion factor*'. Guidance on the selection of representative values is given in EN 1997-1/4.3.2.1 that states that '*Representative values of ground properties to be used in ultimate and serviceability limit state verifications shall be determined from derived values presented in the GIR (Ground Investigation Report)*'. A note to this paragraph states that '*The representative value refers to a particular ground property of a single geotechnical unit*'. As one of the measures to validate the Geotechnical Design Model, it is stated in EN 1997-1/Table 4.6 that derived values from different sources within each geotechnical unit should be compared in order '*to determine representative values of ground properties with an appropriate level of confidence*'.

Guidance on the application of the conversion factor ($\eta$) to account for effects, among others, of scale, moisture, temperature, ageing of materials, anisotropy, stress path or strain level referred to in EN 1997-1/3.1.3.5, is given in Section 4.3.2 of this document.

According to EN 1997-1/4.3.2.1(3), 'The representative value of a ground property shall be determined for each limit state, according to its sensitivity to spatial variability of the ground property in the volume of ground involved'. In 4.3.2.1(4) it is stated that '*If the limit state is insensitive to spatial variability of the ground, the representative value of the ground property shall be determined as an average value*', while in 4.3.2.1(5) it is stated that '*If the limit state is sensitive to spatial variability of the ground, the representative value of the ground property shall be determined as an inferior or superior value*'. The significance of the sensitivity of the limit state to the spatial variability of the ground is discussed in Section 4.3.3 of this document.

According to EN 1997-1/4.3.2.1(6) '*when available data are considered sufficient to establish the characteristic value, the representative value of a ground property ($X_{rep}$), should be determined from Equation 11*'.

**Equation 11.** Determination of representative values as characteristic value [Taken from EN 1997-1/Formula (4.1)].

$$X_{\mathrm{rep}} = X_{\mathrm{k}}$$

where:

$X_k$ is the characteristic value of the ground property (see Section 4.5 of this document).

According to 4.3.2.1(7), '*when the available data are insufficient to establish the characteristic value of a ground property, the representative value should be determined from Equation 12*'.

**Equation 12.** Determination of representative values as nominal value [Taken from EN 1997-1/Formula (4.2)].

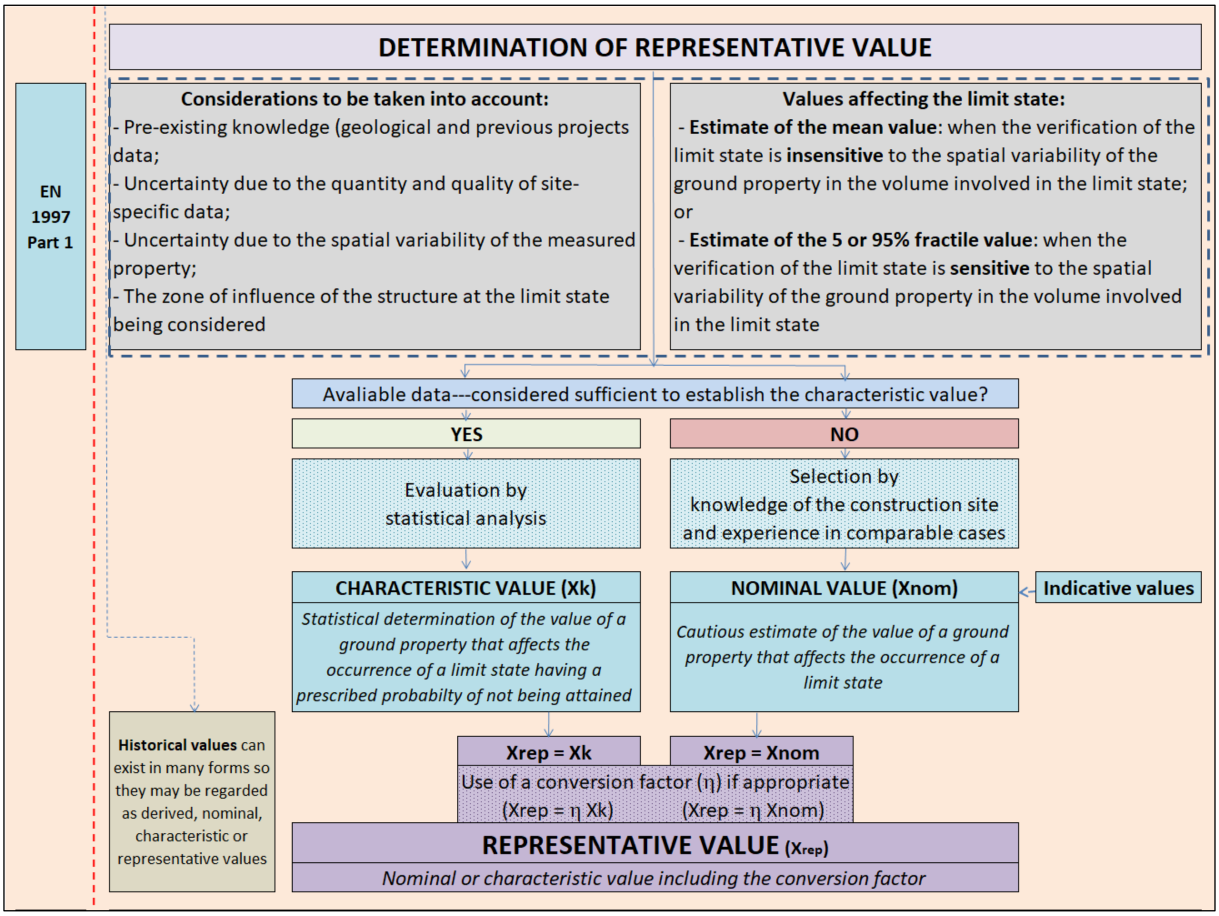
$$X_{\mathrm{rep}} = X_{\mathrm{nom}}$$

where:

$X_{nom}$ is the nominal value of the ground property (see Section 4.4 of this document).

The designer needs to decide whether to determine the representative value as a characteristic or a nominal value, based on their consideration of the sufficiency of the available data, i.e. its quantity, quality, spatial distribution and extent of zone of influence, and also the need to avail of and take into account pre-existing knowledge in the form of comparable experience. However, as ground investigation methods develop and more data become available, it is anticipated that, in the future, there will be a tendency for an increased use of statistics so the representative value will be determined as a characteristic value. Nationally determined practices may also have an influence on whether the representative value is determined as a characteristic or a nominal value. Figure 9 shows a summary of the process for determining the representative value.

**Figure 9.** Process for determining the representative value.




*Source: Developed by the authors.*

### 4.3.2 Conversion factor ($\eta$)

EN 1997-1/4.3.2.1(8) states that, 'when appropriate, a conversion factor to account for effects, among others, of scale, moisture, temperature, ageing of materials, anisotropy, stress path or strain level may be used to obtain the representative value of a ground material property'. In such cases, Equation 11 and Equation 12 are transformed into Equation 13 and Equation 14 respectively.

**Equation 13.** Determination of representative values as characteristic value, including the conversion factor [Taken from EN 1997-1/Formula (4.3)]

$$X_{\mathrm{rep}} = \eta X_{\mathrm{k}}$$

**Equation 14.** Determination of representative values as nominal value, including the conversion factor [Taken from EN 1997-1/Formula (4.4)]

$$X_{\mathrm{rep}} = \eta X_{\mathrm{nom}}$$

According to the Note to EN 1997-1/4.3.2.1(8), '*the value of $\eta$ is 1.0 for cases where effects of scale, moisture, temperature, ageing of materials, anisotropy, stress path and strain level are already included in selecting the nominal or determining the characteristic value.*'

In all cases, traceability of correction or conversion factors applied during the process of moving from measured to representative values must be maintained and recorded. While this is the usual situation, the fact that EN 1997-1 introduces $\eta$ as a correction value is illustrative of the fact that designs must fully account for the situation in practice where conditions determined during the ground investigation may not be representative of the design conditions. Hence, there is a need to take account of possible differences between the ground properties and geotechnical parameters obtained from the test results and those governing the behaviour of the geotechnical structure. Two factors that may also cause these differences are the structure of soil and rock masses and brittleness.

A simple example of when a test result may not be representative of the design conditions is given by the undrained shear strength of a clay layer at the base of an excavation after the excavation, which will be lower due to swelling than the value measured during the ground investigation prior to the excavation. There is no single value of $\eta$ to be applied, as it will vary with depth in such a situation. The inclusion of $\eta$ can be considered to be indicative of the need to take account of the effects of scale, moisture, temperature, ageing of materials, anisotropy, stress path and strain level, listed in EN 1997-1/4.3.2.1(8), when determining the representative value and to allow for possible change in the magnitude of the representative value due to those effects.

### 4.3.3 Considerations to be taken into account when determining the representative value

According to EN 1997-1/4.3.2.1(2), the considerations that need to be taken into account when determining the representative value of a ground property in a single geotechnical unit are the following:

- pre-existing knowledge including geological information and data from previous projects;
- uncertainty due to the quantity and quality of site-specific data;
- uncertainty due to the spatial variability of the measured property; and
- the zone of influence of the structure at the limit state being considered, throughout the design service life.

These four considerations that need to be taken into account when determining the representative value are explained in the following paragraphs.

#### 4.3.3.1 *Pre-existing knowledge*

The first consideration, taking account of pre-existing knowledge including geological information and data from previous projects, when determining the representative value is effectively the same as using comparable experience, which is defined in EN 1997-1/3.1.2.4 as 'documented previous information about ground and structural behaviour that is considered relevant for design, as established by geological, geotechnical and structural similitude with the design situation'.

As noted in Section 4.4 of this document, pre-existing knowledge in the form of experience in comparable cases needs to be considered when selecting a nominal value as the representative value. When determining a characteristic value as the representative value, the consideration of the pre-existing knowledge drives the designer to decide whether the coefficient of variation ($V_X$) is known, unknown or assumed, as shown in Section 4.5.2 of this document.

#### 4.3.3.2 *Quantity and quality of data*

The second consideration, taking account of uncertainty in the derived ground property values due to the quantity and quality of site-specific data, means that a more conservative representative value needs to be selected when the quantity of data is small and when the quality of the data is poor.

When determining the characteristic value as the representative value, the consideration of the quantity of data is taken into account directly by Formula 4.5 in EN 1997-1/4.3.2.2(4) and also Section 4.5.2 of this document, which includes the factor $k_n$ that depends on 'n' (number of data). Consideration of the quality of data is taken into account by the selection of the indicative value of $V_x$ for ground properties from EN 1997-1/Table A.1 where smaller or larger values inside the range provided should be chosen depending on the quality of the data.

#### 4.3.3.3 *Spatial variability and extent of zone of influence*

The third and fourth considerations are concerned with uncertainty due to the spatial variability of the measured property and the extent of the zone of influence of the structure at the limit state being considered. These considerations are related since EN 1997-1/4.3.2.1(3) states that 'The representative value of a ground property shall be determined for each limit state, according to its sensitivity to spatial variability of the ground property in the volume of ground involved'.

The following paragraphs introduce the concept of different types of estimate of the representative value according to the sensitivity of the limit state to the ground variability:

- Type A: estimate of the mean value;
- Type B: estimate of the lower or upper fractile value; and
- Type C: estimate of an intermediate value between Type A and B estimated values.

Type A (the mean value) and Type B (the lower or upper fractile value) are only presented in EN 1997-1/Annex A in connection with the determination of the characteristic value as the representative value. Although not included explicitly in EN 1997-1 in connection with nominal values, the Type A and Type B estimates are also relevant in the selection of the nominal value of the representative value. Type C is a further type of estimate introduced in this document for ground sensitivities between Types A and B.

Determining or selecting the appropriate representative value corresponding to Type A, B or C is a function of the designer's understanding of the extent of the limit state that mobilises the ground property. The magnitude of the ground spatial variability, represented by the extent of the failure mechanism or zone of influence, *L* compared to the magnitude of the scale of fluctuation of the ground property ($\theta$), as explained in the following paragraphs and summarized in Table 7.

**Table 7.** Types of estimate of the representative values

| Type(1) | Concept | L (2) v θ (3) | Representative value | |
|---|---|---|---|---|
| | | | *Value* | *Estimate of* |
| **A** | [In EN1997-1 // 4.3.2.1 (4)]<br>(4) If the limit state is "very" INSENSITIVE to ground spatial variability the Representative | L>> θ | Nominal (Cautious) | AVERAGE VALUE |

| Type[(1)] | Concept | L [(2)] v $\theta$ [(3)] | Representative value | |
|---|---|---|---|---|
| | Value shall be determined as an AVERAGE value | | or | |
| | | | Characteristic (Statistical) | MEAN VALUE |
| **B** | [In EN1997-1 // 4.3.2.1 (5)]<br>(5) If the limit state is “extremely” SENSITIVE to ground spatial variability the Representative Value shall be determined as an INFERIOR (or superior) value | $L < \theta$ | *Value* | *Estimate of* |
| | | | Nominal (Cautious) | INFERIOR/SUPERIOR Value |
| | | | or | |
| | | | Characteristic (Statistical) | 5/95% FRACTILE |
| **C** | [Not in EN1997-1]<br>If the limit state has and INTERMEDIATE sensitivity to ground spatial variability such that:<br>— Type A is considered not to be adequately cautious and<br>— Type B is considered to be overly conservative,<br>the Representative Value shall be determined as an INTERMEDIATE value between the average and inferior (or superior) value | $L > \theta$ | *Value* | *Estimate of an INTERMEDIATE VALUE between* |
| | | | Nominal (Cautious) => | Average and inferior/superior value |
| | | | or | |
| | | | Characteristic (Statistical) | Mean value and 5/95% fractile |

1 The various tones of the brownish colours highlight topics related to the various types of estimate (A, B and C)

2 L: reference length = vertical or horizontal dimension of the failure surface for the limit state upon consideration

3 $\theta$: scale of fluctuation = it describes the distance over which the properties of a ground layer are similar or correlated

*Source: Developed by the authors*

#### 4.3.3.3.1 Type A estimate

4.3.2.1(4) states that ‘If the limit state is insensitive to spatial variability of the ground, the representative value of the ground property shall be determined as an average value’. This is termed a Type A estimate of the representative value in EN 1997-1/Annex A. A Type A estimate of the representative value corresponds to the situation where the volume of ground involved in the limit state is large compared to the magnitude of the spatial variability of the ground property.

This is the situation if the extent of the ground involved in the limit state is very extensive compared to the scale of fluctuation and is not confined to failure within a weak zone or failure plane in a single geotechnical unit, so that the limit state is not sensitive to the spatial variability and is controlled by the average strength of the geotechnical unit involved in the limit state. In this situation:

— The representative/nominal value of the ground property at the limit is the cautious average value over the relevant volume – see Section 4.4.1 of this document.

— The representative/characteristic value is the mean value with 95% confidence over the relevant volume – see Section 4.5.1 of this document.

It should be noted, as stated in the first generation of EN 1997-2:2010/6.4(3), that '*Averaging can mask the presence of a weaker zone and should be used with caution. It is important that weak zones are identified. Variations in geotechnical parameters or coefficients can indicate significant variations in site conditions*.' Hence, if a specific design situation/limit state is sensitive to weaker zones, identifiable weaker zones should be distinguished and treated as separate geotechnical units.

#### 4.3.3.3.2 Type B estimate

4.3.2.1(5) states that '*If the limit state is sensitive to spatial variability of the ground, the representative value of the ground property shall be determined as an inferior or superior value*'. In EN 1997-1/Annex A, this is termed a Type B estimate of the representative value. This corresponds to a local failure situation where the magnitude of the spatial variability of the ground property in relation to the volume of ground involved in the limit state is significant. Consequently, the limit state is extremely sensitive and controlled by the inferior or superior value of the ground property over the extent of the volume of ground involved.

This is the situation if failure is confined to a weak or strong zone within a single geotechnical unit, so that the limit state is sensitive to the ground variability and the failure mode is controlled by the weak or strong zone of ground within the geotechnical unit. In this situation:

— The representative/nominal value is the inferior value, if the ground strength is favourable, or the superior value if the ground strength is unfavourable.

— The representative/characteristic value is the 5% fractile value when the ground property is favourable and the 95% fractile when the ground property is unfavourable.

#### 4.3.3.3.3 Type C estimate

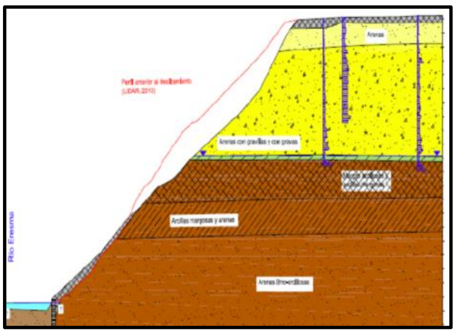
Paragraphs 4.3.2.1(4) and 4.3.2.1(5), referenced in relation to Type A and Type B above, provide the bounds for determining the representative ground property value for a single geotechnical unit with respect to the sensitivity of the limit state to the spatial variability of the ground. These bounds for the representative value are the average value or mean value with 95% confidence (Type A value), when the limit state is very insensitive to the ground spatial variability, and the inferior (5% fractile) or superior (95% fractile) value (Type B value), when the ground is extremely sensitive to the ground spatial variability. In many practical geotechnical design situations, the limit state will have an intermediate degree of sensitivity and hence the representative value will be between the bounds of the Type A and Type B estimates. Note 1 to A.4(3) states that *'the ratio of the scale of fluctuation to the extent of the failure surface can be used in the determination of the characteristic values'*. Use of the scale of fluctuation to determine the characteristic value is explained in Section 4.5.2. of this document

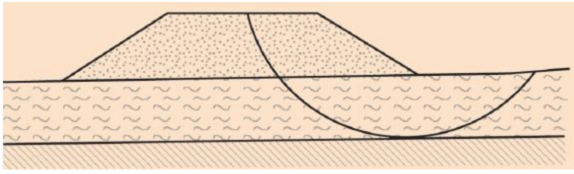
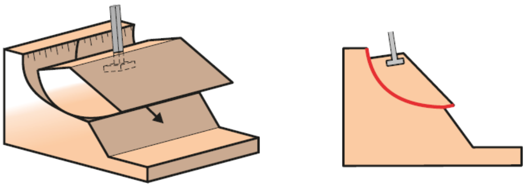
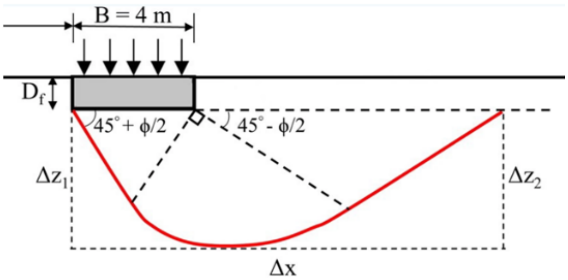
While not included in EN 1997-1, this document introduces a third type of estimate of the representative value (Type C). This type is applicable to situations where the extent of the spatial variability of the ground with respect to the volume of ground involved in the limit state is such that the limit state is sensitive to the spatial variability of the ground property, but not to the extent corresponding to the bounds given by the average and the inferior or superior values. Hence, it provides for a representative value that is between the average and the inferior or superior value. Type C representative values may be chosen for design situations where Type A is considered not to be adequately cautious and where Type B is seen to be overly cautious. As noted above, the choice of a representative value corresponding to Type A, B or C is a function of the designer's understanding of the limit state mechanism that mobilises the ground property and the magnitude of the spatial variability of such ground property. Further guidance is provided in Section 4.5.2 of this document.

The fact that limit states in geotechnical design are sometimes sensitive and sometimes not sensitive to the ground material variability is an important reason why geotechnical design and hence EN 1997 differs from structural design. The reason why limit states in structural design are sensitive to the material variability is because, in structural design, it is assumed all the materials in the volume involved in a limit state are uniform and have no spatial variability. For example, in the verification of limit states in a beam, it is assumed that the strength of concrete is uniform (it has no spatial variability) which is why it is necessary to choose the 5% fractile of the inherent variability, in order to have 95% confidence level that the strength will not be less than the characteristic value when the strength is favourable. However, in geotechnical design, the ground has spatial variability as well as inherent variability. Hence the magnitude of the spatial variability of the ground and the extent of the failure volume needs to be considered together when selecting the representative value of a ground property, not just the inherent variability.

Examples of geotechnical structures, including a slope, embankment, spread foundation, piled foundation, retaining structure and anchor with limit states when the different types of estimate (Types A, B and C) are relevant, depending on the magnitude of the scale of fluctuation and the extent of the failure mechanism, are given in Table 8 and Table 9. In these tables, $L_v$ and $L_h$ are the vertical and horizontal extent of the failure mechanism or zone of influence in the case of spread foundation settlement, while $\theta_v$ and $\theta_h$ are the magnitudes of the scale of fluctuation of the ground property in the vertical and horizontal directions. It should be noted that the choice of the appropriate type of estimate of the ground property value for SLS verification is complicated by the need to consider, in addition to the ground spatial variability, the nature and extent of the supported structure. For example, a more cautious estimate may be appropriate in the case of the settlement of a structure with multiple footings.

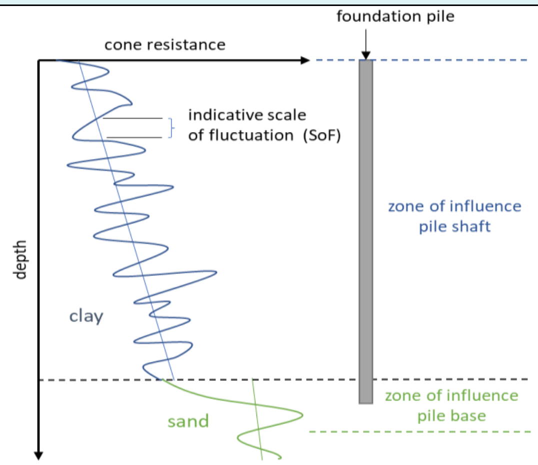
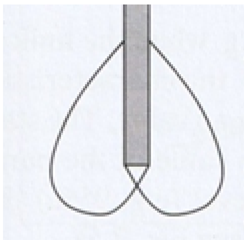
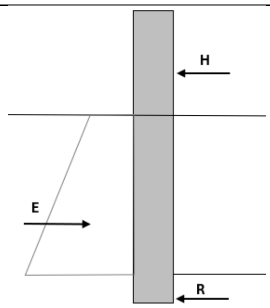
**Table 8.** Examples of a slope, embankment and spread foundation with limit states involving different types of estimates of the representative value.

| Geotechnical Structure | Limit State | Figure | Type of estimate in each Geotechnical Unit |
|---|---|---|---|
| **Slope** | Overall stability |  $L_v >> \theta_v \approx 1,35$ m | Type A |

| Geotechnical Structure | Limit State | Figure | Type of estimate in each Geotechnical Unit |
|---|---|---|---|
| **Embankment** | Bearing resistance | <br>$L_v >> \theta_v$ | Type A |
| **Spread foundation** | Overall stability | <br>$L_v >> \theta_v$ | Type A |
| | Bearing resistance | <br>$L_v = \Delta_{z1} = 1{,}5\ B + D > \theta_v$ | Type C |
| | Sliding resistance | $L_v >> \theta_v$ | Type B |
| | Settlement (SLS) | $L_v$ (effective depth) = $1{,}5\ B > \theta_v$ | Type C |

*Source: Developed by the authors. The illustrations within the table are original works developed by the authors.*

**Table 9.** Examples of a piled foundation, retaining structures and anchor with limit states involving different types of estimates of the representative values.

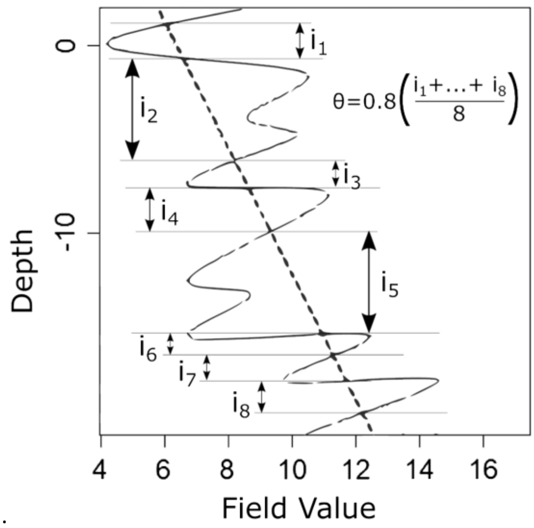
| **Geotechnical Structure** | **Limit State** | **Figure** | **Type of estimate in each Geotechnical Unit** |
|---|---|---|---|
| **Piled foundation** | Axial compressive resistance | <br>Shaft / $L_v = L_{clay} => L_v >> \theta_v$ | Shaft:<br>Type A |
| | | <br>Base / $L_v = 9\ D => L_v > \theta_v$ | Base:<br>Type C |
| | Axial tensile resistance | Similar to shaft bearing resistance | Shaft:<br>Type A |
| | Transverse resistance | <br>$L_v = L$ on left side (in figure) | Left side:<br>Type A<br>Right side of base:<br>Type C |
| **Retaining structures** | Overall Stability | Similar to slope stability situation | Type A |
| | Bearing resistance of gravity walls | Similar to bearing resistance of a spread foundation | Type C |
| | Bearing resistance of | Similar to bearing resistance of a pile | Embedded wall:<br>Type A |

| Geotechnical Structure | Limit State | Figure | Type of estimate in each Geotechnical Unit |
|---|---|---|---|
| | embedded walls | | Embedded wall base: Type C |
| | Rotational resistance | Similar to slope stability situation | Type A |
| **Anchor** | Geotechnical resistance of an anchor | Depends on ratio of scale of fluctuation to anchorage length | Type C |

*Source: Developed by the authors. Illustrations within the table are original works developed by the authors, except the one of ´Base of piled foundation´ by Frank et al. (2004)*

A simple way to visualize the physical meaning of scale of fluctuation is through the rule of thumb procedure to determine the scale of fluctuation proposed by Spry et al. (1988) when a continuous profile with depth is obtained. This procedure involves averaging the distances between intersections (called zero crossings in the signal processing literature) with the trendline. The scale of fluctuation can then be estimated as 80% of this average distance, assuming a Gaussian random field. An example of using this procedure with a Cone Penetration Test (CPT) profile is shown in Figure 10.

**Figure 10.** Illustration of the rule of thumb method for estimating the scale of fluctuation ($\theta$)




*Source: Cami et al (2020)*

Indicative values of the horizontal and vertical scales of fluctuation ($\theta_h$ and $\theta_v$) obtained from literature review for different soils and published by Cami et al. (2020), are presented in Table 10. Summary of scales of fluctuation from literature review by soil type. These indicative values, relative to sedimentary materials not involved in tectonic movements, show that the $\theta_h$ values are much greater than the $\theta_v$ values.

In practice the $\theta_v$ value is the most relevant scale of fluctuation value, as shown by the examples in Table 8 and Table 9. Since $\theta_v$ values are usually not available in practice, a recommended procedure to assess the sensitivity of a limit state to the spatial variability is to use the weighted average of the $\theta_v$ values in Table 10. Summary of scales of fluctuation from literature review by soil type, which is equal to 1.35 m. This $\theta_v$ value has been used to demonstrate that the spread foundation bearing resistance and settlement examples in Table 8 are Type C. The foundation width is 4 m in these examples, so that $L_v$ is greater than $\theta_v$, but of the same order of magnitude so that they are Type C, not Type A. In the case of the spread foundation sliding example, $L_v$ is usually small (much less than 1.0 m), which is less than $\theta_v$, so that it is Type B, not Type C. It should be noted that in the case of sliding, as in the case of bearing resistance and settlement, the sensitivity of the limit state is controlled by $\theta_v$ rather than by $\theta_h$.

**Table 10**. Summary of scales of fluctuation from literature review by soil type

| Soil | Horizontal scale of fluctuation, $\theta_h$ (m) | | | | Vertical scale of fluctuation, $\theta_v$ (m) | | | |
|---|---|---|---|---|---|---|---|---|
| | **Number averaged** | **Min** | **Max** | **Average** | **Number averaged** | **Min** | **Max** | **Average** |
| Alluvial | 9 | 1.07 | 49 | 14.2 | 13 | 0.07 | 1.1 | 0.36 |
| Ankara clay | - | - | - | - | 4 | 1.0 | 6.2 | 3.63 |
| Chicago clay | - | - | - | - | 2 | 0.79 | 1.25 | 0.91 |
| Clay | 9 | 0.14 | 163.8 | 31.9 | 16 | 0.05 | 3,62 | 1.29 |
| Clay, silt and sand mix | 13 | 121.2 | 1 000 | 201.5 | 28 | 0.06 | 21 | 1.58 |
| Hangzhou clay | 2 | 40.4 | 45.4 | 42.9 | 4 | 0.49 | 0.77 | 0.63 |
| Marine clay | 8 | 8.37 | 66 | 30.9 | 9 | 0.11 | 6.1 | 1.55 |
| Marine sand | 1 | 15 | 15 | 15.0 | 5 | 0.07 | 7.2 | 1.43 |
| Offshore soil | 1 | 24.6 | 66.5 | 45.6 | 2 | 0.48 | 1.62 | 1.04 |
| Overconsolidated clay | 1 | 0.14 | 0.14 | 0.14 | 2 | 0.063 | 0.255 | 0.15 |
| Sand | 9 | 1.69 | 80 | 24.5 | 14 | 0.1 | 4.0 | 1.17 |
| Sensitive clay | - | - | - | 33.3 | 2 | 1.1 | 2.0 | 1.55 |
| Silt | 3 | 12.7 | 45.5 | - | 5 | 0.14 | 7.19 | 2.08 |
| Silty clay | 7 | 9.65 | 45.4 | 29.8 | 14 | 0.095 | 6.47 | 1.40 |
| Soft clay | 3 | 22.2 | 80 | 47.6 | 8 | 0.14 | 6.2 | 1.70 |
| Undrained engineered soil | --- | --- | --- | --- | 22 | 0.3 | 2.7 | 1.42 |
| Weighted average | --- | | | **20 - 30** | --- | | | **1.35** |

*Source: Cami et al. (2020).*

#### 4.3.3.4 *Zone of influence throughout the design service life of the geotechnical structure*

The last consideration takes account of potential changes in the zone of influence throughout the design service life of the geotechnical structure, such as the effect of subsequent excavation, placement of overburden or changes in groundwater pressure. This fact is important since, because of such changes in the ground conditions, the following effects may occur:

— The weight density, and hence the strength properties of the ground, may increase as a result of increased effective stresses due to the placement of overburden or due to the lowering of the groundwater level, causing a reduction in groundwater pressure, or

— The weight density, and hence the strength properties of the ground, may reduce as a result of reduction in effective stresses due to excavation or due to the raising of the groundwater level, causing an increase in groundwater pressure.

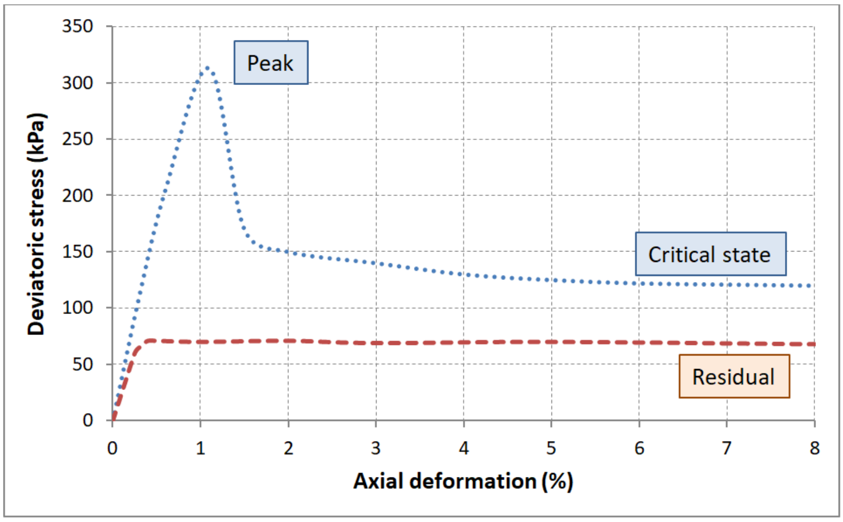
How the four factors listed in 4.3.2.1(2), that need to be taken into account when selecting the representative value of a ground property, are taken into account in the case of nominal and characteristic values is outlined in Sections 4.4 and 4.5 of this document.

### 4.3.4 Considerations on the shear strain magnitude: peak, constant-volume, critical state and residual values

Another factor to be taken into account when selecting the representative ground property value for a particular limit state, but not listed in 4.3.2.1(2) is the effect of the magnitude of the shear strain, including dilatancy. Dense, coarse-grained soils dilate when sheared, exhibiting a peak angle of friction ($\phi'_p$) and then, after sufficient shearing, reach the constant-volume or critical state angle of friction ($\phi'_{cv}$) when the soil is completely remoulded. In the case of fine-grained soils, the soil particles may become aligned along the failure surface after large strains, for example on pre-existing failure surfaces, with the result that a residual angle of friction is reached, which is less than the critical state angle. What is important in geotechnical design is the soil strength that is available to prevent a limit state occurring so, when selecting the representative value, the derived values obtained from test results and other data should be interpreted appropriately for the limit state being considered.

EN 1997-1 and EN 1997-2 generally do not state if the soil shear strength to be used in design verifications is the peak, critical state (constant volume) or residual value. Examples of plots of deviator stress against strain for a dense coarse soil (blue line), a loose coarse soil and a fine soil (red line), showing the peak, critical and residual strengths for these soils. are plotted in Figure 11. Since structures will generally be considered to have failed well before the deformations required to get to the critical state have been reached, verifications have traditionally been carried out using the peak strength. Most calculation models are correlated to the peak strength, this is the strength that is normally used as the representative strength in designs to Eurocode 7. However, in the design of slopes, it is stated in EN 1997-3/4.5.1(3) that '*The resistance of pre-existing sliding surfaces should be determined using residual strength properties*'.

**Figure 11.** Test results with indication of peak, critical state, and residual strengths




*Source: Developed by the authors*

When verifying the sliding resistance of a spread foundation, it is stated in EN 1997-3/5.5.3(11) that '*the representative angle of friction of soil or fill should consider potential disturbance of the soil or fill beneath the foundation*'. Hence, when determining the representative value, it is necessary to consider if it is the peak, critical state or an intermediate value that is relevant.

## 4.4 Nominal and indicative values

### 4.4.1 Nominal value

The nominal value of a ground property ($X_{nom}$) is defined in EN 1990-1/3.1.2.29 as the '*value fixed on a non-statistical basis; for instance, on acquired experience or on physical conditions*'. EN 1997-1/ 4.3.2.3(1) elaborates this by stating that '*the nominal value of a ground property ($X_{nom}$) shall be selected as a cautious estimate of the (average, inferior or superior) value affecting the occurrence of the limit state (based on the knowledge of the construction site and comparable experience)*'. This definition is almost identical to the definition for the characteristic value of a ground property in the first generation of Eurocode 7 (EN 1997-1:2004). This definition was introduced because a hypothetical unlimited test series, which is assumed in the definition for a characteristic value in first generation of EN 1990, is never available in geotechnical design. This implies that the definition for a characteristic value in the first generation of EN 1990 was intended for materials manufactured under carefully controlled conditions, rather than for a natural and variable material like the ground.

The nominal value is selected based on derived values, knowledge of the construction site, consideration of the design situation and limit state, and comparable experience. The nominal value of a ground property may be used as the representative value in geotechnical design verification calculations to EN 1997-3. It is a cautious estimate of the property value chosen to represent the behaviour of the volume of ground involved in the limit state being considered. It is selected by the designer based on the scatter of the derived values and that person's knowledge of the site and experience of similar design situations, i.e. it is based on comparable experience. Depending on the design situation, the nominal value could be chosen as one of the three following alternatives (see also Table 7. Types of estimate of the representative values and Section 4.3.3.3 of this document):

— A cautious estimate of the average of the derived values when the failure involves a large volume of ground, as in the case of a slope stability situation, and failure is not confined to weak zones within the volume of ground affected by the limit state, i.e. as Type A estimate as discussed in 4.3.3.3.1, or

— A more cautious estimate, i.e. an inferior value, which is a lower bound to the derived values, when a small volume of soil is involved and the ground property is favourable, so that it is sensitive to the variability in the ground property value, as in the case of the failure around the base of a pile, i.e. a Type B estimate as discussed in 4.3.3.3.2.

— Since, as noted in 4.3.3.3.3in connection with Type C estimates, in many practical geotechnical design situations, the limit state will have an intermediate degree of sensitivity to the spatial variability ground property and hence the nominal value will be between the bounds of the Type A and Type B estimates. In such situations an intermediate, Type C estimate of the nominal value should be chosen that takes account of the designer's comparable experience and assessment of the magnitude of the scale of fluctuation compared to the extent of the failure surface.

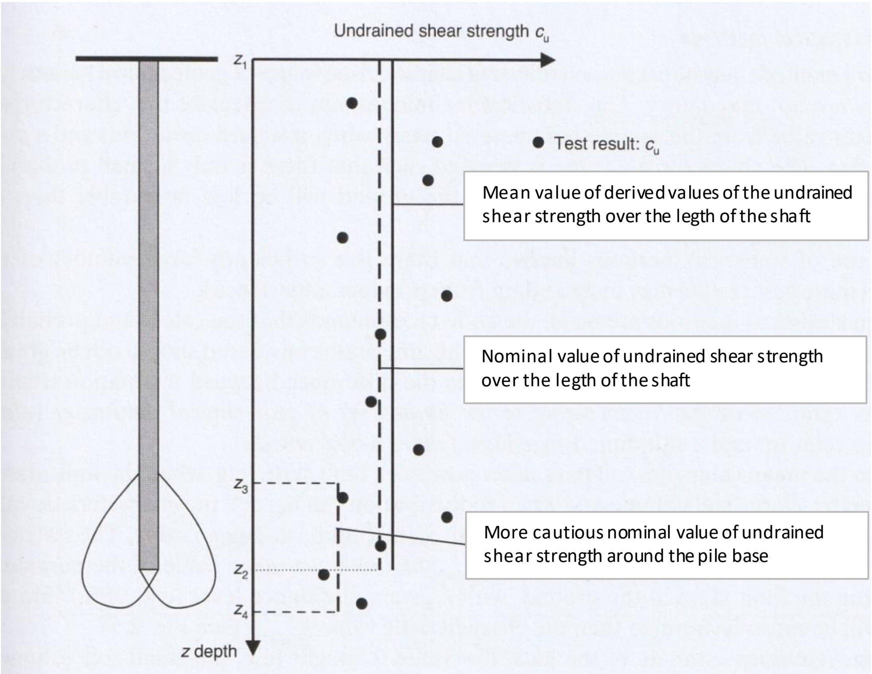
Figure 12, based on Frank et al. (2004), shows that the nominal value of the undrained shear strength when calculating the shaft resistance of a pile may be chosen as a cautious average of derived values but, when assessing the base resistance of a pile, the nominal value of the undrained shear strength would be a more cautious estimate of the derived values. The assumption implicit in Frank et al (2004) is that the failure mechanisms are understood when assessing the pile resistance, so:

— Where it can be determined that the ground is not weaker at the pile base (including through understanding of the ground model and the geological setting, etc.) then a single nominal ground strength profile could be chosen for assessing the shaft and base resistances.

— However, if it is determined that the ground is weaker at the pile base, then, when selecting a single nominal ground strength value for the design of a pile, it will be appropriate to select a more cautious value than the value selected for the design of the pile shaft, i.e. a Type C estimate, due to the smaller extent of the failure volume at the pile toe and hence its sensitivity to variation in the ground strength.

Some guidance on the level of caution to be adopted when selecting the average and inferior/superior nominal values may be obtained from the general formula (Equation 19) in Section 4.5.3 of this document.

**Figure 12.** Nominal values for undrained shear strength along shaft of a pile and at the base of a pile




*Source: Frank et al. (2004)*

### 4.4.2 Indicative value

The indicative value is not defined in EN 1990-1 but is defined in EN 1997-1/ 3.1.3.7 as the '*value of a ground property determined from ground description or classification*'. Indicative values are only referred to twice in the code text of EN 1997-1. There are no references to indicative vales in EN 1997-2 or in EN 1997-3. The references to indicative values in EN 1997-1 are in 4.3.2.3(2), which states that '*Indicative values may be used as nominal values provided they are specified by the relevant authority or, when not specified, agreed for a specific project by the relevant parties*', and in 4.3.2.3(3), which states that '*Where an indicative value is used as a nominal value, it shall be selected as a very cautious estimate of the value affecting the occurrence of the limit state*'.

An indicative value '*specified by the relevant authority*' is the value of a ground property given in a National Annex or agreed for a specific project by the relevant parties. As an indicative value may be used as a nominal value, it should be factored in the same way as nominal value in order to obtain the design value.

The concept of an indicative value should not be regarded as completely new since some National Annexes, for instance that for The Netherlands, have tables with values for various ground properties for some types of ground that can be considered, from the perspective of the second generation of Eurocode 7, as being 'indicative values'.

The only place in EN 1997 where an indicative value is referred to in relation to a design calculation is in Note 1 of EN 1997-1/4.3.2.2(4), which states that '*Annex A gives a procedure to evaluate the different terms in Formula (4.5) and provides indicative values of $V_X$ for common ground properties and test parameters'.* The indicative $V_x$ values given in Tables A.1 and A.2 in EN 1997-1/Annex A and reproduced in Table 11 and Table 12 of this document are for use in Equation 15 of this document (equal to Formula 4.5 in EN 1997-1) to determine the characteristic value of ground properties (See 4.5.2 for more information about determining the characteristic value).

The indicative $V_X$ values presented in Tables 4.1 and 4.2 in EN 1997-1/Annex A, which are used to determine characteristic ground strength values, demonstrate that indicative values are not based on derived values from a specific site but are typical values that may be found in the literature, based on comparable experience, and may be linked to average properties describing the nature or state of the ground. Since an indicative value is not based on derived values obtained from a particular site, there is considerable uncertainty about the relevance of an indicative value for a particular design situation. This is highlighted in the important requirement in 4.3.2.3(3) that '*Where an indicative value is used as a nominal value, it shall be selected as a <u>very</u> cautious estimate of the value affecting the occurrence of the limit state*' – note authors' underlining.

## 4.5 Characteristic value

### 4.5.1 Definition of the characteristic value

The characteristic value of a material or product property ($X_k$) is defined in EN 1990-1/3.1.4.1 as the '*value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series*'. Note 1 to this definition adds: '*This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product*', which indicates that, in contrast to the nominal value, the characteristic value is determined using statistical procedures. Information about the determination of the characteristic value using statistical procedures is given in 4.5.2. The definition of the characteristic value of a ground property given in EN 1997-1/3.1.3.4 is that it is the '*statistical determination of the value of a ground property that affects the occurrence of a limit state having a prescribed probability of not being attained*'. There is no reference to a hypothetical unlimited test series, as in the EN 1990-1 definition since that is not possible in the case of a ground property.

Further to the statistical definition of the characteristic value in EN 1990-1/3.1.4.1, EN 1997-1/4.3.2.2(2) states that 'When the verification of a geotechnical limit state is sensitive to the variability of a ground property, its characteristic value should be defined as:

- An estimate of the 5% fractile value, where a low (inferior) value of the ground property is unfavourable; or
- An estimate of the 95% fractile value, where a high (superior) value of the ground property is favourable.

These definitions correspond to the Type B estimate of the characteristic value discussed in Section 4.3.3.3.2 of this document.

It should be noted that the definitions in EN 1990-1 of the characteristic value are for designs involving structural materials where the verification of the limit state is usually sensitive to the variability of the material property. When the verification of a limit state is insensitive to the variability of a material property, EN 1990-1 states in 6.2(3) that in that situation '*its characteristic value should be defined as the mean value, unless otherwise stated in the other Eurocodes*'. EN 1997-1/4.3.2.2(1) provides an additional definition of the characteristic value of a ground property when it is insensitive to the ground variability by stating that '*In addition to EN 1990-1/6.2(3), when the verification of a geotechnical limit state is insensitive to the variability of a ground property, its characteristic value should be defined as an estimate of the mean value.*' This definition corresponds to the Type A estimate of the characteristic value discussed in Section 4.3.3.3.1of this document.

For a situation where the sensitivity of the limit state depends on the magnitude of the scale of fluctuation of the ground compared to the extent of the failure zone, the characteristic value of the ground property should be chosen as an intermediate value between the inferior or superior fractile and the mean value with 95% confidence that the ground property controlling the limit state is greater or less, respectively. This corresponds to the Type C estimate of the representative value discussed in Section 4.3.3.3.3 of this document.

### 4.5.2 Statistical formulae in EN 1997-1 to calculate the characteristic value

EN 1997-1/ 4.3.2.2(3) states that 'the characteristic value of a ground property ($X_k$) should be determined from either a normal or a log-normal distribution, as appropriate'. In order to determine the characteristic value of a ground property statistically taking account of the number (n) of derived values and when a normal distribution is assumed, EN 1997-1/4.3.2.2(4) states that it should be determined from the Equation 15:

**Equation 15.** Determination of characteristic values [Taken from EN 1997-1/Formula (4.5)]

$$X_k = X_{mean}\,[1 \mp k_n V_X] = X_{mean}\left[1 \mp \frac{k_n s_x}{X_{mean}}\right]$$

where:

$X_{mean}$ is the mean value of the ground property X from a number n of derived values;

$V_X$ is the coefficient of variation of the ground property, equal to the ratio of the standard deviation to the mean, i.e. $s_X/X_{mean}$;

$k_n$ is a coefficient that depends on the number of derived values, *n* used to calculate $X_{mean}$;. The value of $k_n$ also depends on level of knowledge regarding $V_X$, and what type of estimate the characteristic value is. Values of $k_n$ for different $V_X$ cases and types of estimate are given in EN 1997-1/Table A.1, which is included below as Table 13;

$\mp$ denotes that $k_nV_X$ should be subtracted when a lower value of $X_k$ is required and added when an upper value is required;

$s_x$ is the standard deviation of the sample derived values *X*

There are three notes to 4.3.2.2(3) to explain the use of Equation 15:

- Note 1 states that EN 1997-1/Annex A 'gives a procedure to evaluate the different terms in Formula (4.5) and provides indicative values of $V_X$ for common ground properties and test parameters'.
- Note 2 states that 'different expressions are used for other statistical distributions (see EN 1990-1/ Annex C)' but are not given in this document.

— Note 3 states that 'other procedures can be used to determine the characteristic values of a ground property varying with depth (e.g. using least squares with regression analysis) or the characteristic values of dependent properties (e.g., cohesion and friction angle)'. See Sections 5.2 and 5.3 of this document for guidance on determining the characteristic value of ground properties that increase with depth and that are dependent.

In order to account for the fact that the characteristic value calculated using Equation 15 of this document (equal to Formula 4.5 in EN 1997-1) depends on the level of knowledge concerning the parameter $V_X$, EN 1997-1/Annex A.4 identifies three different situations, i.e. whether the $V_X$ value is known, unknown or assumed, referred to as Cases 1, 2 and 3 as follows:

— Case 1: $V_X$ is known from prior knowledge;

— Case 2: $V_X$ is unknown initially with no prior knowledge to indicate its likely value. In this situation the designer should calculate the $V_X$ value from the mean and standard deviation, $X_{mean}$ and $s_X$, of the derived $X$ values using Equation 16:

**Equation 16** Determination of the coefficient of variation [Taken from EN 1997-1/Formula (A.4)]

$$V_X = \frac{s_X}{X_{mean}};\ s_X = \sqrt{\frac{\sum_{i=1}^{n}(X_i - X_{mean})^2}{n-1}}$$

— Case 3: $V_X$ is assumed and the designer chooses indicative value for $V_X$, for example from the values given in EN 1997-1/Table A.1 for ground properties and Table A.2 for test parameters and presented in this document as Tables 7 and 8, respectively.

Note 2 after A.4(8) on the selection of the $V_X$ value when $V_X$ is known from prior knowledge, i.e. in Case 1, states that '*prior knowledge can come from previous tests in comparable situations. Engineering judgement is used to determine what can be considered as 'comparable*''. Note 2 on the selection of $V_X$ values states that 'In practice, it is often preferable to use Case 3 '$V_X$ assumed' together with a conservative upper estimate of $V_X$, rather than to apply the rules given for Case 2', which are given in A.4(11). These rules are that $V_X$ should be calculated using the derived $X$ values and Formula (A.4) in EN1997-1/Annex A. However, the reason it is often preferable to use the Case 3 'assumed' approach and adopt a conservative upper estimate value of $V_X$ rather than calculate the values using Formula (A.4) is because the $V_X$ value calculated using this formula can be unreliable, particularly when the number of derived values is small. Case 3 is normally the only possible case that can be adopted in a situation involving dependent ground properties. In geotechnical practice, prior knowledge of the coefficient of variation enabling the use of Case 1 is a rare situation.

Tables A.1 and A.2 in EN 1997-1/Annex A with indicative values of $V_X$ for different ground properties and test parameters are reproduced below as Table 11 and Table 12.

**Table 11.** Indicative values of coefficient of variation for different ground properties (EN 1997-1/Annex A-Table A.1).

| **Soil/Rock type** | **Ground property** | **Symbol** | **Coefficient of variation $V_X$ (%)** |
|---|---|---|---|
| All soils and rock | Weight density | $\gamma$ | 5-10 |

| Soil/Rock type | Ground property | Symbol | Coefficient of variation $V_X$ (%) |
|---|---|---|---|
| Fine grained soils | Shear strength in total stress analysis | $c_u$ | 30-50 |
| All soils and rocks | Peak or residual effective cohesion | $c'_p$ $c'_r$ | 30-50 |
| All soils and rocks | Coefficient of friction | tan $\varphi'$ | 5-15 |
| All soils and rocks | Shear strength at failure | $t_f$ | 15-25 |
| All soils and rocks | Unconfined compressive strength | $q_u$ | 20-80 |
| All soils | Modulus of deformability(1) | *E or G* | 20-70 |
| Fine-grained soils | Vertical or horizontal consolidation coefficient | $c_v$ *or* $c_h$ | 30-70 |
| All soils | Hydraulic conductivity(2) | *K* | 70-250 |

1 It refers to the modulus of deformation whose symbols appear in 3,2,1 and 3.2.7/EN 1997-2.

2 Given the high value of the coefficient of variation, Equation 15 should not be used, in this case.

*Source : EN 1997-1/Annex A-Table A.1*

**Table 12.** Indicative values of coefficient of variation for different test parameters (EN 1997-1, Annex A, Table A.2).

| Soil/Rock type | Test parameter | Symbol | Coefficient of variation $V_X$ (%) |
|---|---|---|---|
| Coarse soils | SPT blow count | $N_{SPT}$ | 15-45 |
| All soils | Pressuremeter limit pressure | $p_l$ | 5-15 |
| All soils | Cone resistance | $q_c$ | 5-15 |
| All soils | Sleeve friction | $f_s$ | 5-15 |

*Source: EN 1997-1/Annex A-Table A.2*

Equations to determine $k_n$ for the Type A and B estimates of the characteristic value, evaluated with a 95% confidence level and for different numbers of derived values, for the different situations regarding the level of knowledge of $V_X$, i.e. Cases 1, 2 and 3, are given in EN 1997-1/Annex A and presented in EN 1997-1/Table A.3. The formulae in EN 1997-1/Annex A are only for the bounds corresponding to the Type A and B estimates of the characteristic value as discussed in Section 4.3.3.3 of this document. To account for the sensitivity of the limit state to variability of the ground with regard to the extent of the volume of ground involved in the limit state, a more general form of the formulae for $k_n$ given in EN 1997-1/Annex A is proposed as Equation 18 that includes the sensitivity index ($\Gamma^2$) defined by Equation 17 (Vanmarcke, 1983):

**Equation 17**. Determination of the sensitivity index

$$\Gamma^2 = \frac{\theta}{L}$$

where $\theta$ is the scale of fluctuation, i.e. magnitude of the ground property variability, and L the extent of the volume of ground involved in the limit state:

**Equation 18.** Determination of the coefficient $k_n$ accounting for the sensitivity index

$$k_n = \left(N_{95}\ or\ t_{95,n-1}\right)\sqrt{\Gamma^2 + \frac{1}{n}}$$

Equation 18 includes the factors $N_{95}$ or $t_{95,n-1}$ to account for the level of knowledge concerning the $V_x$ value where:

— $N_{95}$ is the factor, equal to 1.64 for all values of $n$, that evaluates the characteristic value for a normal distribution with a 95% confidence level and infinite degrees of freedom when $V_X$ is known or assumed (Cases 1 and 3); and

— $t_{95,n-1}$ is the factor, ranging from 6.31 for $n$ = 1 to 1.66 for $n$ = 100, that evaluates the characteristic value for the Student's t distribution with a 95% confidence level and $n-1$ degrees of freedom when $V_X$ is unknown (Case 2);

In Type A, $L$ is very large compared to $\theta$ and hence the sensitivity index can be considered equal to 0 ($\Gamma^2$ = 0) as the limit state is very insensitive to the ground variability. In Type B, $L$ is equal to or less than θ and the sensitivity index is calculated to be equal to or greater than 1 but a maximum value of 1 is adopted ($\Gamma^2$ = 1) when the limit state is extremely sensitive to the ground variability, corresponding to the 5% or 95% fractile values. In Type C, $L$ has a value between being very large and equal to the magnitude of the scale of fluctuation ($\theta$) and hence the sensitivity index $\Gamma^2$ has a value between 0 and 1.

Tables A.4 to A.7 in EN 1997-1/Annex A provide selected values of $N_{95}$, $t_{95,n-1}$ and $k_n$ values to calculate the characteristic value as the mean value and the inferior or superior fractile value. The values of $N_{95}$ and $t_{95,n-1}$ values may also be used with Equation 18 to calculate the $k_n$ values for Type C. The formulae for $k_n$ for the different types of estimates of the characteristic value and the different $V_x$ cases providing 95% confidence that the calculated value is not less than the actual value controlling the limit state are given in Table 13.

**Table 13.** Formulae for $k_n$ for different combinations of types of estimates and $V_x$ cases.

| **Cases with different knowledge concerning $V_x$** | **Types of estimate of the characteristic value** | | |
|---|---|---|---|
| | Type A: Estimate of the mean value | Type B: Estimate of the inferior (5%) or superior (95%) fractile value | Type C: Estimate of the intermediate value when |
| | ($\Gamma^2$ = 0) | ($\Gamma^2$ = 1) | $\Gamma^2$ = $\theta$/L |
| (1) Case 1: $V_x$ known and Case 3: $V_x$ assumed | $N_{95}\sqrt{\frac{1}{n}}$ (A1) | $N_{95}\sqrt{1+\frac{1}{n}}$ (B1) | $N_{95}\sqrt{\Gamma^2+\frac{1}{n}}$ (C1) |
| (2) Case 2: $V_x$ unknown | $t_{95,n-1}\sqrt{\frac{1}{n}}$ (A2) | $t_{95,n-1}\sqrt{1+\frac{1}{n}}$ (B2) | $t_{95,n-1}\sqrt{\Gamma^2+\frac{1}{n}}$ (C2) |

*Source: Developed by the authors*

EN 1997-1/Annex A refers to combinations of different types of estimate and cases of $V_x$. For example, a Type A estimate when $V_x$ is known or assumed is Combination A1, while a Type B estimate when $V_x$ is unknown is Combination B2, as shown in Table 13.

### 4.5.3 Effect of spatial variability on the characteristic value

The following general formula for the ratio of the characteristic value to the mean value, ($X_k/X_{mean}$) as a function of the spatial variability expressed at the ratio of the scale of fluctuation to the extent of the failure volume ($\theta/L$) is obtained by substituting Formula C1 for $k_n$ in Table 13 in Equation 18:

**Equation 19.** Ratio of the characteristic value to the mean value

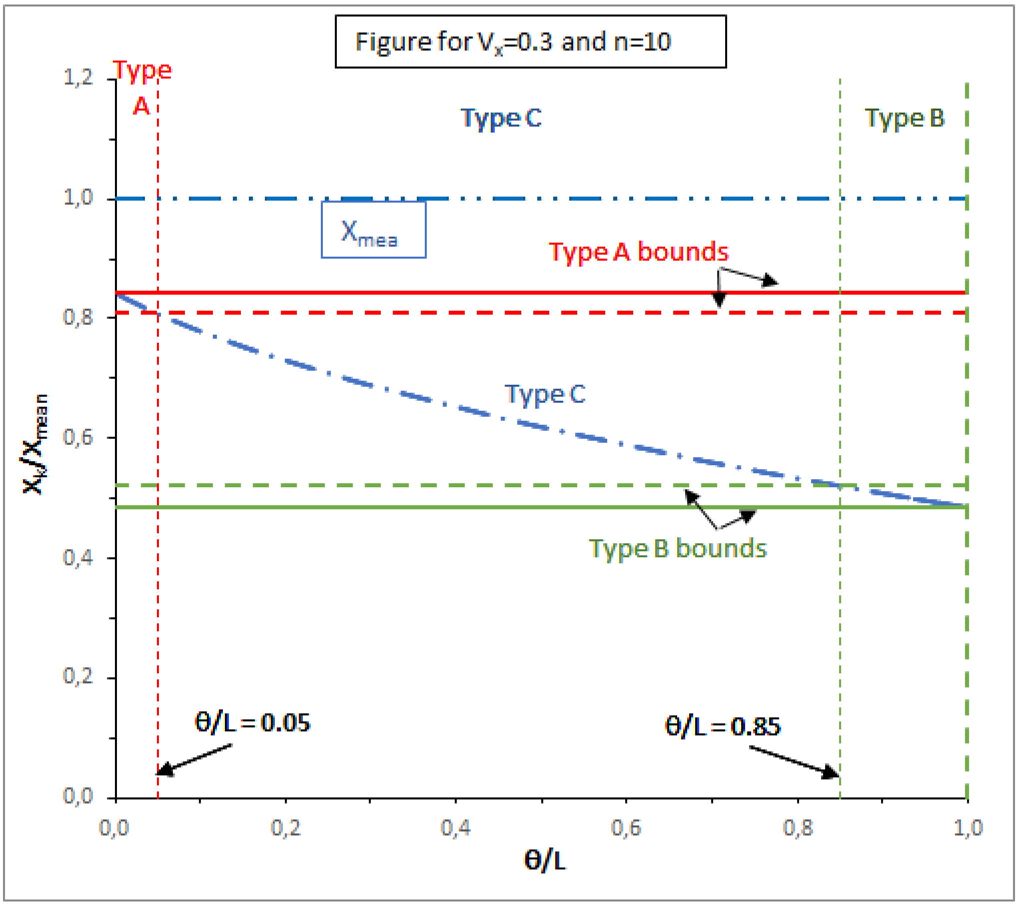
$$\frac{X_k}{X_{mean}} = 1 - \left( N_{95}\sqrt{\Gamma^2 + \frac{1}{n}} \right) V_x = 1 - \left( N_{95}\sqrt{\frac{\theta}{L} + \frac{1}{n}} \right) V_x$$

Taking the example of a ground with $Vx = 0.3$ when there are 10 derived values, and using the normal distribution factor for a 95% confidence level, $N_{95} = 1.64$, the graph of Equation 18 in Figure 13 shows that the characteristic mean value of $X_k/X_{mean}$ decreases from 0.844 when $\theta/L = 0$, i.e. Type A, to 0.484 when $\theta/L = 1.0$, i.e. Type B. For $\theta/L$ values between 0 and 1.0, Type C pertains. However, it is necessary to decide on the bounds for Type A and B and when the $X_k/X_{mean}$ values should be considered Type C rather than Type A or B. In this document it is proposed that:

- the lower bound of the Type A estimate should be when the $X_k/X_{mean}$ value is equal to its value at $\theta/L = 0$ minus ten percent (10%) of the difference between its values at $\theta/L = 0$ and 1.0.
- the upper bound of the Type B estimate should be when the $X_k/X_{mean}$ value is equal to its value at $\theta/L = 1.0$ plus ten percent (10%) of the difference between its values at $\theta/L = 0$ and 1.0.

These bounds do not depend on $V_x$ values but only on n. Figure 13 show that, for this example with n = 10, the lower bound on Type A corresponds to $\theta/L = 0.05$ while the upper bound on Type B corresponds to $\theta/L = 0.85$. Since the average $\theta_v$ value is 1.35 m (see Table 10. Summary of scales of fluctuation from literature review by soil type of this document), this means that, for the conditions in this particular example, the Type A estimate only applies when the vertical extent of the failure volume exceeds 1.35/0.05 = 27 m, while the Type B estimate only applies when the vertical extent of the failure volume is less than 1.35/0.85 = 1.6 m. When the vertical extent of the failure volume lies between these bounds, then Type C applies, and it is necessary to calculate the magnitude of the characteristic value using Equation 18 and the relevant $\theta/L$ value. In practice, this will be the situation for many geotechnical designs, for example in the design of spread foundations, pile foundations, retaining wall and many slopes.

**Figure 13.** Bounds for the Type A, B and C estimates for $X_k/X_{mean}$ vs. θ/L for $Vx$ = 0.3 and $n$ = 10.




*Source: Developed by the authors*

Going further and considering the previous Figure 13 and the ideas behind it, the following 'rule of thumb', relative to the bounds on type of estimate, is proposed in this document for all design situations and types of ground, when on the scale of fluctuation is the weighted average value of 1.35 m for all soils:

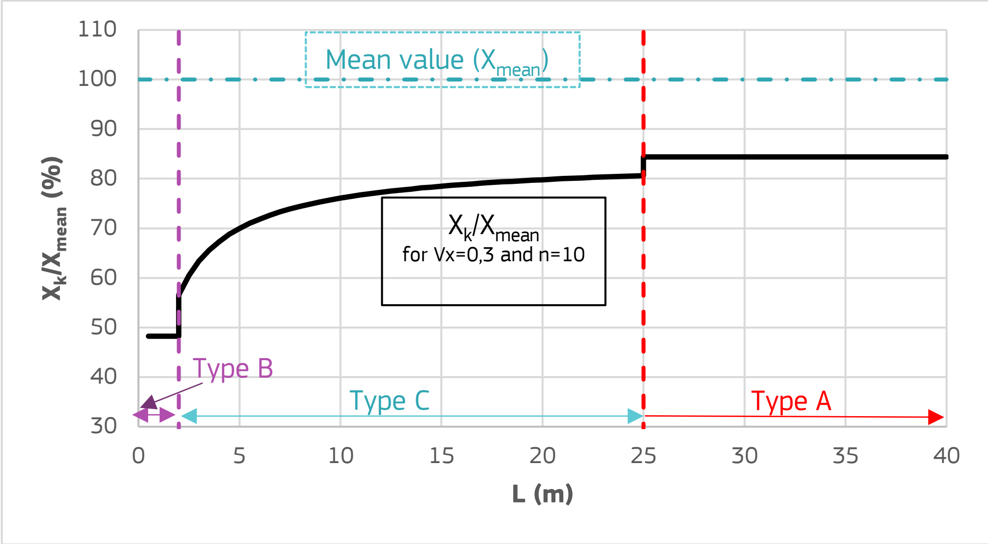
— Type A bound when $L \geq 25$ m

— Type B bound when $L \leq 2$ m

— Type C when 2m > $L$ < 25 m

Figure 14 shows these bounds and the corresponding ratio of the characteristic property value to the mean value in percent obtained for the design situation when $V_x$ = 0.3, $n$ = 10 and $\theta_v$ = 1.35m. For other values of $V_x$, n and $\theta_v$, Equation 19 must be used.

**Figure 14.** Characteristic value ratio ($X_k/X_{mean}$) for 'rule of thumb' type bounds.

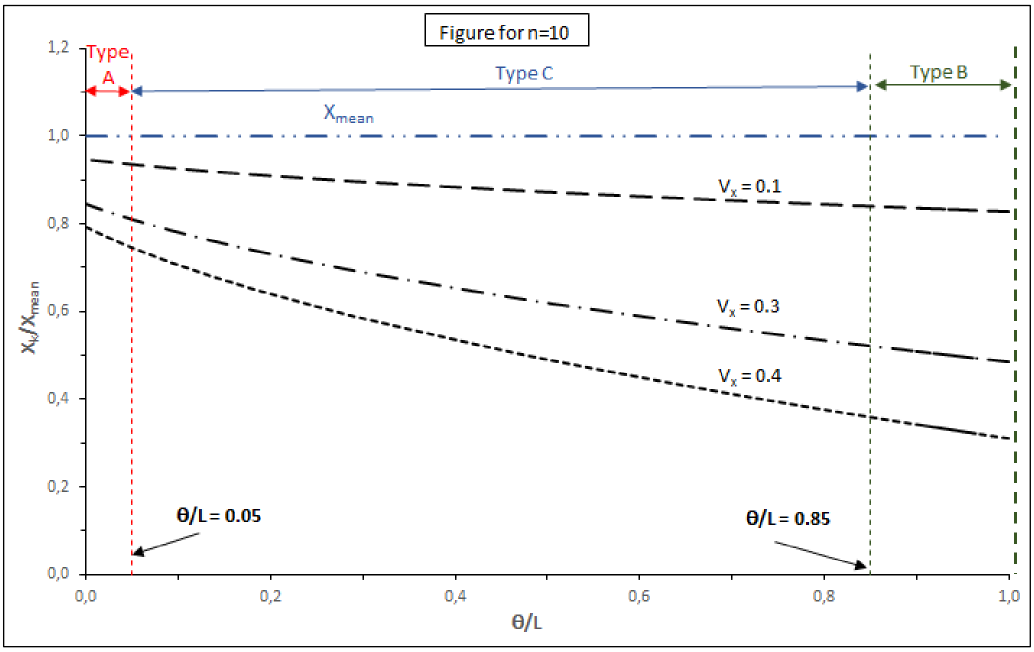



*Source: Developed by the authors*

### 4.5.4 Effect of $V_x$ on the characteristic value

The effect of the value of $V_x$ on the ratio $X_k/X_{mean}$ versus $\theta/L$ is shown in Figure 15 for values of $V_x$ = 0.1, 0.3 and 0.4 when n = 10. These graphs show that the value of $V_x$ has a large effect on the characteristic value, causing the characteristic value to decrease as $V_x$ increases. In this particular example, when $\theta/L$ = 0, as $V_x$ increases from 0.1 to 0.4, $X_k/X_{mean}$ decreases from 0.95 to 0.78. It should also be noted that for the same n value, in this case n = 10, the graphs' shapes remain the same so that the bounds on the Type A and B estimates occur at the same $\theta/L$ ratios.

**Figure 15.** Effect of $V_x$ on the characteristic value ratio ($X_k/X_{mean}$) when $n$ = 10.

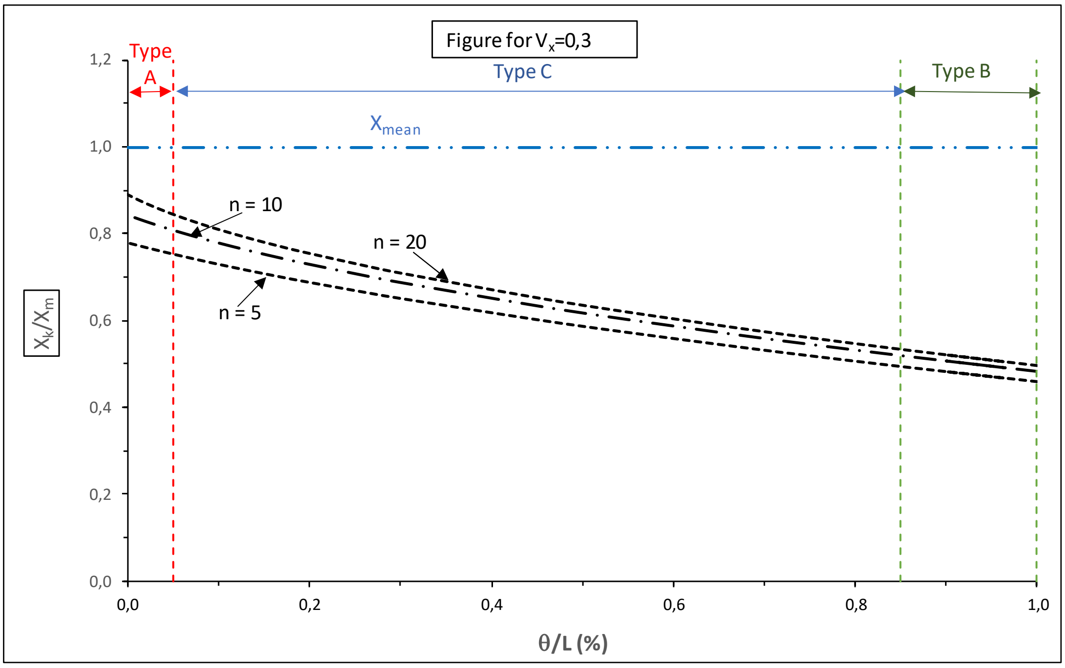



*Source: Developed by the authors*

### 4.5.5 Effect of the number of derived values (*n*) on the characteristic value

The effect of the number of derived values (n) on the ratio $X_k/X_{mean}$, versus $\theta/L$ is shown in Figure 16 for ground with $V_x$ = 0.3 and n = 5, 10 and 20. Comparison of the graphs in Figure 16 with those in Figure 15, shows that the effect of the number of derived values increasing from 5 to 20 on the calculated characteristic value is less significant than the effect of the value of $V_x$, reducing from 0.4 to 0.1. However, both have an effect that needs to be considered when calculating the characteristic value.

**Figure 16.** Effect of number of derived values (*n*) on the characteristic value ratio ($X_k/X_{mean}$) when $V_x$ = 0.3

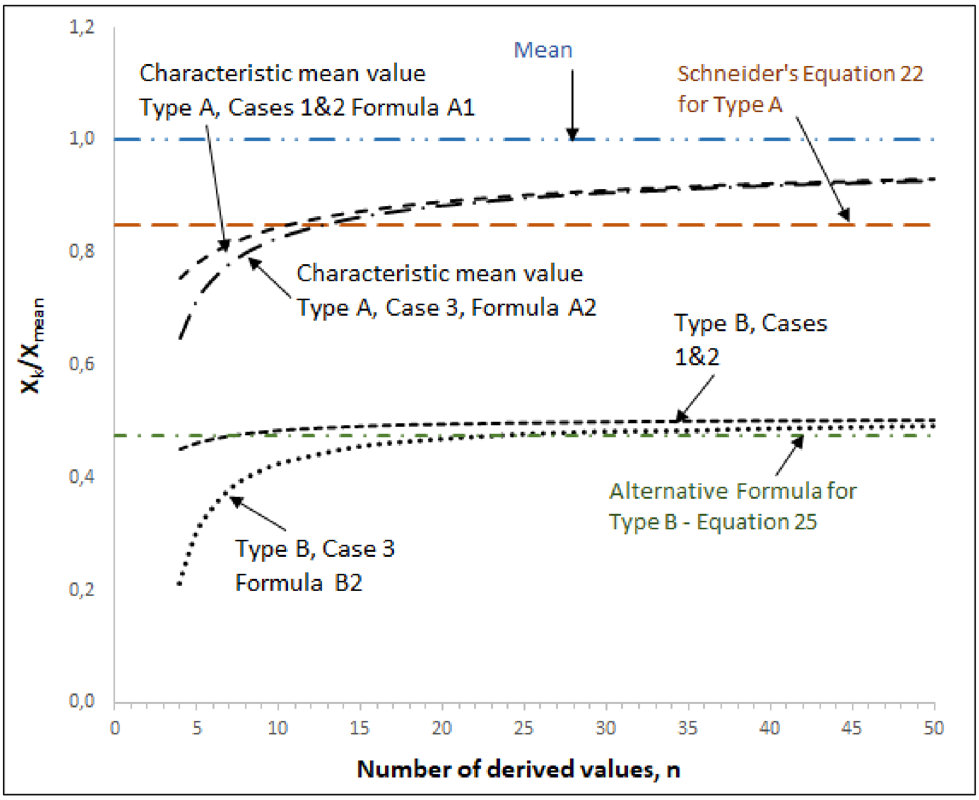



*Source: Developed by the authors*

The effect of the number of derived values on the ratio $X_k/X_{mean}$, is shown in Figure 17. This figure presents the results for ground with $V_x$ = 0.3. The ratio $X_k/X_{mean}$ is plotted against *n* using the formulae in Table 13 for the Types A and B estimates of the characteristic value, considering the different $V_x$ Cases, i.e. whether $V_x$ is known or assumed (Cases 1 or 3), or $V_x$ is unknown (Case 2).

These graphs show that the calculated characteristic value decreases at a significantly increasing rate as the number of derived values reduces below 10 in order to achieve the confidence level of 95% for both Type A and Type B estimates and for both Cases. For both the Type A and B estimates, the graph for $V_x$ unknown, i.e. Case 2, is lower than the graphs for $V_x$ known or assumed, i.e. Cases 1 and 3, but these graphs converge as n increases.

**Figure 17.** Effect on the characteristic value ratio ($X_k/X_{mean}$) of the number of derived values, n and whether $V_x$ is known, unknown or assumed when $V_x$ = 0.3




*Source: Developed by the authors*

### 4.5.6 Effect of the selection of the characteristic value on the confidence level

While the concept of a confidence level has not been included in EN 1990-1, the formulae for the characteristic value given in Table 13, partially taken from EN 1997-1/Annex A/Table A3, are based on a confidence level of 95% and using the Normal distribution $N_{95}$ value to calculate the characteristic value.

To examine the effect of selecting characteristic values with a different level of confidence, the graphs of $X_k/X_{mean}$ are plotted, in Figure 18, against $\theta/L$ for the example when $V_x$ = 0.3 and $n$ = 10 for:

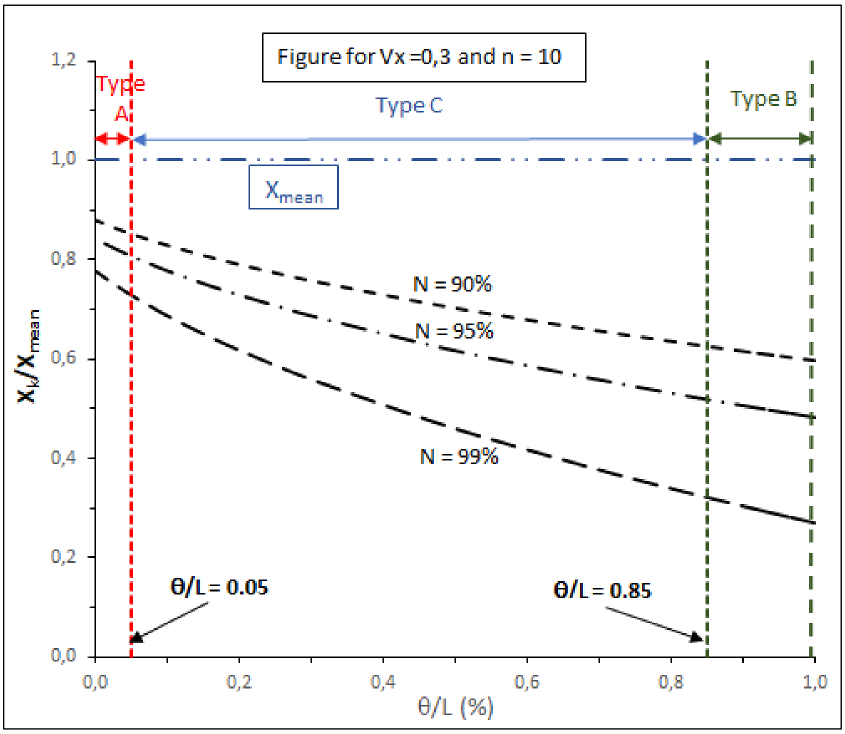
— a confidence level of 90%, obtained using $N_{90}$ = 1.28,

— a confidence level of 95%, obtained using $N_{95}$ = 1.64,

— a confidence level of 99%, obtained using $N_{99}$ = 1.96.

Comparison of these graphs shows that if a larger, i.e. more optimistic, characteristic value is selected for use in design calculations than the characteristic value obtained using the 95% confidence level graph, then the confidence level with be less than 95% and hence the design may not meet the reliability assumed by EN 1990-1. Alternatively, if a smaller, i.e. more conservative, characteristic value is selected for design than that obtained using the 95% confidence level graph, then the confidence level will be greater than 95% and the design may be uneconomical. This effect is most significant when selecting the characteristic value for designs when the spatial variability, given by $\theta/L$, is in the Type C range between $\theta/L$ = 0.05 and 0.85. For example, if a design situation is assumed to be Type A when it is within Type C, then the characteristic value will be overpredicted and the confidence will not have the confidence level assumed for geotechnical designs to EN 1997-1.

**Figure 18.** Effect of the selection of the characteristic value ratio ($X_k/X_{mean}$) on the confidence level for the example of ground with properties $V_x$ = 0.3 and $n$ = 10 samples




*Source: Developed by the authors*

### 4.5.7 Other statistical equations to calculate the characteristic value

According to EN 1997-1/4.3.2.2(5), other acceptable statistical procedures, instead of Equation 15, may be used to calculate the characteristic value. Two simple statistical formulae which have been proposed to calculate the characteristic values are presented in this sub-section.

A equation proposed by Schneider (1997) for the characteristic mean value, i.e. a Type A estimate, is that it is the mean value ($X_{mean}$) of $n$ derived values minus half the standard deviation ($\sigma_x$); i.e. it is given by:

**Equation 20.** Simplified formula to determine the characteristic mean value

$$X_k = X_{mean} - 0.5\sigma_x = X_{mean}\,(1 - 0.5\ V_x)$$

or expressed as $X_k/X_{mean}$:

**Equation 21.** Simplified formula to determine the ratio of the characteristic mean value to the mean value

$$\frac{X_k}{X_{mean}} = 1 - 0.5\ V_x$$

This equation is the equivalent to adopting $k_n$ = 0.5 in Equation 15. If it is assumed that the ground has a $V_X$ value of 0.3, then using Equation 21 gives the following simple formula for the characteristic mean value expressed as $X_k/X_{mean}$:

**Equation 22.** Simplified formula to determine the ratio of the characteristic mean value to the mean value for $V_x$=0.3

$$\frac{X_k}{X_{mean}} = 1 - (0.5).(0.3) = 0.85$$

A simple equation to calculate $X_k$ for when the characteristic inferior value is an estimate of the inferior value, i.e. a Type B estimate, is to assume that it is the mean value minus one and a three-quarter standard deviations (Equation 23). i.e.:

**Equation 23.** Simplified equation to determine the characteristic inferior value

$$X_k = X_{mean} - 1.75\sigma_x = X_{mean}\,(1 - 1.75\ V_X)$$

or expressed as $X_k/X_{mean}$:

**Equation 24.** Simplified equation to determine the ratio of the characteristic inferior value to the mean value

$$\frac{X_k}{X_{mean}} = 1 - 1.75V_x$$

This equation is the equivalent to assuming that $k_n$ is 1.75. Again, as an example, If the ground is assumed to have a $V_X$ value of 0.3, then using Equation 24 gives the following alternative formula for the estimate of characteristic inferior value:

**Equation 25.** Simplified equation to determine the ratio of the characteristic inferior value to the mean value for $V_x$=0.3

$$\frac{X_k}{X_{mean}} = 1 - (1.75).(0.3) = 0.475$$

Graphs of Schneider's Equation 22 for the estimate of the characteristic mean value and the alternative Equation 25 for the estimate of the characteristic fractile value for when $V_X$ = 0.3 are plotted in Figure 18. In this example, Schneider's Formula equals the values given by Formulae A1 and A2 (in Table 13) when $n$ = 14 and 11 respectively, while Equation 25 equals the values given by Formulae B1 and B2 (in Table 13) when $n$= 25 and 7, respectively. These show how using the Case 3 (unknown $V_X$ values) results in more conservative characteristic values.

### 4.5.8 Conclusion regarding the selection of the characteristic value

The conclusions drawn from the analyses in the previous sections regarding the selection of the characteristic value are:

— Due to the effect of spatial variability, in most design situations the characteristic value should be selected as a Type C estimate rather than a Type A or B estimate, which are extreme values and only should be used when the θ/L value is either less than 0.05, for Type A, or greater than 0/85, for Type B.

— Selecting characteristic values that are greater than the values predicted using Equation 15 will result in characteristic values with a confidence level less than 95% and hence the resulting design may not achieve the reliability assumed by EN 1990-1. Alternatively, selecting a characteristic value less than the values predicted using Equation 15 will result in characteristic values with a confidence level greater than 95% and hence the resulting design may not be economical. This is particularly significant for designs when Type C estimates are relevant.

— The characteristic value is more sensitive to changes in the $V_x$ value than changes in the number of derived samples used to calculate the characteristic value.

— The sensitivity of the characteristic value to the number of derived values increases greatly as the number decreases below 10. More guidance on the selection of the characteristic value in the case of a small number of derived values is given in Chapter 5 of this document.

— Two alternative simplified equations have been proposed to calculate the Type A and Type B estimate of the characteristic value, which have been found to be useful.

— These text and graphs presented in this section provide guidance on the uncertainty involved in the selection of the characteristic value, depending on the level of knowledge of $V_x$ and the number of derived value available, when using the different formulae. The decision on the selection of characteristic value should not only take account of the nature of the ground, and the number and quality of the derived values, but also the design situation and the Geotechnical Category of the geotechnical structure.

## 4.6 Recommended procedure to determine the representative value

The recommended procedure to determine the representative value making use of both experience or engineering judgement on the one hand, and statistical information obtained from the available data on the other hand, is:

1. Use statistics to obtain the ‘characteristic value’, assuming the available data are considered sufficient. This can be achieved very easily by implementing the Formulae in EN 1997-1/Annex A or the equations presented in Section 4.5 of this document. (e.g. in a calculation spread sheet).
2. Use engineering judgement and experience (e.g. in comparable design situations) to select the ‘nominal value’, as a cautious estimate of the representative value.
3. Compare both values (nominal and characteristic) and review them critically:
   - If both values are similar, the confidence in the selection of the value is increased;
   - If both values are (very) different, the designer should critically examine the potential reasons for the difference and reconsider the selection of the representative value.

The benefits of this procedure are that the representative value obtained is based on more information than that used when just the nominal or characteristic value is obtained in isolation and also with time will lead to learning and experience in the selection of representative values.

## 4.7 Design value

### 4.7.1 Use of design values

Figure 19 shows the three situations in which design values are used:

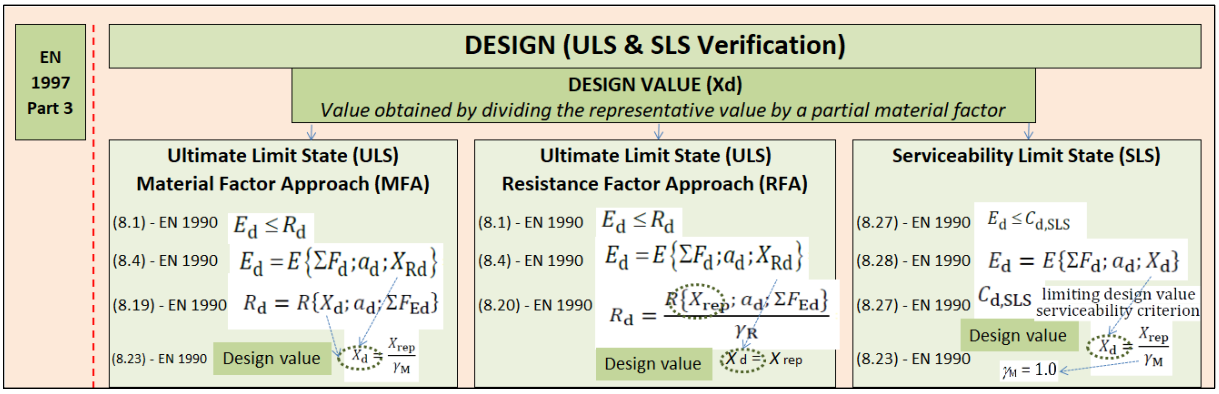
— In ULS verifications using the Material Factor Approach (MFA)

— In ULS verifications using the Resistance Factor Approach (RFA)

— In SLS verifications

These three situations are explained below.

**Figure 19.** Use of representative and design values in the limit state verifications.




*Source: Developed by the authors*

### 4.7.2 Design values determined using partial material factors

When using the Material Factor Approach (MFA), EN 1990-1/8.3.5.2(1) states that the design value of the resistance ($R_d$) should be determined using the following Equation 26:

**Equation 26.** Design values of geotechnical resistances $R_d$, for MFA calculations [Taken from EN 1990-1/Formula (8.19)].

$$R_d = R\{X_d; a_d; \Sigma F_{Ed}\}$$

where, for geotechnical designs, $X_d$ denotes the design values of the ground properties, $a_d$ denotes the design values of geometrical properties and $\Sigma F_{Ed}$ is the design value of actions used in the assessment of the design value of the effect of the actions ($E_d$).

According to EN 1997-1/4.4.1.3(3), for inferior values of $X_{rep}$, the design values of a ground property ($X_d$), should be calculated from Equation 27:

**Equation 27.** Design value of a ground property ($X_d$) for inferior values of $X_{rep}$ [Taken from EN 1990-1/Formula (4.6)].

$$X_d = X_{rep}/\gamma_M$$

i.e. the design value is obtained by dividing the representative value by the appropriate material partial factor. This is for when the ground property is acting favourable, which is the usual situation.

To provide the required level of reliability when the ground property is acting favourably, the representative value is the appropriate value that is less than the average value. When the superior value is needed, i.e. when the ground property is acting unfavourably, for example in the case of downdrag on a pile, according to EN 1997-1/4.4.1.3(4), the design value of a ground property is obtained by multiplying the representative value by the appropriate material partial factor (Equation 28):

**Equation 28.** Design value of a ground property ($X_d$) for superior values of $X_{rep}$ [Taken from EN 1990-1/Formula (4.7)].

$$X_d = \gamma_M X_{rep}$$

To provide the required level of reliability when the ground property is acting unfavourably, the representative value is the appropriate value that is greater than the average value.

Two sets, M1 and M2, of material partial factors ($\gamma_M$) on ground properties for persistent, transient and accidental design situations are given in EN 1997-1/Table 4.8. The default partial factors applied to the coefficient of peak friction ($\tan\varphi'_p$ ) and the peak effective cohesion ($c'_p$), for soil and fill for the partial factor set M2, are both 1,25. These partial factors are greater than those applied to the coefficient of friction at critical state ($\tan\varphi'_{cs}$), the coefficient of residual friction ($\tan\varphi'_r$) and the residual effective cohesion ($c'_r$), which are all 1,10. The values of the partial factor set M1 are all equal to 1.0. In addition to the sets of MFA partial material factors, M1 and M2, for soil and fill, separate sets of partial factors are provided in EN 1997-1/Table 4.8 for rock material and rock mass, rock discontinuities and ground-structure interfaces. These material partial factor values are NDPs, i.e. nationally determined parameters, and are default values which should be used unless the National Annex gives different values.

It should be noted that when using the MFA, partial factors are applied to the ground property values but no resistance factor is applied to calculated resistances. This is evident from the partial factor values in the tables in the clauses of EN 1997-3 on the design of different geotechnical structures. For example, in Clause 5: Spread foundations, EN 1997-3/Table 5.1 gives, under the MFA column, the sets of partial factors to be applied to ground properties and states that the bearing and sliding resistances are not factored.

### 4.7.3 Design resistance determined using a partial resistance factor

When using the Resistance Factor Approach (RFA), EN 1990-1/8.3.5.3(1) states that the design value of the resistance ($R_d$) is determined using the following Equation 29:

**Equation 29.** Design values of geotechnical resistances $R_d$, for RFA calculations [Taken from EN 1990-1/Formula (8.20)].

$$R_d = \frac{R\{X_{rep}; a_d; \Sigma F_{Ed}\}}{\gamma_R}$$

where, for geotechnical designs, $X_{rep}$ denotes the representative values of the ground properties, $\gamma_R$ is the partial resistance factor and $a_d$ and $\Sigma F_{Ed}$ are as defined above in 4.7.2. Thus, when using the RFA, representative, rather than design, values of ground properties are used to calculate the geotechnical resistance to which a partial resistance factor is applied to calculate the design resistance. The design resistance is used together with the design geometrical properties and the design values of the actions in assessing the Ultimate Limit State.

The fact that representative ground property values are used in geotechnical calculations of resistance when using the RFA and no partial material factors are applied to the material ground properties used to calculate the resistance is evident from the partial factor values in the tables in the clauses of EN 1997-3 on the design of different geotechnical structures. For example, in Clause 5: Spread foundations EN 1997-3/Table 5.2 gives, under the RFA column, the partial factors to be applied to the bearing and sliding resistances and states that the representative ground properties are not factored.

### 4.7.4 Design values in Serviceability Limit State verifications

EN 1990-1/8.4.2(1) states that when verifying serviceability limit states, the design value of the effect of actions ($E_d$) should be calculated by the following formula (Equation 30):

**Equation 30.** Design value of the effect of actions ($E_d$) [Taken from EN 1990-1/Formula (8.28)].

$$E_d = E\{F_d; a_d; X_d\}$$

where, $F_d$ is the design values of actions, $a_d$ is the design values of geometrical properties, and $X_d$ is the design values of the ground properties. Since for serviceability limit states all partial factors, e.g. $\gamma_F$ and $\gamma_M$, are equal to 1.0, the design ground properties are not factored but are equal to the representative values, i.e. $X_d = X_{rep}$. In serviceability limit state verifications, the design value of the effect of actions ($E_d$) is compared with the design value of the limiting serviceability criterion, $C_{d,SLS}$, e.g. the limiting settlement.

The required level of safety in serviceability limit state designs is achieved by selecting suitably cautious representative ground properties, for example ground stiffness and compressibility values. As in the case of Ultimate Limit State designs, the spatial variability of ground properties should be considered when selecting the appropriate representative values. When the serviceability limit state is not sensitive to the spatial variability, for example in the case of a deep homogeneous stratum of ground, a representative value close to the average value would be appropriate, like for the Type A ultimate limit state estimate. However, when the serviceability limit state is sensitive to the spatial variability, for example in the case of the settlement of a structure with multiple footings, as noted in the Case of the spread foundation example in Section 4.3.3.3.3 of this document, a more conservative combination of possible local representative values, similar to a Type C Ultimate Limit State estimate, may be appropriate.

### 4.7.5 Consequence and Reduction Factors

According to EN 1997-1/4.4.1.1(4) '*the consequences of failure should be taken into account by use of a consequence factor, $k_F$, $k_M$ or $k_R$*', while EN 1997-1/4.4.1.1(5) states that '*only one of the consequence factors shall be applied in a single verification*.' This means that the partial ground material and resistance factors ($\gamma_M$ and $\gamma_R$) may be multiplied by Consequence Factors $k_M$ and $k_R$, respectively to either reduce or increase the partial factors to account for the consequence of failure, if permitted in the National Annex; i.e. in the case of $\gamma_M$, (and similarly for $\gamma_R$).

**Equation 31.** Design value of a ground property ($X_d$) for inferior values of $X_{rep}$ accounting for the consequence of failure

$$X_d = X_{rep}/(\gamma_M \, . \, k_M)$$

The brackets around $\gamma_M$ $k_M$ in Equation 31 indicate its use is optional. The values of $k_M$ and $k_R$ are given in EN 1997-1/Table 4.9 and are 0,9 for Consequence Class 1 (CC1), 1,0 for Consequence Class 2 (CC2) and 1,1 for Consequence Class 3 (CC3).

According to EN 1997-1/4.4.1.3(7) the value of the partial material factor $\gamma_M$ for transient design situations may be multiplied by a reduction factor $k_{tr}$, provided the resulting partial factor is not less than 1,0 and any constraint on its use is satisfied, i.e. (Equation 32):

**Equation 32.** Design value of a ground property ($X_d$) for inferior values of $X_{rep}$ for transient design situations

$$X_d = X_{rep}/(\gamma_M.k_{tr})$$

The bracket around $\gamma_M$ $k_{tr}$ indicates its use is optional. According to 4.4.1.5(2) the value of the partial resistance factor $y_R$ may also be multiplied by a reduction factor $k_{tr} \leq 1{,}0$, so as to reduce the partial factor for transient design situations. As in the case of $y_M$, the multiplication of $\gamma_R$ by $k_{tr}$ is permitted, provided the resulting partial factor is not less than unity and any constraints on its use are satisfied.

A transient design situation is defined in EN 1990-1/3.1.2.4 as '*temporary conditions of use or exposure of the structure that are relevant during a period much shorter than the design service life of the structure' while the note to this definition states that 'A transient design situation refers to temporary conditions of the structure, of use, or exposure, e.g. during construction or repair*'. Use of the reduction factor can enable a more economical design for a temporary condition, such as during construction.

### 4.7.6 Directly determined design values

The first note to EN 1990-1/3.1.4.3 on the definition of the design value of a material or product property states that '*in special circumstances, the design value may be obtained by direct determination'*, i.e. determined by not applying a partial factor to a representative value. A second note to this paragraph states that other Eurocodes should be seen for specific rules. An important provision stated in EN 1990-1/8.1(3) is that design values may be determined directly provided the resulting level of reliability is no less than that required by EN 1990-1.

Adopting the first note of EN 1990-1/3.1.4.3 for geotechnical design, EN 1997-1/4.4 1.3(2) states that the design value of a ground property may be determined directly in accordance with EN 1990-1/8.3.6(2). This lists six methods how design values may be determined directly provided the level of reliability is no less than that implied by the use of Equation 27 in this document. Two methods are noted as being particularly relevant to geotechnical design; these are determining design values directly from:

— prescriptive rules; and

— the most unfavourable value that the property could practically adopt.

Other methods listed for determining design values directly that may be relevant to geotechnical design include:

— empirical or theoretical relations with measured physical properties;

— previous experience; and

— results of tests.

The note to EN 1990-1/8.3.6(2) states that guidance on the assessment of design values of ground properties is given in the relevant part of EN 1997 and that permission to use specific prescriptive rules is given in the relevant part of EN 1997. EN 1997-1/4.5(2) states that prescriptive rules shall be suitably conservative and justified by comparable experience. EN 1997-3/Clauses 4, 5, 6, 10 and 12 on the design of different geotechnical structures all have a sub-clause on verification by prescriptive rules. In the case of Clause 5 on the design of spread foundations, it is stated in the note to 5.6.3(1) that '*guidance on the use of the presumed bearing pressures can be given in the National Annexes*'. It should be noted that, when using presumed bearing pressures given in a National Annex, they are design values and hence a partial factor should not be applied to them.

Determining the design value of ground property directly in geotechnical design, rather than by applying a partial factor to a representative value, may be appropriate in order to select the most unfavourable value that the property could practically adopt. In this situation the worse credible value may be selected as the design value so as to preclude the design value determined by applying a partial factor to the representative value being so low or so high that it is not feasibly possible. In this situation the worst credible value would have to be selected so that the resulting level of reliability is no less than that required by EN 1990-1.

Another situation where a design value might be selected directly based on previous experience is in the case of a pre-existing failure surface when the design value could be determined from previous experience.

EN 1990-1/Annex D.7 provides an example of the direct assessment of design values from test results by giving a table with $k_n$ values, greater than those used to determine characteristic values, to be used with test results to determine design values directly rather than to determine characteristic values to which partial factors are then applied.

Another situation where it is explicitly stated in EN 1997-1 that design values may be determined directly is in 6.5.1(1) in connection with the determination of design groundwater pressures for Ultimate Limit State design. This paragraph states that design groundwater pressures may be determined by one of the following methods:

- direct assessment; or
- applying a deviation to the representative piezometric level or to the representative groundwater pressure; or
- applying a partial factor to the representative groundwater pressures or to their action effects.

Note that the first method listed above to determine the design value by direct assessment, is often adopted in geotechnical design verifications. The note to this paragraph states that methods involving direct assessment or application of a deviation to determine design geometric parameters are usually suitable in cases where groundwater pressures are used to calculate shear strength from effective stresses (e.g. overall stability analyses or retaining wall design). The application of partial factors is usually suitable in cases where groundwater pressures are used to calculate forces and bending moments on structural elements. Additionally EN 1997-1/6.5.1(2) provides a reminder that, '*when assessing design groundwater pressures directly or by applying a deviation to the representative piezometric level or groundwater pressure, the design values of groundwater pressures for ultimate limit states shall have a probability of exceedance as specified in EN 1990-1:2024*'.

## 4.8 Best estimate value

The best estimate value of a ground property is defined in EN 1997-1/3.1.3.6 as the "*estimate of the most probable value of a ground property*". Paragraph 4.3.2.4(1) further explains that the best estimate value is the value used to predict the most likely behaviour of a structure. It may be determined in one of the following three ways as the:

- 'Most probable value obtained from a sample of derived values of the considered ground property;
- Mean, median or mode of a sample of derived values, whichever is most appropriate; or
- Most probable value obtained by back-analysis carried out to model the performance of the structure by monitoring'.

The best estimate value of a ground property differs from the representative value in that it is not a conservative estimate of the ground property value selected with the appropriate level of confidence. Instead, it is an estimate of the value of the ground property that allows prediction of the most likely, i.e. anticipated, behaviour of the structure, and hence it does not provide any margin of safety.

While the best estimate of a ground property value is defined and explained in EN 1997-1, there are no references to best estimate ground property values in EN 1997-2 or to their use in geotechnical design in EN 1997-3. This is because in order to provide the required reliability or margin of safety required in both ultimate and serviceability limit state designs, design values of ground properties based on representative values must be used. However, best estimate values may be adopted when using the Observational Method to verify limit states in accordance with EN 1997-1/4.7(1). In this situation, they may be used to predict the actual behaviour of the structure. Best estimate values may also be used to verify designs through the monitoring of the actual behaviour, e.g. settlement, of structures during construction.

Another application of best estimate values is in the verification of correlations for ground property values when monitoring the behaviour of the ground or a structure in the ground; a typical example of this would be the back analysis of pile load tests wherein the best estimate value of the ground property must be used to obtain the best estimate assessment of the design method (the empirical link between ground properties and pile resistance). Thereafter the best estimate of the design method is adopted using the representative ground properties to obtain the representative pile resistance.

# 5 Special topics

## 5.1 The role of few samples

### 5.1.1 General ideas

This subsection outlines classical statistical methods to deal with situations in which there is a limited number of derived values. Furthermore, it should be noted that this section focuses mainly on the determination of characteristic values, as the available number of sample derived values particularly affects the quality of statistical considerations. Since in geotechnical engineering, designers often have prior knowledge, in some cases, Bayesian statistics may offer a more appropriate tool set (see European Commission: Joint Research Centre, Schweckendiek, Van den Eijnden et al., 2024 for more information on this topic).

By adopting all the measures set out in Eurocode 7, the level of reliability required by EN 1990-1/4.2 for geotechnical designs will be achieved. However, with an increasing number of sample derived values the level of confidence in the representative value increases. A safe, sustainable and economic design cannot be achieved if highly uncertain representative values are used in ultimate and serviceability limit state verifications. This consideration applies equally to the determination of the nominal as well as the characteristic values.

To ensure that the appropriate level of reliability required by EN 1990:2023, 4.2 is obtained, the Geotechnical Category (GC) should be called upon. The GC combines the uncertainty and complexity of the ground and ground-structure interaction with the consequence of failure of the structure (EN 1997-1/4.1.3.2 (1)). It should be determined from a combination of the Consequence Class of the structure (CC) and Geotechnical Complexity Class (GCC) (EN 1997-1/4.1.3.2 (3). The Consequence Class (CC) accounts for the consequence of failure of the structure. The Geotechnical Complexity Class (GCC), selected for a design situation, indicates to what extent the engineer should, among aspects, expect highly variable or difficult ground conditions associated with considerable uncertainty. Note that the GCC needs to be reviewed and, if appropriate, changed at each stage of design and execution.

The GC shall be used to specify, among aspects, the extent and amount of the measures to achieve appropriate representative property values for design, including an appropriate extent of the ground investigation (EN 1997-2:2024).

Additionally, using engineering judgement, a rough estimate on the required level of confidence in the representative value can be inferred from the GC, CC and GCC. It may range from the need for a highly accurate and precise estimate of the representative value as, e.g., in the case of critical infrastructure projects, to cases where a moderate level of confidence allows for a vague estimate of the representative value resulting in a less sustainable and less economic preliminary design of a geotechnical structure of low complexity.

According to 4.3.2.1 (6) the engineer should determine the representative value ($X_{rep}$) as characteristic value ($X_k$) 'when available data are considered sufficient to establish the characteristic value of a ground property', whereas 'when the available data are insufficient to establish the characteristic value of a ground property', the representative value ($X_{rep}$) should be determined as nominal value ($X_{nom}$). This recommendation is made because the quality of statistical evaluations depends, among aspects, on the statistical sample size. When the number of sample derived values is limited, it reduces the confidence in the determination of the characteristic value.

The increase of uncertainty due to a limited number of derived values is also reflected in Tables A.4 to A.7 in 1997-1/Annex A. The smaller the number of sample derived values, the larger the $k_n$ value which is multiplied by the coefficient of variation ($V_x$) to determine a characteristic value (see also Section 4.5.2 of this document), as seen in Table 14 that reproduces Table A.4, as an example. For a small number of sample derived values, $k_n$ increases to ensure the level of reliability required by EN 1990-1/4.2.

**Table 14.** Selected values of $N_{95}$ and $k_n$ to estimate the characteristic value as the mean value for 'V$_x$ assumed/known' [Combination A1], according to Formula in EN 1997, Annex A, Table A.3, Cell B2.

| ***n*** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** | **12** |
|---|---|---|---|---|---|---|---|---|---|---|
| $N_{95}$ | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 |
| $k_n$ | 1.16 | 0.95 | 0.82 | 0.74 | 0.67 | 0.62 | 0.58 | 0.55 | 0.52 | 0.47 |
| ***n*** | **14** | **16** | **18** | **20** | **25** | **30** | **35** | **40** | **50** | **100** |
| $N_{95}$ | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 | 1.64 |
| $k_n$ | 0.44 | 0.41 | 0.39 | 0.37 | 0.33 | 0.30 | 0.28 | 0.26 | 0.23 | 0.16 |

*Source: EN 1997-1*

The tables given EN 1997-1/Annex A distinguish four combinations of Cases of $V_x$ and Types of estimate, to determine a characteristic value (see also Table 10. Summary of scales of fluctuation from literature review by soil type in Section 4.5.2 of this document) which must be selected by the designers according to the requirements of their project:

— Case 1 and Case 3 are used when Vx is known or assumed;

— Case 2 is used when Vx is unknown;

— Type A estimates yield a cautious estimate of the mean of the sample derived values, and

— Type B estimates yield a much more conservative estimate, which is an estimate of the inferior or superior value towards a lower bound or upper bound to the sample derived values (see also Section 4.3.3.3.2 of this document).

The combinations with '$V_x$ known' or '$V_x$ assumed' (Case 1 / Case 3) require at least two sample derived values to obtain a characteristic value (EN 1990-1/Annex D, Table D.1). Case 2 requires at least three sample derived values to obtain characteristic values (EN 1990-1/Annex D, Table D.1). In all cases, a larger number of sample derived values may be required to achieve a sustainable and economic design. For an infinite number of sample derived values, the characteristic values obtained using the '$V_x$ assumed/known' and '$V_x$ unknown' approaches converge.

In addition, this document introduces a Type C estimate for a representative value that is between the average (Type A estimate) and the inferior or superior value (Type B estimate) (see Section 4.3.3.3.2 of this document). The guidance provided in this section focuses on Type A and Type B estimates. Yet, some of the herein proposed methods can be adapted to account for Type C estimates using the formulae given in Section 4.5.3 of this document. For this purpose, knowledge on the scale of fluctuation and the extent of the volume of ground involved in the limit state is required. In general, the required minimum number of sample derived values for Type C estimates ranges between the values for Type A and Type B estimates.

The authors of this document stress the importance of obtaining a statistically relevant number of sample derived values and of having a sound knowledge of the property and its dispersion to calculate the characteristic value. In the following sections, different approaches are presented that allow the minimum number of sample derived values required to estimate the characteristic value with an appropriate level of confidence for a particular combination of Case of $V_x$ and Type of estimate. The five selection criteria concerning the sample derived values are based on Equation 33 and summarized in Table 15. The presentation of the individual approaches does not include any prioritisation. Instead, the presented criteria are intended to show a range of possibilities that can be considered for the selection of the minimum number of sample derived values required.

**Equation 33.** Determination of characteristic values [Taken from EN 1997-1/Formula (4.5)].

$$X_k = X_{mean}[1 \mp k_n V_X] = X_{mean}\left[1 \mp \frac{k_n s_x}{X_{mean}}\right]$$

where the symbols have been explained below Equation 15 of this document.

**Table 15.** Overview of minimum number n of the sample derived values to estimate the characteristic value for the different combinations and selection criteria.

| **Selection criterion** | **Formula** | **Comb. A1** | **Comb. B1** | **Comb. A2** | **Comb. B2** |
|---|---|---|---|---|---|
| Minimum number of sample derived values based on the confidence interval (see 5.1.2) | $n = \left(\frac{N_{95}\,\sigma}{w}\right)^2$ | calculation required | calculation required | -- | -- |
| Non-negative characteristic values (see 5.1.3) (1) | $n = (N_{95}\,V_x)^2$ | no lower limit (2) | no lower limit (2) to 3 | no lower limit (2) to 3 | no lower limit (2) to 10 |
| Minimum number of sample derived values based on characteristic value ratios (see 5.1.4) (1) | $\frac{X_k\,(for\ n = n_i)}{X_k\,(for\ n = 100)} < 20\%$ | no lower limit (2) to 10 | 3 - 12 | no lower limit (2) to 12 | 3 - 30 |
| Minimum number of sample derived values based on the gradient (see 5.1.4) (1) | $\frac{\left(\frac{X_{k,i}}{X_{mean}} - \frac{X_{k,j}}{X_{mean}}\right)}{n_i - n_j} < 1\%$ | 3 - 12 | 5 - 14 | 2 – 5 | 5 - 12 |
| Minimum number of sample derived values based on the accuracy a (see 5.1.5) (3) | $k_{n,req} = a \cdot k_n$ | no lower limit (2) | 3 | 3 – 11 | 11 - 33 |
| Minimum number of sample derived values based on other standards (see 5.1.6) | 7 – 11 | | | | |

1 Note: for $V_x$ ranging between 5% and 50%.

2 Note: Not a number – no strict limit was found; engineering judgement and general statistical considerations should be accounted for.

3 Note: accuracy of $V_x$ indicated by a coefficient of variation of $k_n$ ranging between 5% and 15%.

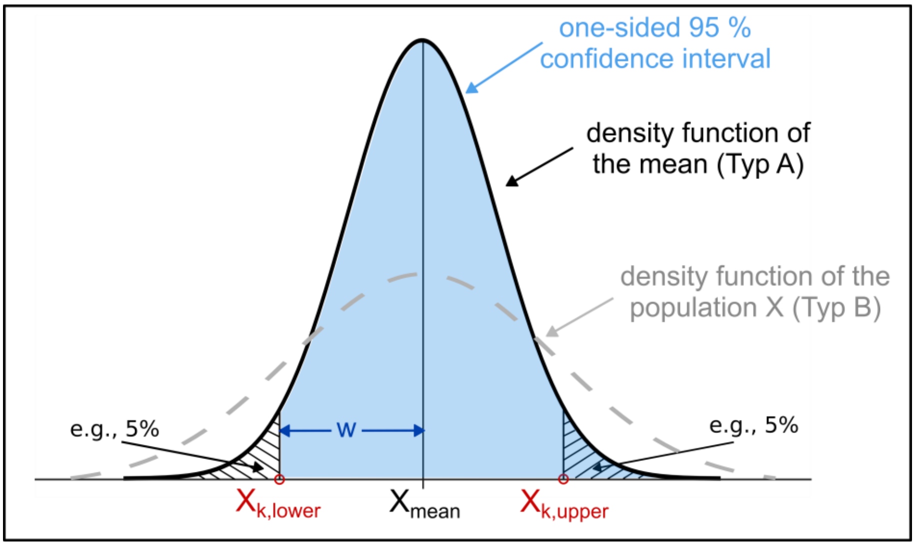
*Source: Developed by the authors*

Finally, it may be noted there is no unique way to select a minimum number of sample derived values that covers all aspects of the design of different geotechnical structures and design stages. The minimum number of required sample derived values may even vary throughout the different project phases. Ultimately, it is therefore the designers' choice to select an appropriate number of sample derived values for their project.

### 5.1.2 Minimum number of sample derived values based on the confidence interval

For a Type A estimate (estimate of the mean value), the characteristics value is, from a purely statistical point of view, the upper or lower end of the one-sided confidence interval of the mean estimate, with a confidence level of 95%, as shown in Figure 20.

**Figure 20.** Illustration of half width and confidence interval for the calculation of a minimum number of required samples




*Source: Developed by the authors*

Based on this definition, the minimum number of derived values can be determined taking into account the half of the width of such confidence interval (*w*), as shown in Equation 34 and Equation 35. The length of the half interval width (*w*) can be considered as the margin of error associated with the estimate of the mean. For instance, to determine the lower characteristic mean value of the undrained shear strength $c_u$ within $\pm$5 kPa, it should be assumed that $w$ = 5 kPa. It is a common practice to express the half interval width (*w*) in terms of a number of standard deviations on either side of the mean (for instance, $w = 1/2\ \sigma$).

**Equation 34.** Determination of minimum number n of the sample derived values for Vx known or assumed (Combination A1 in Table 13)

$$X_k = X_m(1 - k_n\, V_x\, N_{95}) = X_m - \frac{N_{95}\,\sigma}{\sqrt{n}}$$

$$X_k - X_m = \frac{N_{95}\,\sigma}{\sqrt{n}} = w \Rightarrow n = \left(\frac{N_{95}\,\sigma}{w}\right)^2 = \left(\frac{1.64\,\sigma}{w}\right)^2$$

**Equation 35.** Determination of minimum number n of the sample derived values for Vx unknown (Combination A2 in Table 13)

$$X_k = X_m\left(1 - k_n\, V_x t_{(95,n-1)}\right) = X_m - \frac{t_{(95,n-1)}\, s}{\sqrt{n}}$$

$$X_k - X_m = \frac{t_{(95,n-1)}\, s}{\sqrt{n}} = w \Rightarrow n = \left(\frac{t_{(95,n-1)}\, s}{w}\right)^2$$

where:

| | |
|---|---|
| $N_{95}$ | is the value of Gaussian distribution at a confidence level $\alpha = 0.95$; |
| $s$ | is the standard deviation determined from the sample derived values; |
| $t_{(95,n-1)}$ | is the value of the student-t distribution at a confidence level $\alpha = 0.95$ and *(n - 1)* degrees of freedom; |
| $w$ | is the length of the half interval width; commonly given by the number of standard deviations on either side of the mean; |
| $\sigma$ | is the known or assumed standard deviation. |

Note that the Combination A2 estimate requires an iterative procedure as the critical value of the Student-t distribution depends on the number of available sample derived values. For the analysis of a Lognormal distribution, the original data can be transferred into Normal space.

For Type B and Type C estimates, the required minimum number of samples must be evaluated using more advanced methods such as bootstrapping (Efron, 1982; Efron and Tibshirani, 1993) which are beyond the scope of this document.

### 5.1.3 Minimum number of sample derived values based on non-negative characteristic values

A mathematical limitation of the formulae A.2 to A.7 for the characteristic value $X_k$ provided in EN 1997-1/Annex A is that, if a Normal distribution is assumed, negative or very low characteristic values can be obtained in the case of high $V_x$. Thus, to ensure the mathematical applicability of the formulae, some limits for the required number of samples can be postulated to avoid the characteristic mean $X_{k,mean}$ or the characteristic fractile values $X_{k,5\,\%}$ or $X_{k,95\,\%}$ become negative. Based on this consideration, the minimum number of samples to ensure non-negative characteristic values with an appropriate level of confidence depending on $V_x$ is provided in Table 16 from Equation 36. Note that the postulated limits derived from the non-negative number criterion should be evaluated cautiously and combined with other criteria presented in this document.

**Equation 36.** Determination of minimum number n of the sample derived values to avoid $X_k$ being a negative value

$$X_k = X_m(1 - k_n\, V_x\, N_{95}) = X_m\left(1 - \frac{N_{95}\, V_x}{\sqrt{n}}\right) = 0$$

$$X_k = 0 \Rightarrow 0 = \left(1 - \frac{N_{95}\, V_x}{\sqrt{n}}\right) \Rightarrow n = (N_{95}\, V_x)^2$$

**Table 16.** Minimum number n of the sample derived values to ensure non-negative characteristic values depending on $V_x$.

| $V_x$ | 5% | 10% | 20% | 30% | 40% | 50% |
|---|---|---|---|---|---|---|
| Min. number of sample derived values (Combination A1) | - (1) | - (1) | - (1) | - (1) | - (1) | - (1) |
| Min. number of sample derived values (Combination B1) | - (1) | - (1) | - (1) | - (1) | - (1) | 3 |
| Min. number of sample derived values (Combination A2) | - (1) | - (1) | - (1) | 3 (2) | 3 (2) | 3 (2) |
| Min. number of sample derived values (Combination B2) | - (1) | - (1) | 3 (2) | 4 (2) | 5 (2) | 10 (2) |

1 Note: No limits can be postulated, as even for a small number of samples the formulae do not yield negative values.

2 Note Postulated limits should be evaluated cautiously and always combined with other criteria presented in this document.

*Source: Developed by the authors*

For Combination A1 this consideration does not result in a minimum required number of samples because, even for a small number of samples, the formulae do not yield negative values. Yet, it is stressed that the formulae still perform less accurately with a low number of sample derived values (see Sections 5.1.4 – 5.1.7 of this document). Furthermore, even if the low characteristic values obtained by a statistical analysis are not used for design, their pure mathematical existence should be taken as an indication to assess whether the available data satisfy a Normal distribution. If they do not, the designer may consider using a Lognormal distribution with an appropriately adjusted $V_x$.

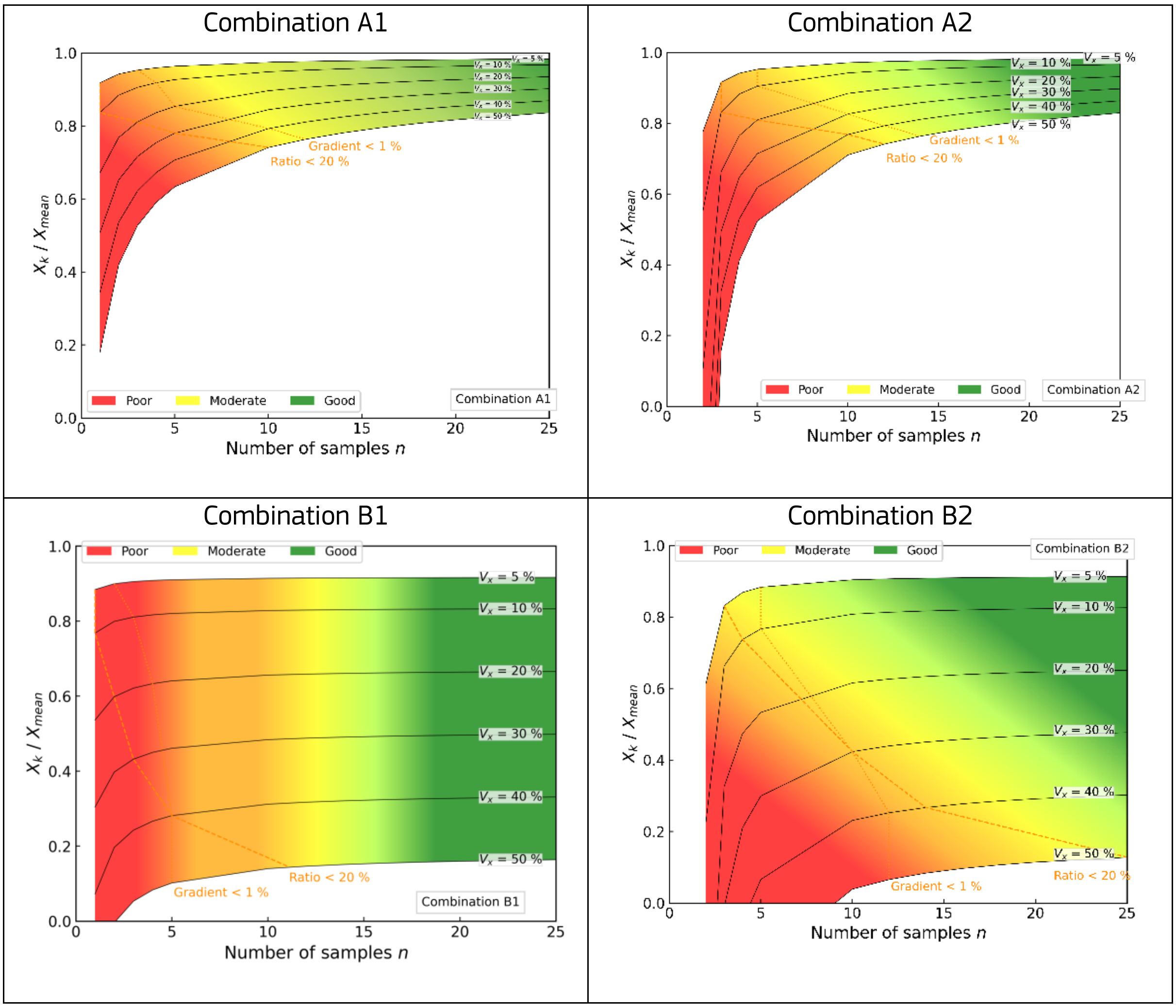
### 5.1.4 Minimum number of sample derived values based on $X_k/X_{mean}$ ratios and $V_x$ values

This subsection provides guidance on the minimum required number of samples to estimate the characteristic value with an appropriate confidence level as a function of the ratio $X_k/X_{mean}$ and of the $V_x$ value, for the different Combinations of $V_x$ cases and Types of estimate of the characteristic values.

For this purpose, Figure 21, that provides some qualitative guidance, should be interpreted as follows:

- In the greenish-coloured areas, the number of samples is considered sufficient to determine a characteristic value.
- In the yellow-coloured areas, the determined characteristic value should be used more cautiously than in the greenish coloured area and cross-checked, for instance, by comparison with the nominal value and engineering judgement.
- In the red coloured area, it is recommended to increase the number of sample derived values or, if that is not possible, statistics are not used to determine the representative value but it is obtained as a nominal value. A low number of sample derived values is only applicable if either the variability of the ground or the required accuracy of the characteristic value is low, again, provided consideration is given to the fact–that an engineering structure should not only be designed safely, but also sustainably.

**Figure 21.** Qualitative guidance on the minimum required number of samples to estimate the characteristic value for the different Combinations of $V_x$ cases and Types of estimate.




*Source: Developed by the authors*

In Figure 21, there are two criteria to estimate the minimum number of derived values:

— The ratio-based criterion (dashed lines): for this criterion, it is assumed that the estimate of the characteristic value obtained with a limited number of sample-derived values should be within the range of 20 % of the estimate obtained with 100 sample-derived values, as shown in Equation 37. This means that the determined characteristic value may be up to 20 % higher or lower than the characteristic value obtained with $n$=100.

**Equation 37.** Determination of ratio-based criterion.

$$\frac{X_k\ (for\ n = n_i)}{X_k\ (for\ n = 100)} < r;\ r = 20\%\ (Recomended\ value)$$

— The gradient-based criterion (dotted lines): for this criterion, it is assumed that the ratio $(X_k/X_{mean})/n$ (which is the gradient of the curve) should not exceed 1%, as shown in Equation 38. This means that additional sample derived values will not lead to a large change in the characteristic value. The determined characteristic value is close to the actual characteristic value. For some projects, the deviations chosen in this document may be too small or too large. The same procedure can thus be followed with other percentages, selected by the designers.

**Equation 38.** Determination of gradient-based criterion.

$$\frac{\left(\frac{X_{k,i}}{X_{mean}} - \frac{X_{k,j}}{X_{mean}}\right)}{n_i - n_j} < g; g = 1\% \ (Recommended\ value)$$

Comparing the results of the ratio-based and gradient-based analyses, it can be noted that the gradient-based criterion is stricter for Type A estimates as they are characterized by a steeper gradient. For Type B estimates, the 20% ratio of the $X_k\ /X_{mean}$ requires more sample derived values, since Type B estimates convergence more slowly when increasing *n*. It is the designers' choice to select the appropriate criterion for their project.

Finally, it is noted that the better the knowledge of $V_x$, the more applicable are the formulae in EN 1997-1/Annex A for use with a limited number of sample derived values. Moreover, larger uncertainty or less accurate estimates may be acceptable in preliminary design stages compared to the requirements of the final design.

### 5.1.5 Minimum number of sample derived values based on the accuracy of the $V_x$ estimate

There may be cases where the designer may not only want to account for the statistical uncertainty resulting from a limited number of sample derived values, but also for the uncertainty resulting from the limited knowledge on $V_x$. The less confidence concerning the value of $V_x$, the more sample derived values are required to reach the required level of reliability.

For this purpose, Table 17 collects the minimum required number of sample derived values that allows three different levels of accuracy of $V_x$ to be achieved. An indicator of the level of accuracy of $V_x$ is obtained by increasing $k_n$ = 1.645, obtained for 100 sample derived values, by a margin of safety of either 5%, 10% or 15%. The thereby obtained $k_{n,req}$ is compared with Tables A.4 - A.7 in EN 1997-1/Annex A to get the minimum required number of sample derived values. Note that the different levels of accuracy of $V_x$ do not represent a specific uncertainty associated with $V_x$.

**Table 17.** Minimum number n of the sample derived values depending on $k_n$ = 1.645 and the accuracy-of the $V_x$ estimate.

| **Level of accuracy of $V_x$** | $k_{n,req}$ | **Minimum number of required sample derived values** | | | |
|---|---|---|---|---|---|
| | | **Comb. A1 (Table A.4)** | **Comb. A2 (Table A.5)** | **Comb. B1 (Table A.6)** | **Comb.B2 (Table A.7)** |
| 5% | 1.72 | -- | 3 | 11 | 33 |
| 10% | 1.80 | -- | 3 | 5 | 17 |
| 15% | 1.89 | -- | 3 | 3 | 11 |

*Source: Developed by the authors*

Note that for Type B estimates, where $V_x$ is unknown, the formulae provided in EN 1997-1, Table A.3 already account for the uncertainty inherent to $V_x$.

The herein described procedure can also be applied to other $k_n$ values in those cases where the number of sample derived values is too small to achieve $k_n$ = 1.645, but the designer still prefers to account for the uncertainty associated with $V_x$.

### 5.1.6 Minimum number of sample derived values based on other standards

The following standards and studies state the minimum number of derived values that should be considered for design purposes:

— EN 1998-1 provides statistical procedures to obtain characteristic values for seismic loads from a small set of sample derived values (e.g., nonlinear time histories). There is a minimum of three sample derived values required to find a minimum or maximum. To determine a mean value at least 7 sample derived values should be available.

— More advanced research by Haselton (2014) recommended a set of 11 sample derived values but this recommendation has not yet been included in design codes.

— For wind engineering (e.g., Cooke & Mayne, 1979) usually 10 sample derived values are used to obtain the characteristic value.

### 5.1.7 Example: Settlements of an embankment on clay peat – Property: vertical hydraulic conductivity $k_v$ (Case 4)

#### 5.1.7.1 *Provided results from ground investigation*

The provided results from ground investigation are given in Table 18.

**Table 18.** Provided results from ground investigation – Case 4.

| **Test** | **#Data points** | **Comment** |
|---|---|---|
| Dissipation test | 9 | For CPTU it is generally assumed that dissipation is dictated by horizontal seepage, and, thus, returns $k_h$ in horizontal layered ground. In that case, a $k_h/k_v$ conversion factor is needed, which could vary significantly. |

*Source: Developed by the authors.*

#### 5.1.7.2 *Evaluation*

A simple test for a sufficient number of sample derived values may be performed by subdividing the samples in two groups. It is assumed that $V_x$ is unknown (Combinations A2 / B2).

An example taken from Case 4, with 6 initial derived hydraulic conductivity values from dissipation tests (numbered 3 to 11), subdivided into two groups with 4 samples each, is analysed based on a lognormal distribution.

Table 19 show the characteristic values determined with the whole set of data (6 data) and also with the two subsets with only four samples each, selected arbitrarily by the sequence numbers (shown in the second and third rows).

**Table 19.** Effect of the number n of the characteristic inferior (mean and fractiles) values.

| **Used data** | ***n*** | ***$k_n$* (A2)** | ***$k_n$* (B2)** | ***$X_{mean}$*** (1) | ***$V_Y$*** (2) | **$X_{k\ (mean)}$** | **$X_{k\ (fract\ 5\%)}$** | **$X_{k\ (fract\ 95\%)}$** |
|---|---|---|---|---|---|---|---|---|
| 3 to 8 | 6 | 0.82 | 2.18 | 1.11E-08 | 4.6% | 5.60E-09 | 1.80E-09 | 6.86E-08 |
| 3 to 6 | 4 | 1.18 | 2.63 | 6.20E-09 | 2.3% | 3.70E-09 | 1.96E-09 | 1.96E-08 |
| 5 to 8 | 4 | 1.18 | 2.63 | 1.54E-08 | 4.7% | 5.71E-09 | 1.69E-09 | 1.40E-07 |

1 Note: X = $k_v$ (Vertical hydraulic conductivity).

2 Note: Small variability values due to the use of lognormal distribution

*Source: Developed by the authors.*

The analysis of Table 19 makes it possible the followingobservations:

- The characteristic mean values ($X_{k\ (mean)}$) show a somehow expected relation. Depending on the selected data a value slightly smaller or larger than the average of the complete data set is obtained.
- The characteristic 5% fractile values ($X_{k\ (fractile\ 5\%)}$) of the two subsets are quite similar, whereas for the 95% fractile values ($X_{k\ (fractile\ 95\%)}$) a significant difference between the subsets and the total data set is observed.
- The $X_{k\ (fractile\ 95\%)}$ values obtained with the reduced data set are by factor of 0.3 to 2.0 smaller or larger than the $X_{k\ (fractile\ 95\%)}$ value of the total data set.

## 5.2 Properties increasing with depth or stress

### 5.2.1 General ideas

Many geotechnical ground properties vary with confining pressure and, hence, show a correlation with depth. Of particular importance for design in relation to this are, among others:

- shear strength, e.g., undrained shear strength[7];
- deformation properties, e.g., stiffness modulus, compression modulus, Over Consolidation Ratio (OCR);
- dynamic properties, e.g., shear wave velocity, $G_{max}$

For depth or stress-dependent ground properties, the regular statistical procedures cannot be used. Either a trend reduction should be conducted before using the data or, if the designer would like to include the depth-dependency, the subsequently outlined methods should be used.

Note that the approaches presented in this section can also be applied to Type C estimates. For this purpose, the effect of spatial averaging should be accounted for in the selection or calculation of $V_x$. However, this requires a rather advanced knowledge on the characterisation of uncertainties inherent to ground properties whose explanation is beyond the scope of this document (see

[7] Effective friction angle φ' and effective cohesion c' are considered as dependent ground properties (see Section 5.3 of this document).

European Commission: Joint Research Centre, Schweckendiek, Van den Eijnden et al., 2024 for more information on this topic).

To deal with depth-dependency, it is necessary that *n* observations of pairs of depth - ground property ($z_i$, $x_i$) are available from a ground investigation. These can be either field tests at different depths, e.g., field vane tests or CPT data, or laboratory tests at different effective stress levels, e.g., triaxial or shear tests.

The most common approach is to assume that the variation of the ground property *X* over depth *z* can be modelled by a linear function[8]. The variation of *X* over *z* is then expressed as set in Equation 39:

**Equation 39** Variation of the ground property over depth

$$X(z) = a_0 + a_1 z + \varepsilon$$

where:

| | |
|---|---|
| $a_0$ | is the intercept of the mean of *X;* |
| $a_1$ | is the depth gradient of the mean of *X*; |
| $\varepsilon$ | is a measure for the variability of the ground. |

The coefficients $a_0$ and $a_1$ in the linear expression can be estimated using least squares (Equation 40):

**Equation 40.** Coefficients of the linear expression of the variation of the ground property over depth

$$a_1 = \frac{\sum_{i=1}^{n}(z_i - \bar{z})(x_i - \bar{x})}{\sum_{i=1}^{n}(z_i - \bar{z})^2} a_0 = \bar{x} - a_1\bar{z}$$

where:

| | |
|---|---|
| $x_i, z_i$ | are the values of the $i^{th}$-sample pair of derived value; |
| $\bar{x}, \bar{z}$ | are the mean values of *x* and *z*; |
| *n* | is the number of sample derived values used for the evaluation *X.* |

### 5.2.2 Nominal value of the depth-dependent trend

In case of the nominal value trend, a statistical description of uncertainties is replaced by expert knowledge. The initial regression that is calculated with, e.g., an ordinary least square (OLS) regression represents the depth-dependent mean. The 'nominal line' should be lowered (or increased, depending on the cases) cautiously. Thus, to determine the nominal value the intercept $a_0$

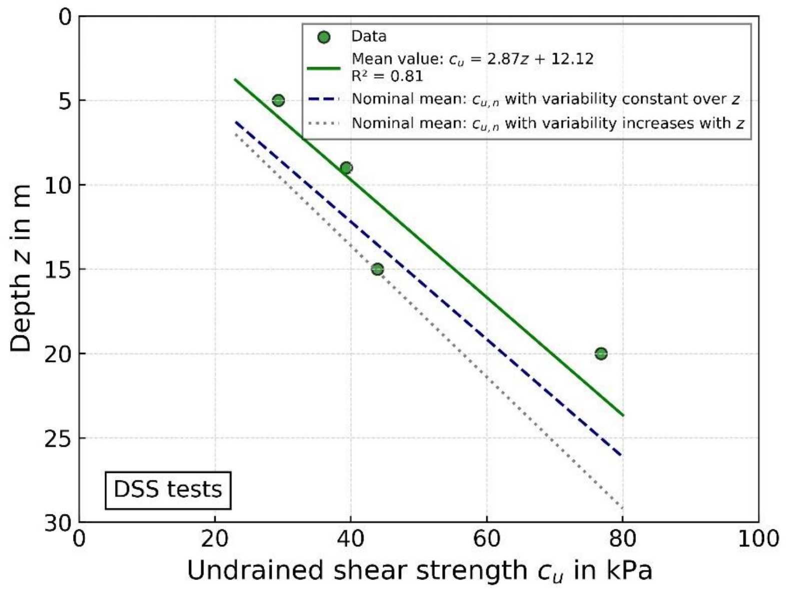
[8] In principle, the herein presented procedures can also be adapted to other trend functions, based perhaps on geological and geotechnical knowledge of the site. Furthermore, it should be noted that ground properties do not always exhibit a linear trend. Examples of other trend functions may be found in Ang and Tang (2007).

should be lowered (or increased) cautiously depending on the property and the limit state function (see Figure 22).

The degree of reduction or increase in the nominal value line should reflect the local experience of the designer, the variability of the test results and the number of available samples. Consideration should also be given to the failure mechanism, which may contribute to an averaging of ground properties. Results obtained with the adjusted nominal regression line at a certain depth may be validated based on experience or literature values. In the case of particularly thick geotechnical units, i.e. layers, a division into smaller 'sub-units' or 'sub-layers' may be beneficial in order to realistically represent the ground conditions in-situ.

Note that a change in the gradient of the linear regression corresponds to a change in variability of the ground property *X* with depth, e.g., when the gradient decreases with depth, the dispersion about the mean of the parameter *X* increases (see Figure 22). The linear regression line 'widens' with depth.

**Figure 22.** Schematic illustration for the definition of nominal values for depth-dependent ground properties.




Note: The solid line represents the average over depth that should be carefully lowered (or increased) using engineering judgement to obtain the 'nominal line' (dashed and dotted)

*Source: Developed by the authors*

### 5.2.3 Characteristic values of the depth-dependent trend

A purely statistical approach to cover uncertainty inherent to depth dependent ground properties is outlined by Bond & Harris (2008). The statistical procedures described in Eurocode 7 for a single variable are replaced by multi-variate statistics. The characteristic value trends can be determined as follows (Equation 41 and Equation 42):

**Equation 41.** Determination of the characteristic mean value trend.

$$X_k(z) = \bar{x} + a_1(z - \bar{z}) \pm t_{95,(n-2)} \cdot se \sqrt{\left[\frac{1}{n} + \frac{(z-\bar{z})^2}{\sum_{i=1}^{n}(z_i - \bar{z})^2}\right]}$$

**Equation 42.** Determination of the characteristic 5% or 95% fractile value trend.

$$X_k(z) = \bar{x} + a_1(z - \bar{z}) \pm t_{95,(n-2)}.se\sqrt{\left[1 + \frac{1}{n} + \frac{(z-\bar{z})^2}{\sum_{i=1}^{n}(z_i - \bar{z})^2}\right]}$$

where:

| | |
|---|---|
| $t_{95,(n-2)}$ | is Student's t-value for (*n* - 2) degrees of freedom at the 95% confidence level (see Table 20); |
| *se* | is the residual standard deviation, given by Equation 43 |

**Equation 43.** Determination of the residual standard deviation.

$$se = \sqrt{\frac{1}{n-2}\sum_{i=1}^{n}[(x_i - \bar{x}) - a_1(z_i - \bar{z})]^2}$$

**Table 20.** Selected values of $t_{95,(n-2)}$[9] to estimate characteristic values.

| ***n*** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** | **12** |
|---|---|---|---|---|---|---|---|---|---|
| $t_{95,(n-2)}$ | 6.31 | 2.92 | 2.35 | 2.13 | 2.02 | 1.94 | 1.90 | 1.86 | 1.81 |
| ***n*** | **14** | **16** | **18** | **20** | **25** | **30** | **35** | **50** | **100** |
| $t_{95,(n-2)}$ | 1.78 | 1.76 | 1.75 | 1.73 | 1.71 | 1.70 | 1.69 | 1.68 | 1.66 |

*Source: Developed by the authors*

As a result of the (*n*-2) degrees of freedom, the calculated characteristic value trends follow a hyperbolic function. The difference between the linear regression line and the hyperbolic function representing the characteristic value trend is smallest at the centre of gravity of the measurement data. However, both functions diverge towards the beginning and the end of the data Therefore, the function for the characteristic values must be linearized in the area of interest (Bauduin, 2001). To avoid this problem, Bond (2011) recommends a simple procedure with the following steps:

1. Determination of a (linear) trend line X (z), see Equation 39 and Equation 40.
2. Determination of the deviation between the trend line and each data point (residuals)
3. Calculation of the residual standard deviation se
4. Determination of the statistical coefficient kn (see EN 1997-1/Tables A.4 and 5) with $t_{95,(n-1)}$
5. Plotting the representative line using the expression $X(z) = a_0 + a_1 z \pm k_n \cdot se$

[9] In contrast to uncorrelated variables, the degrees of freedom are given by (n-2) as a result of two correlated variables.

For this kind of analyses, it is not necessary to distinguish between normally distributed or lognormally distributed ground properties. However, the above-described procedures assume that the variance is constant over depth; that the scatter in the data is random (i.e., there is no systematic influence affecting the data points), and that the residuals fit a Gaussian distribution. All these three assumptions should be checked during the procedure. If they are not fulfilled, the assumptions for an ordinary least square (OLS) regression are violated. In this case, the results of the linear regression should be analysed carefully.

### 5.2.4 Advanced applications

If the distribution of the residuals (and, thus, the likely distribution of the response) is heavily skewed, *X* can be transformed to log(*X*) or ln(*X*). The transformed data can then be used for OLS regression and the determination of the characteristic 5% and 95% fractile trends given by the prediction intervals in Equation 42. Subsequently, the regression results must be transformed back to the original scale by exponentiating the obtained end points.

To obtain the characteristic mean value trend, the transformation from lognormal into normal space requires a more elaborate procedure. Since the transformed mean will not be the mean of the original distribution, in fact, it will be the median, simply exponentiating the confidence intervals of the mean obtained with the transformed data (naïve approach) gives the confidence intervals of the median. This may be sufficient if the mean and median do not differ significantly. If a more exact solution is required, the one-sided 5% and 95% confidence intervals (CI) of the characteristic mean of the natural logarithm-transformed data can be approximated by Equation 44 to 46 (Angus, 1988)[10]:

**Equation 44.** Determination of the one-sided 5% confidence interval of the characteristic mean of the natural logarithm-transformed data

$$CI_{5\,\%} = \exp\left( \overline{Y_{ln}(z)} + \frac{\sigma_{ln}^2}{2} - \frac{t_{95,(n-1)}}{\sqrt{n}} \sqrt{\sigma_{ln}^2 \left(1 + \frac{\sigma_{ln}^2}{2}\right)} \right)$$

**Equation 45.** Determination of the one-sided 95% confidence interval of the characteristic mean of the natural logarithm-transformed data

$$CI_{95\,\%} = \exp\left( \overline{Y_{ln}(z)} + \frac{\sigma_{ln}^2}{2} + \frac{q_{95}}{\sqrt{n}} \sqrt{\sigma_{ln}^2 \left(1 + \frac{\sigma_{ln}^2}{2}\right)} \right)$$

**Equation 46.** Expression of the variable $q_{95\%}$

$$q_{95} = \sqrt{\frac{n}{2}\left(\frac{n-1}{\chi_{95,(n-1)}^2} - 1\right)}$$

where:

[10] There are several other approaches to estimate the confidence intervals for ln-transformed data which are not discussed in this document, e.g., Cox's method (Land, 1972) or Zhou & Gao (1997). An exact solution to determine confidence intervals for lognormal means is provided in Land (1971, 1975). However, the applicability of Land's approach is diminished by a poor accessibility of required tabulated values.

| | |
|---|---|
| $\overline{Y_{ln}}$ | is the mean of the transformed values at depth *z;* |
| $\sigma_{ln}$ | is the standard deviation of the transformed sample; |
| $t_{95,(n-1)}$ | is Student's t-value for (*n* - 1) degrees of freedom at the 95% confidence level (see EN 1997-1/Table A.7); |
| $\chi^2_{95\ ,(n-1)}$ | is the Chi-squared distribution for (*n* - 1) degrees of freedom at the 95% confidence level (see Table 21 of this document). |

**Table 21.** Selected values of $X^2_{95\%\ (n-1)}$ to estimate characteristic mean values with a lognormal distribution.

| ***n*** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** |
|---|---|---|---|---|---|---|---|---|---|
| $\chi^2_{(95,(n-1)}$ | 0.004 | 0.10 | 0.35 | 0.71 | 1.15 | 1.64 | 2.17 | 2.73 | 3.33 |
| ***n*** | **12** | **14** | **16** | **18** | **20** | **25** | **30** | **50** | **100** |
| $\chi^2_{95,(n-1)}$ | 4.58 | 5.89 | 7.26 | 8.67 | 10.12 | 13.85 | 17.71 | 33.93 | 77.05 |

*Source: Developed by the authors*

From a practical engineering point of view, the use of original data should be favoured over transformed data for linear regression. It must be noted that using a logarithmic transformation can be quite problematic and should be applied carefully. There are a number of limitations the designer must be mindful about. For instance, the transformed data cannot easily facilitate inferences concerning the original data, e.g., mean and variance. Before applying a logarithmic transformation, the designer might consider using more advanced regression methods such as a Generalised Linear Model (GLM), Weighted Least Squares (WLS) or the Maximum Likelihood Method (MLM). However, this is likely to require the use of more advanced statistical expertise.

Furthermore, the variance of *X* must be conditional on a given value of *z.* In Equation 41 and Equation 42 it is assumed that this conditional variance is constant with depth *z* and an unbiased estimate of the variance can be obtained from the sample derived values. In the case of a non-constant or unknown variance, an extension of Equations 41 and 42 is required.

For a conservative simplification (Mašín, 2015), it may be assumed that the measurement errors can be neglected when the field and laboratory tests are carried out according to standards such as CEN or ISO standards (Schneider & Fitz, 2011; Schneider & Schneider, 2013). It is also argued that by using established transformation models, the transformation uncertainty can be reduced to negligible values (Schneider & Fitz, 2011; Schneider & Schneider, 2013). In order to take the above uncertainties explicitly into account, more advanced, yet not state-of-the-art methods may be applicable. (see European Commission: Joint Research Centre, Schweckendiek, Van den Eijnden et al., 2024 for more information on this topic).

### 5.2.5 Example: Undrained slope stability – Property: undrained shear strength $c_u$ (Case 2)

#### 5.2.5.1 *Provided results from ground investigation*

The provided approaches to determine the depth or stress-dependent parameters are illustrated using the example of Case 2 (undrained shear strength $c_u$), detailed in Table 12. For reasons of simplicity, the depth-dependency of $c_u$ is determined only using the results of the fall cone tests and direct simple shear tests (DSS), as shown in **Error! Reference source not found.**. Case 2 provides additionally test results from fall cone tests and CPTUs. Guidance on how to work with data from different sources is given in Section 5.4 of this document.

**Table 22.** Example Case 2 – Provided data.

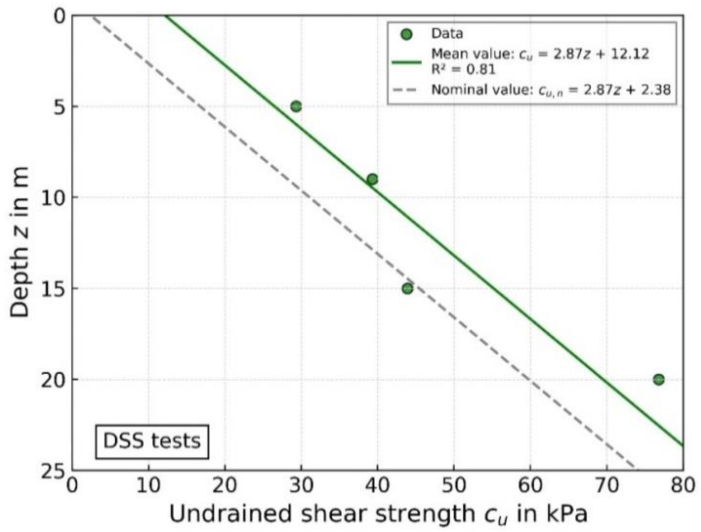
| Test | #Data points | Comment |
|---|---|---|
| Fall cone | 19 | Corrected values, unconfined test. Subjected to sample disturbance, given for depths between 5 m and 27 m |
| Direct simple shear tests (DSS) | 4 | Limited number of available samples, but most reliable test procedure |

*Source: Developed by the authors.*

### 5.2.5.2 *Evaluation*

Based on the formulae outlined in this document, the depth- or stress-dependent linear formulae are determined. One may decrease the depth-dependent $c_u$ given by the initial linear regression and provide nominal values for the area of interest using engineering judgement. In addition, the standard deviation of the regression error may be used as guidance for the definition of nominal values (see Figure 23).

**Figure 23.** Nominal values for depth-dependent undrained shear strength $c_u$.



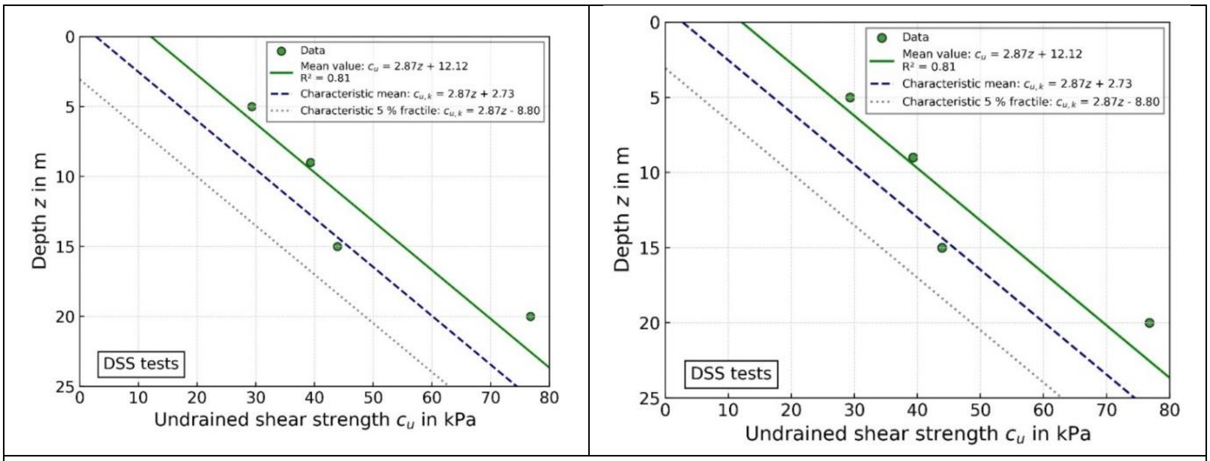

Note: the solid line represents the depth-dependent mean, the dashed line the 'nominal line'.

*Source: Developed by the authors.*

For the determination of depth- or stress-dependent characteristic values, the approaches according to Bond & Harris (2008), and Bond (2011) are solved for Case 2. Generally, the 5% or 95% fractiles are used if the limit state is governed by a small volume of soil (local failure), whereas the mean values are used when an averaging of soil properties is expected, e.g., as in Case 2. Depending on the quality and quantity of the existing information about the ground, e.g., from the desk study, the coefficient of variation can be assumed to be known, assumed or unknown.

As is common in statistics, the coefficient of determination, denoted by $R^2$, can be used as a measure of how well the variation in the dependent variable $X$ is explained by the independent variable $z$. The larger $R^2$, the better the regression model. Furthermore, the smaller the number of samples or the larger the variability, the greater the distance of the regression line giving the characteristic values from the original regression line. As shown for the example of the 5% fractile of the DSS test results with unknown coefficient of variation, this may even result in physically implausible results in terms of a negative undrained shear strength close to the surface. Thus, at a depth ranging between 0.0 m and 4.5 m, a characteristic 5% fractile value $c_u$=0 kPa would have to be assumed when a locale failure is expected (see Figure 24).

**Figure 24.** Characteristic values for depth-dependent undrained shear strength $c_u$ using the approach by Bond & Harris (2008).




Note: the solid line represents the depth-dependent mean, the dashed lines show the 'characteristic lines'.

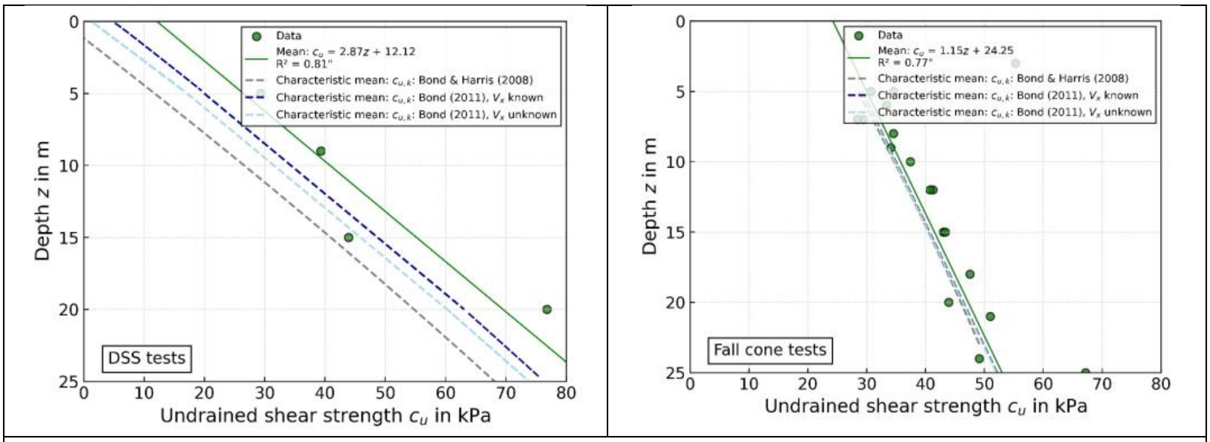
*Source: Developed by the authors.*

When using the Bond & Harris (2008) approach the difference between known and unknown coefficients of variation is not actively distinguished, whereas the simplified approach by Bond (2011) makes this distinction through different $k_n$–values. The approach by Bond (2011) assumes an unknown coefficient of variation.

A comparison of the different approaches shows that, especially when dealing with small samples, the approach by Bond & Harris (2008) is the most conservative (see Figure 25). Consequently, when using one of the proposed approaches, the engineer should be aware that the different approaches lead to different results, and the approach should be adapted to the required safety requirements. Especially in the case of few samples, the results of the different approaches can differ significantly. It may therefore be beneficial to compare the characteristic value with the nominal value (see Section 5.1 of this document).

Moreover, as explained before, the characteristic values that are based on the approach by Bond & Harris (2008) follow a hyperbolic function. The difference between the linear regression line and the function for the characteristic values is smallest at the arithmetic mean depth of the measurement data. However, both functions diverge towards shallow and deep depths. Therefore, the function for the characteristic values must be linearized in the area of interest. This can be done, for example, in a simplified way by an additional linear regression through the determined characteristic values.

**Figure 25.** Characteristic mean values for mean of the depth-dependent undrained shear strength $c_u$ with three approaches descripted herein using the data of the Direct Simple Shear (DSS) and fall cone tests.




Note: Formulae for each regression are omitted in favour of clarity; the solid line represents the depth-dependent mean, the dashed lines show the ‘characteristic mean lines’

*Source: Developed by the authors*

## 5.3 Dependent properties

### 5.3.1 General ideas

A clear definition of ‘dependent properties’ is not available in the current version of the Eurocodes. EN 1997-2/8.1.2 solely notes that the Mohr-Coulomb parameters for soil mechanics (effective cohesion *c*’ and coefficient of effective friction angle tan *φ*’) and the Hoek-Brown parameters for rock mechanics (the dimensionless empirical constants *m* and *s*) are mutual dependent parameters as they cannot be determined independently of each other.

In the case of such a dependency of ground properties, the statistical procedures that are described in EN 1997-1/Annex A which are developed for a single variable must be replaced by multi-variate statistics (Bond & Harris, 2008; DNV, 2021). Dependent ground properties can then be treated similarly to depth-dependent ground properties by linear regression. The variation of a ground property *X* over another ground property *Y* is then expressed as follows (Equation 47):

**Equation 47.** Linear expression for dependent properties

$$X(y) = a_0 + a_1 y + \varepsilon$$

where:

| | |
|---|---|
| $a_0$ | is the intercept of the mean of *X;* |
| $a_1$ | is the *y*-dependent gradient of the mean of *X*; |
| $\varepsilon$ | is a measure for the variability of the soil. |

The coefficients $a_0$ and $a_1$ in the linear expression can be estimated using least squares (Equation 48):

**Equation 48.** Coefficients of the linear expression for dependent properties

$$a_1 = \frac{\sum_{i=1}^{n}(y_i - \bar{y})(x_i - \bar{x})}{\sum_{i=1}^{n}(y_i - \bar{y})^2}$$

$$a_0 = \bar{x} - a_1\bar{y}$$

where:

| | |
|---|---|
| $x_i, y_i$ | are the values of the *i^th^*-sample derived value; |
| $\bar{x}, \bar{y}$ | are the mean values of *x* and *y*; |
| *n* | is the number of sample derived values used for the evaluation *X*. |

The characteristic mean and fractile values are then obtained as follows (Equation 49 and Equation 50):

**Equation 49.** Determination of the characteristic mean value expression for dependent properties

$$X_k(y) = \bar{x} + a_1(z - \bar{y}) \pm t_{(n-2)}^{95\,\%} \cdot se\sqrt{\left[\frac{1}{n} + \frac{(y-\bar{y})^2}{\sum_{i=1}^{n}(y_i - \bar{y})^2}\right]}$$

**Equation 50.** Determination of the characteristic 5% or 95% fractile value expression for dependent properties

$$X_k(y) = \bar{x} + a_1(y - \bar{y}) \pm t_{(n-2)}^{95\,\%} \cdot se\sqrt{\left[1 + \frac{1}{n} + \frac{(y-\bar{y})^2}{\sum_{i=1}^{n}(y_i - \bar{y})^2}\right]}$$

where:

| | |
|---|---|
| $t_{95,(n-2)}$ | is Student's t-value for (*n* - 2) degrees of freedom at the 95% confidence level (see Table 20) |
| *se* | is the residual standard deviation, given by Equation 51 |

**Equation 51.** Determination of the residual standard deviation

$$se = \sqrt{\frac{1}{n-2}\sum_{i=1}^{n}[(x_i - \bar{x}) - a_1(z_i - \bar{z})]^2}$$

Note that the procedures described in Section 5.2 of this document for the use of logarithmic transformed data are not applicable to the dependent ground properties φ' and c'. The input data for the determination of φ' and c' should not be transferred to logarithmic space considering the assumption of linearity of the untransformed data in the Mohr-Coulomb failure criterion. Moreover, many geotechnical properties are correlated, or one may also argue that they are to a smaller or larger extend dependent. Their dependency may be analysed by the correlation coefficient *ρ* that is used to measure the strength of linear association between two variables *X, Y* (Equation 52):

**Equation 52.** Determination of the correlation coefficient $\rho$

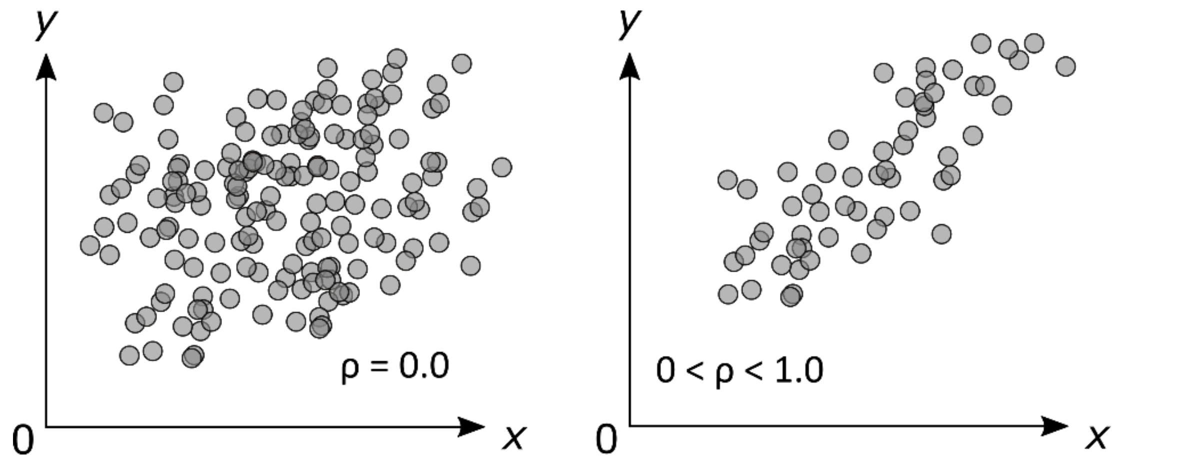
$$\rho(x,y) = \frac{Cov(X,Y)}{\sigma_X\,\sigma_Y}$$

where:

| | |
|---|---|
| $Cov(X, Y)$ | is the covariance of the variables *X* and *Y* |
| $\sigma_X; \sigma_Y$ | are the standard deviations of the variables *X* and *Y* |

Figure 26 illustrates the dependency or correlation of two variables schematically. This type of dependency or correlation is further addressed in the third Guideline of this series (European Commission: Joint Research Centre, Schweckendiek, Van den Eijnden et al., 2024).

**Figure 26.** Schematic illustration of two dependent or correlated variables




Note: when correlation coefficient ρ is close to 0, no dependency can be assumed, whereas increasing ρ indicates low to strong dependency or correlation

*Source: Developed by the authors*

### 5.3.2 Example: Stability of a slope – Properties: cohesion and friction angle (Case 1)

In Case 1 (see Annex 1 of this document) the data (vertical stress and shear stress) of six direct shear tests with each three subsamples are used to make a linear approximation of the 95 % confidence interval for the mean values. As c' and φ are dependent properties, the 95 % confidence assessment is calculated with multi-variate statistics.

#### 5.3.2.1 *Provided results from ground investigation*

The provided results from ground investigation are given in Table 23.

**Table 23.** Provided results from ground investigation – Case 1.

| **Test** | **#Data points** | **Comment** |
|---|---|---|
| Direct shear | 18 | The sample derived values (effective cohesion / phi) for each of the six separate tests are already provided, but for this assessment, the provided vertical stress and shear stress measured for each subsample is the starting point. |

*Source: Developed by the authors.*

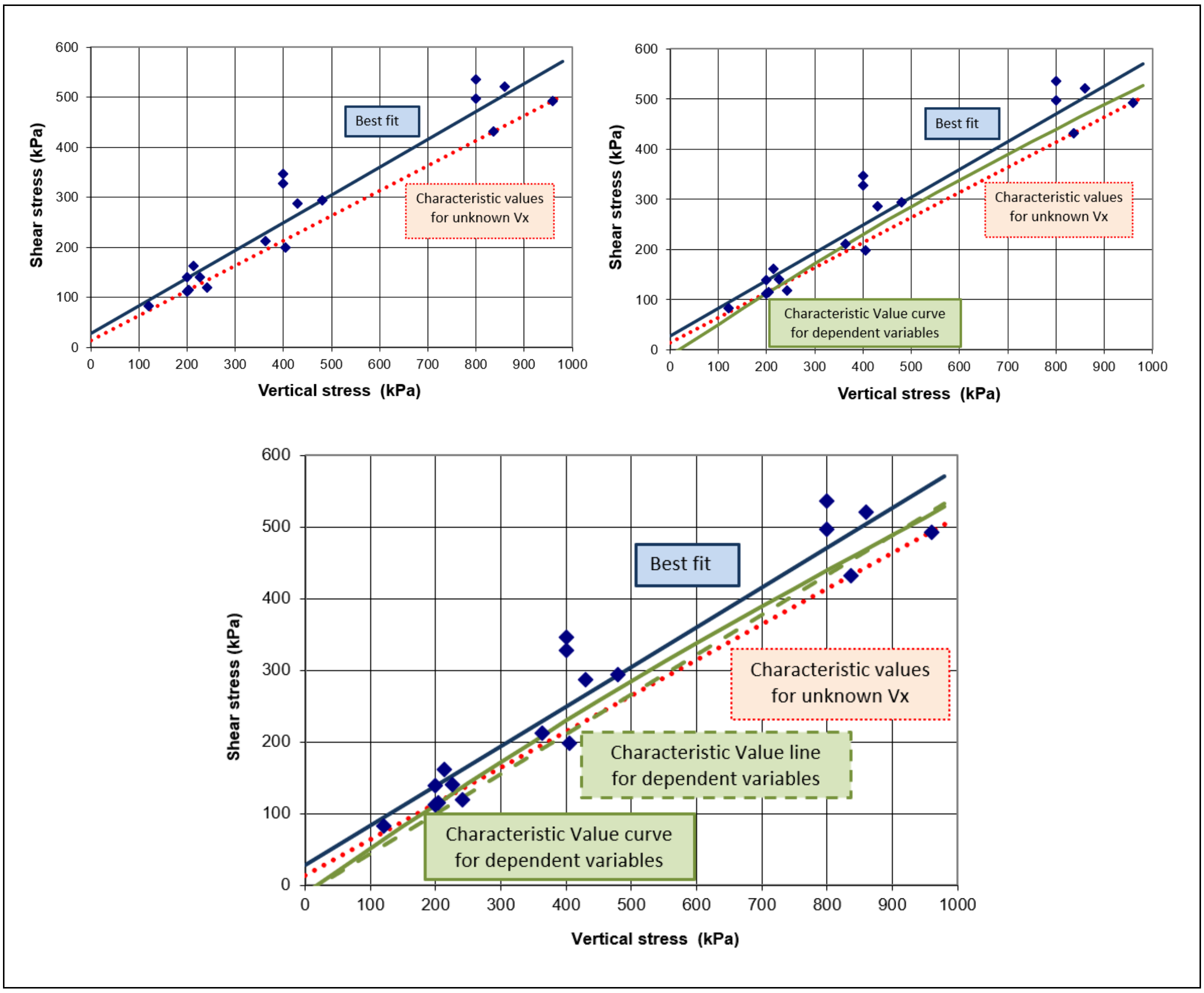
### 5.3.2.2 *Evaluation*

In this case, the Mohr-Coulomb parameters (effective cohesion and friction angle) are considered as dependent properties since they are correlated through the Mohr-Coulomb failure criteria. So, the expressions given in Section 5.3 of this document are used to determine the characteristic mean Mohr-Coulomb parameters. Since some of those expressions lead to a curve, not a line, as shown below with the green curve and line, a further linearization was done.

Figure 27 shows the failure points in a normal-shear stress plot together with:

- Blue continuous line representing the Mohr-Coulomb failure criteria as the line that best fits the failure points.
- Red dotted line representing the Mohr-Coulomb failure criteria with the characteristic values determined considering independent values and $V_x$ unknown (obtained in previous section).
- Green continuous curve representing the characteristic curve (note that this is not a line, it is a hyperbolic function, so it is difficult to manage and to compare with the other results)
- Green dashed and dotted line representing the characteristic line from which the characteristic values can be deduced.

**Figure 27.** Failure points together with characteristic Mohr–Coulomb failure criteria lines.




*Source: Developed by the authors*

Table 24 collects the Mohr-Coulomb parameters determined by the different procedures used to facilitate their comparison.

**Table 24.** Characteristic mean values obtained when considering dependent data.

| **Procedure** | **Effective Cohesion (kPa)** | **Effective Friction angle (º)** |
|---|---|---|
| Independent properties ($V_x$ known) | 21.82 | 26.86 |
| Independent properties ($V_x$ unknown) | 14.0 | 26.6 |
| Dependent properties (Best fit = "Mean value") | 27.8 | 29.0 |
| Dependent properties (Characteristic line) | -11.3 ([1]) | 29.0 |

[1] Note: the effective cohesion value corresponding to the characteristic mean line for dependent parameters is negative; the data must be re-evaluated with c' = 0.

*Source: Developed by the authors.*

## 5.4 Method for data from different sources

### 5.4.1 General ideas

In practice, the nominal or characteristic value of a ground property is often derived from data from different sources, mainly different test methods but also different measurement campaigns or laboratories. In these cases, the data from different data sets must be combined to one single representative value. For this purpose, there are several applicable approaches, and it is up to the designers to decide which approach is the most suitable for their own project depending, among other aspects, on the uncertainty due to the quantity and quality of site-specific data and to the spatial variability of the measured property, as mentioned in EN 1997-1/4.3.2.1 (2).

In the following, three approaches for determining representative values from multiple data sources are described. This document focuses on methods that can be easily applied in practice.

More advanced approaches to multivariate data analysis, such as Bayesian parameter estimation, can be found in the literature (e.g., Müller et al., 2014) and in the third Guideline of this series (European Commission: Joint Research Centre, Schweckendiek, Van den Eijnden et al., 2024). It is furthermore stressed that with increasing digitalisation and the emergence of data-centric geotechnics (Phoon & Ching, 2021; Phoon & Zhang, 2022), machine learning tools may also find their way into geotechnical practice, especially for complex problems. These methods may provide better solutions for the selection of representative values from multiple data sources.

### 5.4.2 Data review and preparation

If the representative value is to be chosen as a characteristic or a nominal value, it should be based on the amount and the quality of data in the established data set, and the use of comparable experience. There is often an inadequate number of datapoints available from certain sources, particularly high-quality laboratory testing (which are more expensive), resulting in the need to select a/some nominal value(s) instead of (a) characteristic value(s).

Additionally, in some cases, individual data points or a whole test series should be discarded from the data set if problems during the test performance are identified. Data observations that are far below or above the majority of data in a data set, are called outliers. Outliers can occur for a number of reasons, for example due to changes in system behaviour, fraudulent behaviour, human error, instrument error or simply through natural deviations in populations. Outliers should be carefully considered and, if considered not reflective of the actual ground behaviour, then omitted before carrying out a statistical analysis of a data set. As a rule of thumb and if there is no reason to disregard it a priori, any data value which is located more than two standard deviations away from the mean value of the data set can be considered as an outlier (DNV, 2021).

### 5.4.3 Weighting

Independently of the selected approaches to determine representative values, the weightings to the particular test methods selected for the evaluation could be related to the applicability of the test methods, given in EN 1997-2/Tables B.2 & B.4 (fragments of those tables are shown in Figure 28 and Figure 29). Taking this concept into account, the test methods with the highest applicability should be given greatest weighting values in the evaluation, e.g., when evaluating soil strength, unconfined tests have medium applicability, and when evaluating soil compressibility, consolidated tests have high applicability. The comments about the applicability of certain tests for determining particular ground properties especially apply for ground conditions subjected to sample disturbance, e.g., low plastic soft clay. Furthermore, the weighting given to a specific data set should be linked to the available number of sample derived values per source, the sample quality, the in-situ test quality, the available information from the desk study (experience, neighbouring designs, etc.), the values obtained from back calculation, and engineering judgement.

**Figure 28.** Simplified overview of the applicability of laboratory tests covered by EN 1997-2, Clause 5 (fragment of EN 1997-2/Table B.2).

| | Field investigation tests | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Property | BDP Borehole Dynamic penetration test | BJT Borehole Jack Test | BST Borehole Shear test | CPT/CPTU Cone penetration test | DMT Flat Marchetti dilatometer test | DPT Dynamic Penetration test | Electrical density method | FDP Full displacement poressiometer | FDT Flexible Dilatometer test | FVT Field Vane Test | ISRM- Flat Jack | ISRM - Geophysical Methods | ISRM - Hydraulic Fracturing | ISRM - Overcoring in Borehole | ISRM - Total pressure Cells |
| 7.1.2 Bulk mass denstity | | | | | | C2 | FCR 2-3 | | | | | | | | |
| 7.1.3 Water content | | | | | | | | | | | | | | | |
| 7.1.6 Density Index | C2 | | | C2 | | C1 | | | | | | | | | |
| 7.1.7 Horizontal stress, | | | | | FC3 | | | FC 1-2 | | | R3 | | | | |
| 7.1.7 Hor stress state / orientation | | | | | | | | | | | | | R2-3 | | |
| 7.1.7 insitu stress stae (stress tensor) | | | | | | | | | | | | | | R3 | |
| 8.2 (Undrained) strength | | | CR2 | FC3 | FC3 | C2 | | FC3 | | F3 | | | | | |
| 9.1 Oedometer modulus | | | | FC2 | | | | | FC2 | | | | | | |
| 9.1 E-Modulus | | R2 | | FC1 | FC3 | FC1 | | FC3 | FC3 | | | | | | |
| 9.1. Shear Modulus | | | | FC2 | | | | FC3 | | | | R3 | | | |
| 9.2 Horizontal consoldidation Ch | | | | F2-3 | F2-3 | | | | F2-3 | | | | | | |
| 10.4 Shear wave velocity | | | | FC1 | FC1 | | | FC1 | | | | | | | |

— F = Fine Soils, C = Coarse Soils, R = Rock, 1 = Low Confidence/Applicability, 2 = Medium Confidence/Applicability, 3 = High Confidence/Appli

*Source: EN 1997-2/Table B.2*

**Figure 29.** Simplified overview of the applicability of laboratory tests covered by EN 1997-2/Clauses 7 to 10 (fragment of EN 1997-2/Table B.4).

| | Laboratory Tests | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Property | Atterberg - Casagrande | Atterberg - Fall cone | Atterberg- Tread Method | BE - Bender Element test | CDSS - Cyclic Direct Simple Shear | CTS - Cyclic Torsional Shear | CTX - Cyclic Triaxial Test | Direct shear test | DSS - Direct Simple shear | ISRM - creep characteristics of Rock Methods | ISRM - Huder Amberg method | ISRM - TX - Consildated triaxial compression test | ISRM - UCT - Unconfined Compression test | IST - Interface Shear test | OED CRS Oedometer - CRS | OED - IL Oedometer - incremental | Point Load test |
| 7.1.7 At rest coefficient K0 | | | | | | | | | | | | | | | F3 | F3 | |
| 7.1.7 Pre-consolidation, OCR | | | | | | | | | | | | | | | F3 | F3 | |
| 8.2 Soil Strength | F1 | F1 | F1 | | | | | FC2 | FC3 | | | | | | | | |
| 8.3 Rock Strength | | | | | | | | | | | | R3 | R3 | | | | R2 |
| 9.1 Oedometer Modulus | | | | | | | | | | | | | | | F3 | F3 | |
| 9.1 E-modulus | | | | | | | FC3 | | | | | R3 | R3 | R2 | | | |
| 9.1 Shear modulus | | | | FC2-3 | | | FC3 | | FC3 | | | | | R3 | | | |
| 9.2 Compression, Consolidation and Creep Properties | F1 | F1 | | | | | | | | R3 | | | | | F3 | F3 | |
| 9.2.4 Swelling properties | | | | | | | | | | | R2-3 | | | | F3 | F3 | |
| 10.3 Secant shear modulus and damping ratio curves | | | | | FC3 | R3 | FC1 | | | | | | | | | | |
| 10.4 Very small strain shear modulus | | | | FC2 | FC1 | R2 | FC1 | | | | | | | | | | |
| 10.5 Excess pore pressure | | | | | FC3 | R2 | FC3 | | | | | | | | | | |
| 10.6 Cyclic shear strength | | | | | FC3 | R2 | FC3 | | | | | | | | | | |

F = Fine Soils, C = Coarse Soils, R = Rock, 1 = Low Applicability, 2 = Medium Applicability, 3 = High Applicability

*Source: EN 1997-2/Table B.4*

If weighting is to be applied, this may either be achieved by means of engineering judgement or by structured weighting methods, such as ordinary ranking or pairwise comparison. Structured weighting methods are common procedures adopted for weighting individual factors, for instance in a multi-criteria decision analysis (Köksalan, Wallenius & Zionts, 2011). Structured weighting methods require engineering judgement which is utilized in a formalized framework. Within this document, a weighting method described by Saaty & Vargas (1980) is illustrated in the example developed in Section 5.4.8 of this document.

Note that there are options regarding how to place weightings on the data before analysing a complete data set. However, these structural weighting methods require a rather advanced statistical knowledge whose explanation is beyond the scope of this document. In the context of a Bayesian property estimation as shown in the JRC Report 'Reliability based verification of limit states for geotechnical structures' (European Commission: Joint Research Centre, Schweckendiek, Van den Eijnden et al., 2024), the assignment of weightings is replaced by the use different measurement errors per test type.

### 5.4.4 Combination of continuously measured data and data measured at certain depths

Continuously measured data such CPTs and similar methods are convenient for defining the thickness of a geotechnical unit as they provide a continuous profile. However, continuously measured data should be combined carefully with field and laboratory tests at certain depths. The evaluation of geotechnical properties from continuously measured data often relies on correlations with laboratory tests, back-calculations and/or (local) site experience. Despite numerous available (site-specific) correlations, the ground properties thereby derived exhibit larger uncertainty than more direct field and laboratory tests (see also Figure 28 and Figure 29). A low influence weighting on the representative should be assigned to these more uncertain data sets when using either engineering judgement or a formal weighting method. If field or local laboratory data are of good quality, these results may be used to make a local correlation which is characterized by less uncertainty.

When combining sample derived values from continuously measured data with field or laboratory data at certain depths, the question arises regarding how many data points the continuously measured data represent. The number of sample derived values $n$ in most formulae refers to independent measurements. Considering an average vertical spatial variability of the ground of about 1.0 – 1.35 m (see Section 4.3.3.3.3 of this document), continuously measured data points within the range 1.0 – 1.35 m are dependent on each other. Hence, within this range, continuously measured data cannot be treated similarly as data measured at a certain depth. It is thus recommended to evaluate continuously measured data as a separate data set with Approach 2 (see Section 5.4.6 of this document).

### 5.4.5 Approach 1

Approach 1 simply combines the data from different sources (and/or test methods) into one data set and then use statistics. This approach should comprise the following steps:

— To combine directly the sample derived values from different sources to form one data set, without any previous weighting; and

— to determine the characteristic value of the total data set, as outlined in Chapter 4 and Section 5.1 to 5.3 of this document.

In this approach, it is crucial to be sure that all the data from different sources relate to the same property, before combining them to form one data set. Correlations used to convert in situ and laboratory tests must be consistent, and the respective correlation and correction factors, e.g., fall cone and field vane correction factor, must be relevant. Due to the above limitations, it is not possible to include nominal values from a desk study or back calculations when using Approach 1. Furthermore, this approach does not allow for any weighting of the sample derived values.

Approach 1 is only applicable in few cases. It should be carefully evaluated if the data under consideration meet the required conditions. In the majority of projects, Approaches 2 and 3 should be favoured.

### 5.4.6 Approach 2

Approach 2 combines the characteristic values determined from different sources (and/or test methods) by using statistics and engineering judgement. This approach should comprise the following steps:

— to determine the characteristic values separately for each source and/or test method,

— to weight the obtained characteristic values according to the ideas set in 5.4.2, and

— to determine the characteristic value, based on the weighted individual characteristic values (this characteristic value can be considered a nominal value as the weighting values are based on engineering judgement).

Particularly when there is an inadequate number of sample derived values available from certain test methods, each data set should be evaluated separately before combining the data sets. A limited number of available sample derived values results in the need to select a/some nominal value(s) which, in turn, involves assumptions and engineering judgement.

Approach 2 has the disadvantage that the total number of available sample derived values is not directly taken into account resulting in comparatively high $k_n$-values being calculated and, thus, more conservative characteristic values being obtained. In particular, high-quality laboratory tests, commonly available in smaller numbers due to economic considerations, are penalised in this procedure. Consequently, when combining several characteristic values to obtain a single nominal value, the designer should, amongst other things, account for the total number of tests and also for the quality of the tests (see also Section 5.4.2 of this document).

### 5.4.7 Approach 3

Approach 3 combines the nominal values selected from different sources (and/or test methods) using engineering judgement. This approach should comprise the following steps:

— to determine the nominal value, for a single data set or several combined data sets, and

— to combine the individual nominal values to obtain one nominal value for all the data sets using engineering judgement or weighting according to the ideas set in 5.4.2.

This approach has the drawback that the number of available sample derived values is only considered implicitly by means of engineering judgement. Moreover, similar to Approach 1, it is crucial that all the considered sample derived values describe the same property. Approach 3 is mainly applicable when the available data are insufficient to establish a characteristic value.

One specific method of weighting the sample derived values based on a pairwise comparison is illustrated in Section 5.4.8 of this document. A detailed presentation of the example can be found in Annex 2. Another example using data from multiple sources is explained in Annex 4.

### 5.4.8 Example: Undrained slope stability – Property: undrained shear strength $c_u$ (Case 2)

Note: In order to be able to calculate one representative value based on a combination of different sources that provide the same property, some assumptions need to be made. The presented example is intended as an illustration. When adopted in practice it should be modified depending on the particular design situation and the local boundary conditions, as other assumptions may be more appropriate for a specific application.

#### 5.4.8.1 *Provided results from ground investigation*

The provided results from ground investigation are given in Table 25.

**Table 25.** Provided results from ground investigation – Case 2.

| Test | #Data points | Comment |
|---|---|---|
| Fall cone | 19 | Corrected values, unconfined test. Subjected to sample disturbance |
| Field vane | 20 | Corrected values |
| Direct simple shear tests (DSS) | 4 | Should have better applicability than fall cone test. |
| CPTU | 609 | OCR is included in the correlation used but is not provided in the data set. Applicability depends on pc' and p0' actually being measured or assumed. |

*Source: Developed by the authors.*

#### 5.4.8.2 *Evaluation: Nominal values with weightings (pairwise comparison)*

It should be noted that this example is only presented as an illustration of the relative weightings of the different test methods to determine the nominal values using the weighting approach provided Saaty & Vargas (1980). The different test methods may be given different relative weightings than those presented herein, e.g., in today's practice the data obtained from CPTs, with local or regional/national correlations, will normally be weighted significantly higher than in this example. Furthermore, one may also need to differentiate between different DSS tests due to differences in sample quality (see e.g., Lunne et al., 1997).

The weighting system used for this example is the degree of importance, ranging from 1 to 9, presented in Table 26. This is a common weighting system. However, other systems are also conceivable. The application of this weighting system to compare the different test methods against each other is shown in Table 27.

**Table 26.** Explanation of weighting system.

| Degree of importance | Definition |
|---|---|
| 1 | Equal importance |
| 2 | Weak |
| 3 | Moderate importance |
| 4 | Moderate plus |
| 5 | Strong importance |
| 6 | Strong plus |
| 7 | Very strong |
| 8 | Very strong |
| 9 | Extreme importance |

*Source: Developed by the authors.*

**Table 27.** Pairwise comparison of different test methods for weighting.

| | **Fall cone** | **Field vane** | **DSS** | **CPTU** |
|---|---|---|---|---|
| **Fall cone** | 1 | 1/3 | 1/7 | 5 |
| **Field vane** | 3 | 1 | 1/7 | 5 |
| **DSS** | 7 | 7 | 1 | 9 |
| **CPTU** | 1/5 | 1/5 | 1/9 | 1 |

*Source: Developed by the authors.*

Since the ranking is conducted using just expert judgement it may include partially inconsistent assumptions. The consistency of the ranking in Table 26 can be evaluated using the consistency ratio (CR). As shown in Table 27, the CR is determined as the quotient of the consistency index (CI) and the Random Index (RI) with *n* corresponding to the number of items compared against each other and the maximum eigenvalue $\lambda_{max}$ of the matrix in Table 27. In the herein presented example, *n* = 4 represents the available test methods. The RI depends on the number of sample derived values and is given by Saaty & Vargas (1980). The CR should be less than 10% (up to 20% is tolerable), greater than 38% is too inconsistent. In the case of an inconsistent matrix, the weightings must be revised. Table 28 shows the results of this procedure.

**Table 28.** Results of consistency evaluation.

| | | |
|---|---|---|
| $\lambda_{max}$ | 4.380 | (max eigenvalue) |
| **CI** | 0.127 | $(\lambda_{max} - n) / (n - 1)$ |
| **RI** | 0.900 | (Saaty, 1980) |
| **CR** | 14.1% | CI/RI |

Source: Developed by the authors. The individual weightings for each test are then determined by dividing the weighting for one method in Table 28 by the total of the weightings in the column (see Table 29, columns 2 to 5). Secondly, the average weighting per row is calculated (see Table 29, column 6).

**Table 29.** Weighting results.

| | **Fall cone** | **Field vane** | **DSS** | **CPTU** | **Average weighting** |
|---|---|---|---|---|---|
| **Fall cone** | 0.089 | 0.039 | 0.102 | 0.250 | 0.120 |
| **Field vane** | 0.268 | 0.117 | 0.102 | 0.250 | 0.184 |
| **DSS** | 0.625 | 0.820 | 0.716 | 0.450 | 0.653 |
| **CPTU** | 0.018 | 0.023 | 0.080 | 0.050 | 0.043 |

*Source: Developed by the authors.*

Using the method for depth- or stress-dependent ground properties outlined in Section 5.2 of this document, characteristic values are obtained for the individual test methods. Annex 2 shows the derivation of these characteristic values in detail.

For now, the characteristic values shown in Table 30 are assumed as given. These results are multiplied by the weightings in Table 29 to obtain the nominal undrained shear strength $c_{u,nom,mean}$ (see Table 30). It can be observed that, especially at shallow depths, the large weighting on DSS tests leads to a presumably very conservative nominal value. Compared to the approach presented in Annex 2 of this document that combines the different test methods from two data sets to determine a characteristic value, the nominal value obtained in this example at a depth of 3 m is only about half of the characteristic value. However, as discussed earlier, e.g., a stronger weighting of the CPTUs due to local experience would lead to comparable values.

**Table 30.** Characteristic mean values from different tests and resulting nominal values based on weighting.

| | $c_{u,k,mean}$ | | | | $c_{u,nom,mean}$ |
|---|---|---|---|---|---|
| ***z*** | **Fall cone** | **Field vane** | **DSS** | **CPTU** | |
| m | kPa | kPa | kPa | kPa | kPa |
| 3 | 26.1 | 22.7 | 5.7 | 21.0 | 11.7 |
| 30 | 56.3 | 81.4 | 81.1 | 67.6 | 76.8 |

*Source: Developed by the authors.*

# 6 Cases analysed

Five different cases were developed according to the guidelines given in the previous chapters to show and check their applicability.

In those five cases, the following issues were tackled:

— Cases relative to different geotechnical structures and limit states;

— Cases with different number of derived values: from 8 to 80;

— Cases in which different distribution function should be used: normal and log-normal;

— Cases in which ground properties are dependent and independent of depth;

— Cases in which ground properties are independent (as undrained shear resistance) and dependent (cohesion and friction angle).

The cases are collected in the following annexes and their general features are shown in Table 31:

- Annex I => Case 1: Stability of a slope (Coca, Spain)
- Annex II => Case 2: Undrained slope stability (Göta älv river valley, Sweden)
- Annex III => Case 3: Embankment on soft sensitive clay (Perniö, Finland)
- Annex IV => Case 4: Settlements of an embankment on clay peat (Puurs, Belgium)
- Annex V => Case 5: Stability of a retaining wall (Bucharest, Romania)

**Table 31.** General features of the Cases.

| Case | Geotechnical Structure | Limit State | Ground property | Number of data | Distribution function | Type of estimate |
|---|---|---|---|---|---|---|
| 1 | Slope | Overall stability | • Cohesion<br>• Friction angle | 6 | Normal | Type A ([1]) |
| 2 | Slope | Overall stability | Undrained shear strength | 43 | Normal | Type A ([1]) |
| 3 | Embankment | Overall stability | Undrained shear strength | 32+6 +6 | (Depth-dependent) | Type A ([1]) |
| 4 | Embankment | SLS: duration of settlement | Hydraulic conductivity | (various sources) | Normal | Type A ([1]) |
| 5 | Self-supported (cantilever) pile retaining wall | • SLS movement<br>• Stability<br>• Wall resistance | • Unit weight,<br>• deformation modulus total strength (un-drained cohe-sion and friction angle) | 56 (various sources) | (Depth-dependent) | Type A ([1]) |

[1] Type A: estimate of the mean value

*Source: Developed by the authors.*

# 7 Conclusions

This document:

- makes a summary of the global process of the design of geotechnical structures, as given in EN 1997, to frame the step in which the designer must determine the representative and design values to be used in the limit state verifications, based on the values collected in the ground investigation;
- highlights the fact that the values involved in the limit state verifications are not only values related to ground properties, but also to geometrical properties, which include, among others, all the ground water related levels and the geometrical description of rock discontinuities;
- explains the different concepts related to the ground property values that are named as: measured, derived, representative, nominal, indicative, characteristic, design and best estimate values;
- describes the two possible ways to determine the representative value of a ground property, as established in EN 1997-1: selecting the value from a limited number of test results, based on engineering judgment and comparable experience in similar cases, being in this case termed a 'nominal value'; or evaluating the value by statistical methods, being in this case termed a 'characteristic value;
- introduces a new type of estimate in the determination of the representative vale, not included in EN 1997-1, for those cases in which the limit state is sensitive to the variability of the ground but not so sensitive that the representative value is an inferior or superior value. In the case of the characteristic values this new estimate is based on the sensitivity index;
- describes procedures to determine the representative values in special cases, although very common in usual practice. These include situations, such as when there are few samples, when the properties are depth-dependent, and when there are data from different sources; and
- presents five examples that cover different aspects: different geotechnical structures and limit states, different number of derived values, different distribution function, independent and depth-dependent ground properties.

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## List of abbreviations and definitions

| Abbreviations | Definitions |
|---|---|
| CC | consequence class |
| CEN | European Committee for Standardization (CEN, French: Comité Européen de Normalisation) |
| CI | confidence intervals |
| CI | consistency index |
| CPT | cone penetration test |
| CPTU | piezocone penetration tests |
| CR | consistency ratio |
| DSS | direct simple shear tests |
| FC | fall cone test |
| FV | field vane test |
| GC | geotechnical category |
| GCC | geotechnical complexity class, |
| GDM | geotechnical design model |
| GDR | geotechnical design report |
| GIR | ground investigation report |
| GLM | generalised linear model |
| GM | ground model |
| MFA | material factor approach |
| MLM | maximum likelihood method |
| OCR | over consolidation ratio |

| Abbreviations | Definitions |
|---|---|
| OLS | ordinary least square |
| RFA | resistance factor approach |
| RI | random index |
| SLS | serviceability limit state |
| TX | triaxial test |
| TXC | compression triaxial test |
| TXE | extension triaxial test |
| ULS | ultimate limit state |
| WLS | weighted least squares |

## List of symbols

The symbols used in this document correspond to those used in the Eurocodes EN 1990 and EN 1997, except the following ones:

| Symbols | Definitions |
|---|---|
| $L_v$ | extent of the volume of ground involved in the limit state, in the vertical direction |
| $\Gamma^2$ | sensitivity index |
| $\mu$ | factor, function of the liquid limit, to correct field vane strength values |
| $\Theta$ | scale of fluctuation (in EN 1997-1, the symbol used is $\delta_x$) |

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## Annexes

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Annex II. Case 2: Undrained slope stability (Göta älv river valley, Sweden)

Annex III. Case 3: Embankment on sensitivity clay (Perniö, Finland)

Annex IV. Case 4: Settlements of an embankment on clay peat (Puurs, Belgium)

Annex V. Case 5: Stability of a retaining wall (Bucharest, Romania)

# Annex I: Case 1 – Stability of a slope (Coca, Spain)

## I.1. Description of the case

### I.1.1. General features

Table 32 collects the main general features of Case 1 relevant for the evaluation of representative values.

**Table 32.** General features of Case 1.

| | |
|---|---|
| **Geotechnical structure involved** | Slope with a historical building on its top |
| **Design situation** | Persistent |
| **Limit state under verification** | Overall stability |
| **Calculation model** | Stability analysis by limit equilibrium method |
| **Ground property to be determined** | Strength<br>In this case: effective cohesion and friction angle |
| **Type of representative value estimate** | Type A: estimate of the mean value (see 4.3.3.3.3 ) |

*Source: Developed by the authors*

In recent years, a series of landslides have occurred at the about 40 m high slope, with an inclination between 37° and 44°, carved by the Eresma river, tributary of Duero river, on its left bank as it passed through the historical town of Coca, in Central Spain (Figure 30 and Figure 31). The most important instability is located in the vicinity of the medieval tower of San Nicolás, of Mudejar-style, declared of National Cultural Interest. This required an in-depth study to know the safety level of the slope, to analyse the causes of the landslides and to design possible solutions. (Pardo et al, 2021).

**Figure 30.** Aerial photograph of the natural slope.

*Source: Pardo de Santayana et al. (2021)*

**Figure 31.** Frontal view of the natural slope

*Source: Pardo de Santayana et al. (2021).*

### I.1.2. The Ground Model

<u>Ground profile</u>

Figure 32 shows the ground profile of the slope in which signs of instability can be shown. The red line represents the ground surface prior to the last slide whose origin is the scour in its toe due to river water.

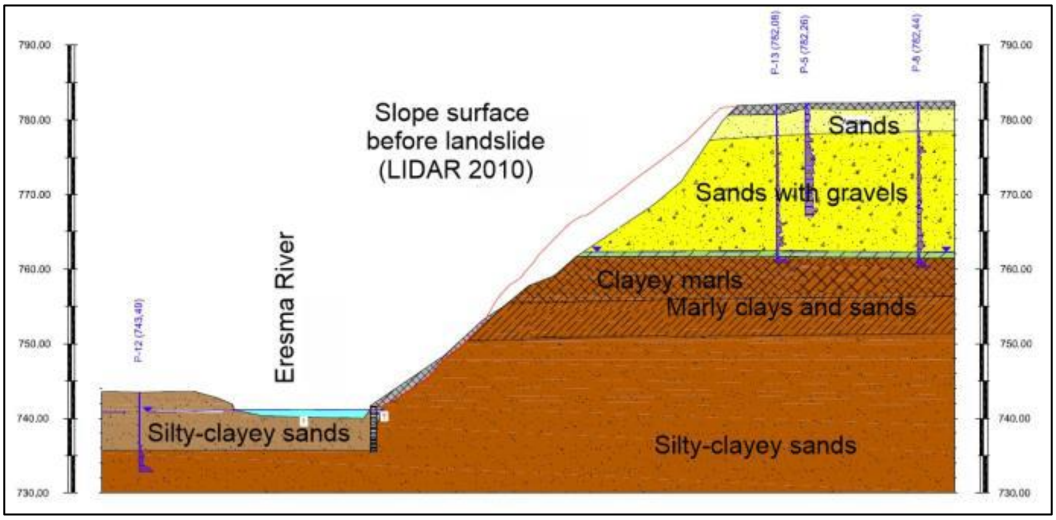
From top to bottom, three geotechnical units can be identified:

— Unit 1- sands with gravels (in yellow);

— Unit 2- clayey marls (in dashed brown) and

— Unit 3- silty-clayey sands (in brown).

**Figure 32.** Ground Model of the natural slope.



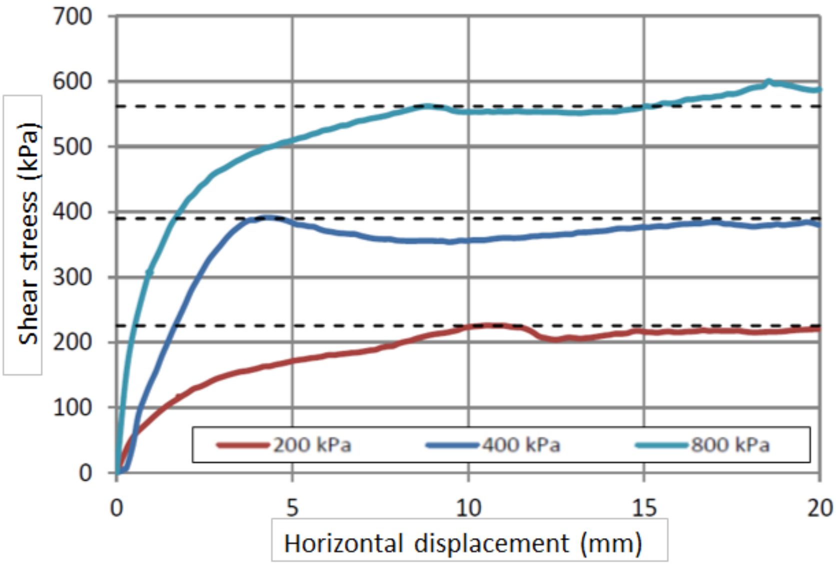

*Source: Pardo de Santayana et al. (2021)*

<u>Derived values</u>

For this document, the characteristic and representative value determination will be done only on Unit 3.

To determine the strength characteristic of the silty-clayey sand materials (Unit 3), six drained direct shear tests were performed, according to EN-ISO 17892-10, whose horizontal displacement v shear stress curves can be considered as the 'measured values' (see Figure 33). The normal stresses used in the tests were in the range between 120 and 960 kPa and the failure shear stresses obtained were between 100 and 550 kPa.

**Figure 33.** One of the 'measured values': the horizontal displacement v shear stress curve of a direct shear test.




*Source: Developed by the authors*

The interpretation of the measured values in the six direct shear tests was done using the Mohr-Coulomb failure criterion which made it possible to obtain the 'derived values' relative to effective cohesion (c) and friction angle (φ). The numerical measured and derived values are collated in Table 33.

**Table 33.** Measured and derived values.

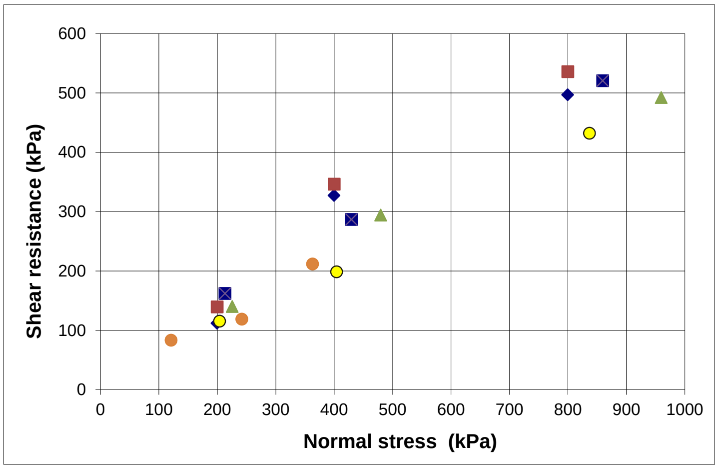
| Test | Measured values | | Derived values | | |
|---|---|---|---|---|---|
| | **Normal stress (kPa)** | **Shear resistance (kPa)** | **c´ (kPa)** | **Tan φ´** | **φ´ (º)** |
| 8707 | 200<br>400<br>800 | 112<br>327<br>496 | 27 | 0.61 | 32.40 |
| 8709 | 200<br>400<br>800 | 139<br>346<br>535 | 45 | 0.63 | 32.36 |
| 8714 | 226<br>480<br>960 | 140<br>294<br>492 | 47 | 0.47 | 25.21 |

| Test | Measured values | | Derived values | | |
|---|---|---|---|---|---|
| | Normal stress (kPa) | Shear resistance (kPa) | c´ (kPa) | Tan φ´ | φ´ (º) |
| 8715 | 214<br>430<br>860 | 162<br>287<br>520 | 46 | 0.55 | 28.95 |
| 8717 | 204<br>404<br>837 | 115<br>199<br>432 | 4.5 | 0.51 | 26.86 |
| 28892 | 121<br>242<br>363 | 83<br>119<br>212 | 9.5 | 0.53 | 27.98 |

*Source: Developed by the authors*

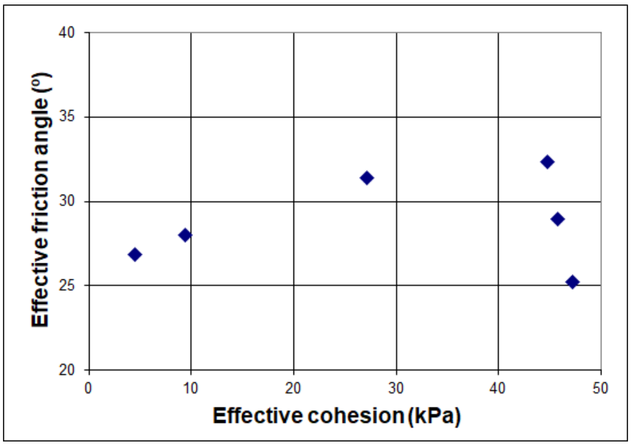
The values of normal stress and shear resistance from the direct shear tests are plotted in Figure 34, while the derived values (pairs of effective cohesion and friction angle) are shown in Figure 35

**Figure 34.** Normal stress and shear resistance from the direct shear tests.




*Source: Developed by the authors.*

**Figure 35.** Derived values obtained from the direct shear tests.




*Source: Developed by the authors.*

## I.2. General considerations

The determination of the characteristic values of cohesion and friction angle was done applying two different procedures:

- Considering them as as independent values from a statistical point of view, so the set of derived values of effective cohesion and friction angle were treated separately, according to the procedure set in Annex A/EN 1997-1.
- Considering them as dependent values, correlated through the Mohr-Coulomb failure criterion. In this case, the procedure used is the one set in Chapter 5.3 of this document.

According to the ULS under verification, the representative values must correspond to the estimate of the average value of the property in the volume affected by the limit state (Type A estimate, see 4.3.3.3.3).

In a parallel way, some TG-C1 members sent their selected nominal values which allows making a comparison with the characteristic values determined through the procedures.

## I.3 Analysis of the values

### I.3.1 Management of the derived values

As said in EN1997-1, 4.3.2.1 (1), 'Representative values of ground properties to be used in ultimate and serviceability limit state verifications shall be determined from derived values collected in the GIR' and in Note 1 'The representative value refers to a particular ground property of a single geotechnical unit', the derived values to be considered in this document are collected in Table 33. They refer to the shear strength (in this case, effective cohesion and friction angle) of Unit 3 (silty-clayey sand materials).

Although not stated in Parts 1 or 2, an internal quality control of the data can be done by a critical review for consistency. Some data can be disregarded if some problems during their gathering are reported in the GIR.

In this case, all the derived values are going to be considered. No relation of the data with depth is going to be considered.

### I.3.2 Selection of 'Nominal Values'

As said before, some TG-C1 members sent their selected nominal values (as a cautious estimation of the mean value) which allows making a comparison among them and with the characteristic values determined in next section. Table 34 collects the values selected.

**Table 34.** Nominal values selected by different TG-C1 members.

| TG-C1 member | Effective Cohesion (kPa) | Effective Friction angle (°) |
|---|---|---|
| #1 | 20 | 25 |
| #2 | 20 | 28 |
| #3 | 0 | 25-27 |
| #4 | 5 | 30 |
| #5 | 4 | 27.5 |

*Source: Developed by the authors*

It can be seen that the nominal values for effective cohesion are widely spread with values between 0 and 20 kPa, while the nominal values selected for the effective friction angle range between 25° and 30°.

### I.3.3 Evaluation of 'Characteristic Values'

As independent properties

According to the ULS under verification, the representative values must correspond to the average value of the property in the volume affected by the limit state, so calculations must be done using Case A 'Estimate of the mean value' with the two alternatives of Cases 3 '$V_{X,assumed}$' and 2 '$V_{X,unknown}$', as shown in Figure 36, taken from EN 1997-1.

**Figure 36.** Values of kn for different cases and type of estimations.

| $V_X$ CASES | Estimate of the mean value | Estimate of the inferior or superior value (5 or 95 % fractile) |
|---|---|---|
| Case 1: "$V_X$ known" & Case 3 "$V_X$ assumed" | $k_n = N_{95}\sqrt{\frac{1}{n}}$ | $k_n = N_{95}\sqrt{1+\frac{1}{n}}$ |
| Case 2: "$V_X$ unknown" | $k_n = t_{95,n-1}\sqrt{\frac{1}{n}}$ | $k_n = t_{95,n-1}\sqrt{1+\frac{1}{n}}$ |

*Source: EN 1997-1, Annex A.*

Table 35 collects the values of the statistical parameters used in the calculation and the characteristic values obtained, that are represented in Figure 37 together with the derived values.

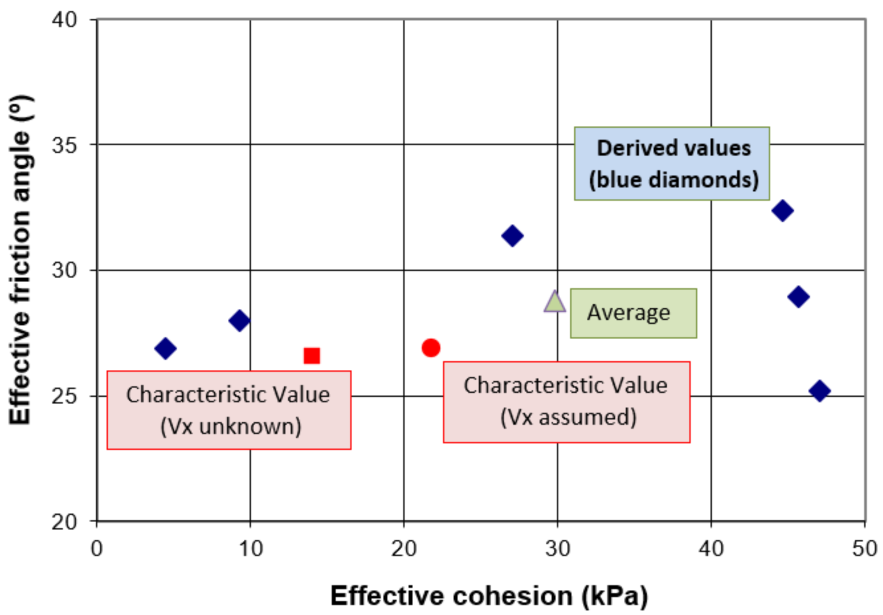
**Table 35.** Statistical values for the determination of characteristic values of c and $\phi$.

| **Parameter** | **Effective cohesion (kPa)** | | **Effective friction angle (º)** ([1]) | |
|---|---|---|---|---|
| Number of data (n) | 6 | | 6 | |
| $X_{mean}$ | 29.81 | | 28.79 | |
| Standard Deviation | 19.20 | | 2.71 | |
| Case<br><br>$N_{95}$ - $t_{95,\,n-1}$ | *Case A2*<br>*Vx,assumed* ([2])<br>1.645 | *Case A3*<br>*Vx,unknown*<br>2.02 | *Case A2*<br>*Vx,assumed* ([2])<br>1.645 | *Case A3*<br>*Vx,unknown*<br>2.02 |
| $Vx$ | 0.40 | 0.64 | 0.10 | 0.09 |
| $k_n$ | 0.67 | 0.82 | 0.67 | 0.82 |
| $X_k$ | 21.82 | 14.01 | 26.86 | 26.56 |

([1]) T Determination done in terms of coefficient of friction angle (tan $\varphi'$)
([2]) Values of *Vx*, assumed taken from Table A1 in EN1997-1, Annex A

*Source: Developed by the authors*

**Figure 37.** Derived and characteristic values.




*Source: Developed by the authors.*

As dependent properties

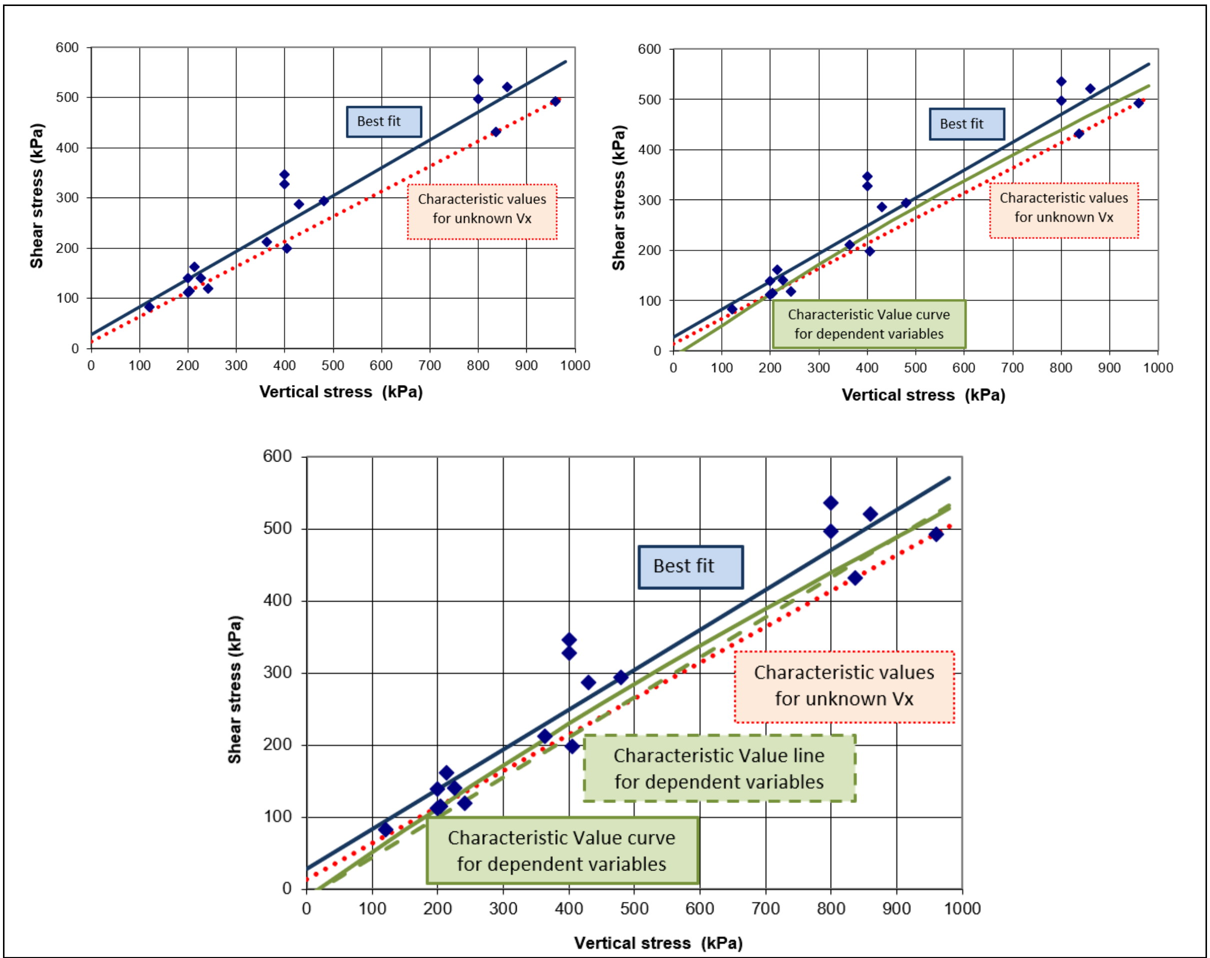
In this case, the Mohr-Coulomb parameters (effective cohesion and friction angle) are considered as dependent properties since they are correlated through the Mohr-Coulomb failure criteria. So, Equation 49 in Section 5.3 are used to determine the characteristic Mohr-Coulomb parameters.

However, a further step is taken since those expressions lead to a curve, not a line, as shown below with the green curve and line. In order to get a line, expressions from Section 2.5.2 of DNV-RP-C207 (2012) were used

Figure 38 shows the failure points in a normal-shear stress plot together with:

- Blue continuous line representing the Mohr-Coulomb failure criteria as the line that best fits the failure points;
- Red dotted line representing the Mohr-Coulomb failure criteria with the characteristic values determined considering independent values and $V_x$ unknown (obtained in previous section);
- Green continuous curve representing the characteristic curve (note that this is not a line, it is a hyperbolic function, so it is difficult to manage and to compare with the other results);
- Green dashed and dotted line representing the characteristic line from which the characteristic values can be deduced.

**Figure 38.** Failure points together with characteristic Mohr –Coulomb failure criteria lines.




*Source: Developed by the authors.*

Table 36 collects the Mohr-Coulomb parameters determined by the different procedures used to facilitate their comparison.

**Table 36.** Characteristic values obtained when considering dependent data.

| Procedure | Effective cohesion (kPa) | Effective friction angle |
|---|---|---|
| Independent properties (Vx known) | 21.82 | 26.86 |
| Independent properties (Vx unknown) | 14.01 | 26.56 |
| Dependent properties (Best fit = 'Mean value') | 27.80 | 29 |
| Dependent properties (Characteristic line) | -11.27 ([1]) | 29 |

([1]) The cohesion value corresponding to the characteristic line for dependent parameters is negative so that would lead to re-evaluate the data considering in this case a null value for cohesion (c=0).

*Source: Developed by the authors.*

### I.3.4 Evaluation of results: determination of representative values

According to EN 1997, 4.3.2.1(6), '*the representative value of a ground property $X_{rep}$ shall be determined from either Formula (4.1) or Formula (4.2)*', shown in Equations 53 and 54:

**Equation 53** Determination of representative values as nominal value [Taken from EN 1997-1/Formula (4.2)].

$$X_{\mathrm{rep}} = X_{\mathrm{nom}}$$

**Equation 54** Determination of representative values as characteristic value [Taken from EN 1997-1/Formula (4.1)].

$$X_{rep} = X_k$$

Where $X_{nom}$ is the nominal value of the ground property and $X_k$ is the characteristic value of the ground property.

So, the last step is to choose the representative value from either the nominal or the characteristic values. It is out of the scope of this case to make a preference for one of those values since its selection is in the hands of the designers, according to their experience, their knowledge of the site and any other constrain from the project.

## I.4 Summary and conclusions

The main features of this case are collected in the following table.

**Table 37.** General features of Case 1

| | |
|---|---|
| **Geotechnical structure involved** | Slope with a historical building on its top |
| **Design situation** | Persistent |
| **Limit state under verification** | Overall stability |
| **Calculation model** | Stability analysis by limit equilibrium method |
| **Ground property to be determined** | Strength<br>In this case: effective cohesion and friction angle |
| **Type of representative value estimate** | Type A: estimate of the mean value (see 4.3.3.3.3) |

*Source: Developed by the authors*

In the slope, three geotechnical units were identified. For this case, only the shear strength of one of those units will be analyzed through the results of six direct shear tests.

The derived values, in terms of effective cohesion and friction angle, were analyzed to determine the nominal, characteristic and representative values of effective cohesion and friction angle as independent properties.

Additionally, the derived values, in terms of pairs of normal stress and shear stress were analyzed to determine the effective cohesion and friction angle as dependent properties.

## References

Pardo de Santayana, F., Díez, J.A., and Perucho, A., 'Stability analysis of a natural slope at left bank of Eresma river in the historical town of Coca, Spain'. *20th International Conference on Soil Mechanics and Geotechnical Engineering*, May 1st to May 5th, Sydney, Australia, 2022, pp. 2541 – 2546, Online library of the International Society for Soil Mechanics and Geotechnical Engineering (ICSMGE), – Rahman and Jaksa (Eds), Australian Geomechanics Society, Sydney, Australia, ISBN 978-0-9946261-4-1, https://www.issmge.org/uploads/publications/1/120/ICSMGE_2022-435.pdf

# Annex II: Case 2 – Undrained slope stability (Göta älv river valley, Sweden)

## II.1. Description of the case

### II.1.1. General features

Table 38 collects the main general features of Case 2 relevant for the evaluation of representative values.

**Table 38.** General features of Case 2.

| | |
|---|---|
| **Geotechnical structure involved** | Natural slope |
| **Design situation** | Persistent |
| **Limit state under verification** | Overall stability |
| **Calculation model** | Stability analysis by limit equilibrium method |
| **Ground property to be determined** | Strength<br>In this case: undrained shear strength |

*Source: Developed by the authors*

The Göta älv river is an area in Sweden that is very prone to landslides and the Swedish Geotechnical Institute has registered over 200 landslides in modern time. The geology in the Göta älv river zone consists of deep layers of clay and some of those areas of clay are classified as highly sensitive 'quick clay'. During past 20–30 years several large government-financed projects have been carried out to develop calculation models to estimate landslide risk, mapping of quick clay and landslide risk, including a vast amount of geotechnical investigations and slope stability analyses. Examples of a landslide that has occurred along the Göta älvriver is presented in Figure 39. Notably, Eurocode 7 is not applicable for natural slope stability, therefore this case should be regarded as an example of stability consideration for a potential construction.

**Figure 39.** Example of landslide at Göta Älv river.

*Source: Picture by Linus Olsson, 2019.*

### II.1.2. The Ground Model

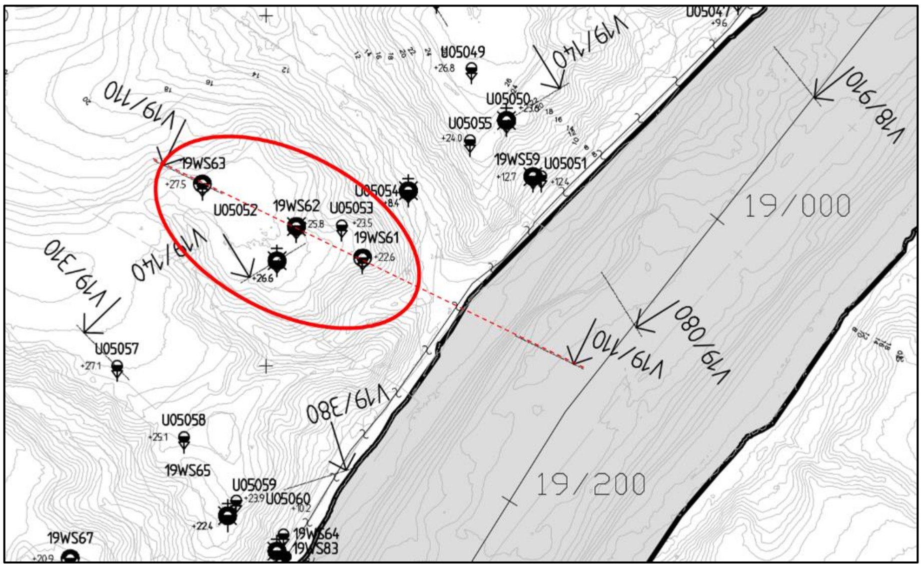
Ground profile

An investigation plan of the ground investigation is presented in Figure 40. A representative Ground Model of the slope is presented in Figure 41, in which signs of erosion on the riverbed and clay slope can be shown.

From top to bottom, two geotechnical units were identified:

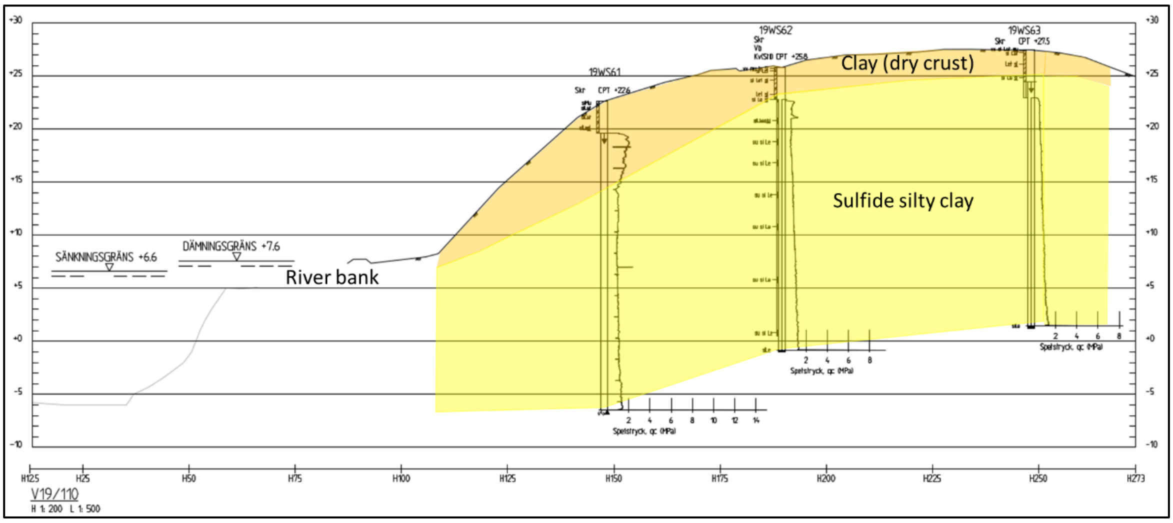
- Unit 1- Clay (dry crust) (in brown),and
- Unit 2- Sulphide silty clay (in yellow).

**Figure 40** Plan of ground investigations (investigations included in the evaluation of representative values are presented within the red line).




*Source: Developed by the authors.*

**Figure 41** Ground Model of the natural slope.




*Source: Developed by the authors.*

Derived values

The ground investigations included in the evaluation of representative values are presented in Table 39 and consists of:

- Index tests, performed in two locations (investigation points), including evaluation of unit weight, water content, liquid limit and sensitivity. Index properties are presented in Figure 42 and Figure 43.
- Measured and derived values of undrained shear strength presented in Figure 44.

In this example, derived values of undrained shear strength were provided directly by the Swedish Geotechnical Institute. Values of undrained shear strength are commonly determined from measurements of Cone Penetration Tests (CPT) and Field vane (FV)/Fall cone (FC) according to Swedish standards by Equations 55 and 56, respectively.

**Equation 55**. Determination of undrained shear strength from measurements from CPT [Taken from Swedish standards].

$$\mathrm{C}_u = \frac{\mathrm{q}_t - \sigma_{v0}}{13{,}4 + 6{,}65\mathrm{w}_L}\left(\frac{OCR}{1{,}3}\right)^{-0{,}2}$$

**Equation 56**. Values of undrained shear strength from measurements from Field Vane and Fall cone [Taken from Swedish standards].

$$\mathrm{C}_u = \tau_{FV,FC}\left(\frac{0{,}43}{\mathrm{w}_L}\right)^{0{,}45}\left(\frac{OCR}{1{,}3}\right)^{-0{,}15}$$

Where $(\mathrm{q}_t - \sigma_{v0})$ is the net cone-resistance, $\mathrm{w}_L$ is the liquid limit, OCR is the over consolidation ratio and $\tau_{FV,FC}$ is the shear strength evaluated from the Field vane or Fall cone test procedure.

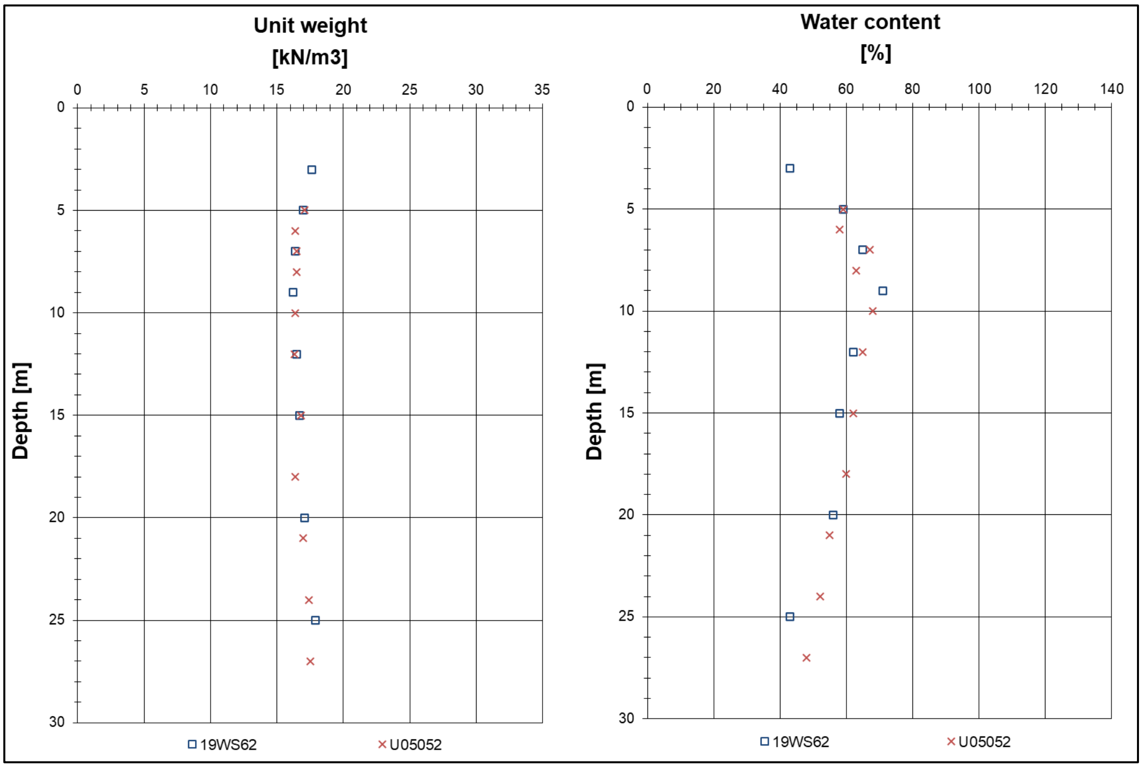
**Table 39.** Measured values obtained in the ground investigation.

| **ID** | **Number of measured values** | | | | |
|---|---|---|---|---|---|
| | **CPT** (1) | **Field vane (FV)** | **Fall cone (FC)** | **Direct shear (DS)** | **Index** |
| 19WS62 | 118 | 8 | 8 | 4 | 8 |
| U05052 | 120 | 12 | 11 | - | 11 |
| 19WS61 | 130 | - | - | - | |
| 19WS63 | 107 | - | - | - | |
| U05053 | 102 | - | - | - | |

(1) Value registered every 0.2 m.

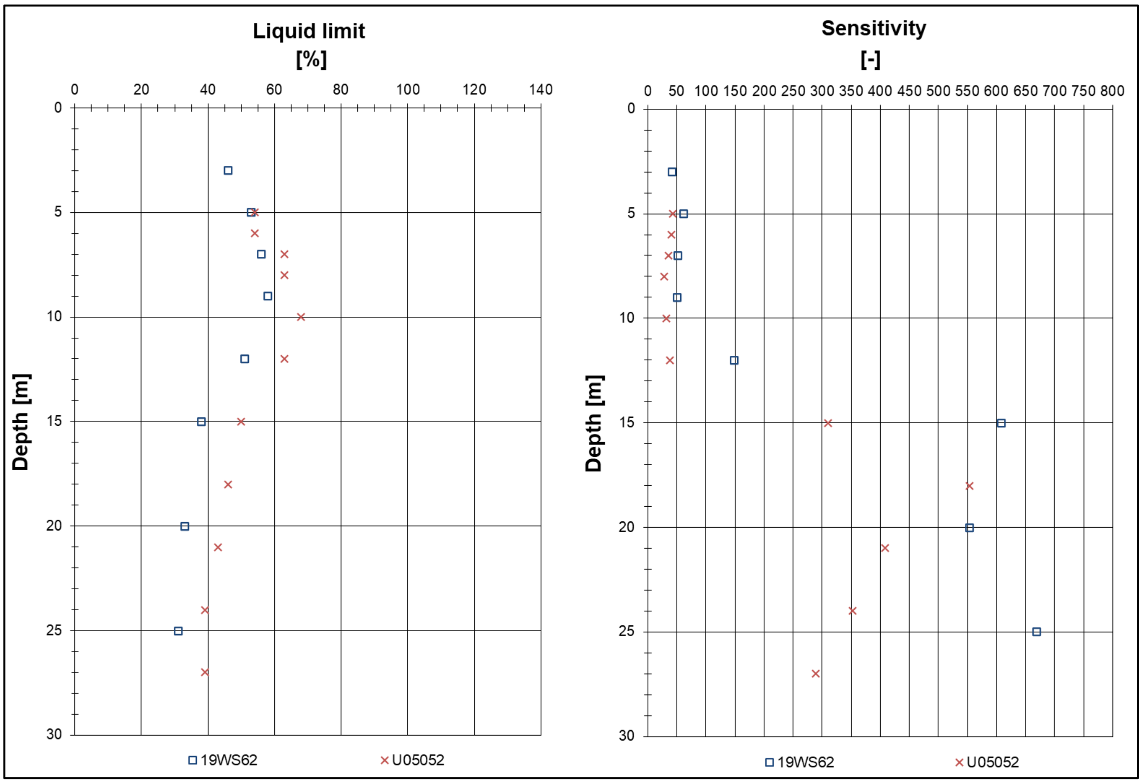
*Source: Developed by the authors.*

**Figure 42** Measured values of unit weight and water content.



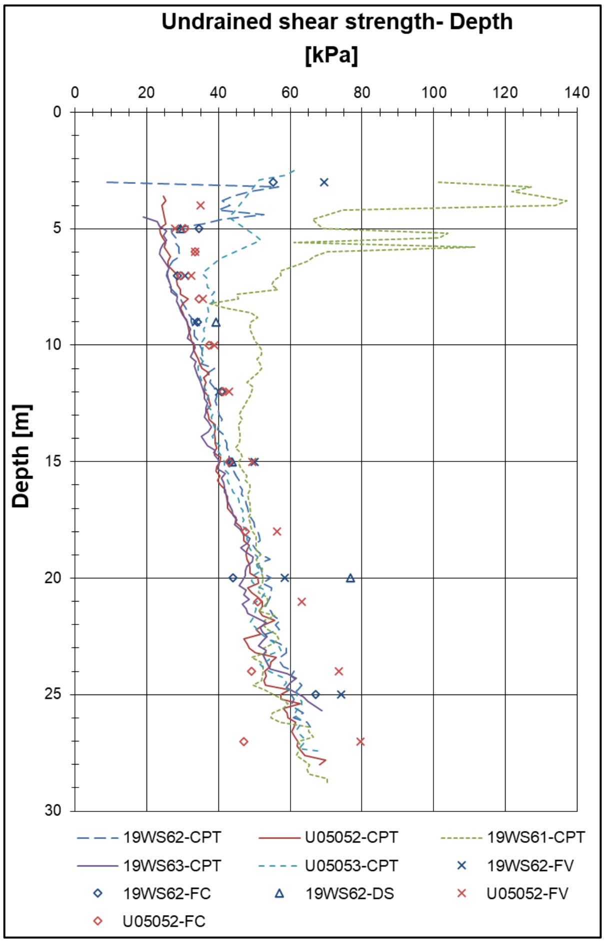

*Source: Developed by the authors.*

**Figure 43** Measured values of liquid limit and sensitivity.




*Source: Developed by the authors.*

**Figure 44** Measured and derived values of undrained shear strength.




*Source: Developed by the authors.*

## II.2. General considerations

Given the data provided in Table 39 and Figure 42 to Figure 44, TG-C1 members were invited to send their representative values, both nominal and characteristic. All members were provided with the data in digital format (by an Excel file). As a benchmark for evaluation of this case example, the representative value of the undrained shear strength of the clay was also calculated statistically (provided by the main drafter) in Section 3.3, i.e. calculating the characteristic value.

The aim of this exercise is to show the readers of this guideline document that the determination of representative values is always influenced by the designers´ engineering judgement, mainly based, in this context, on their personal experience and their potential knowledge of the site and the geotechnical units involved.

## II.3 Analysis of the values

### II.3.1 Management of the derived values

According to EN1997-1, 4.3.2.1 (1), '*Representative values of ground properties to be used in ultimate and serviceability limit state verifications shall be determined from derived values presented in the GIR*' and in Note 1 '*The representative value refers to a particular ground property* *of a single geotechnical unit'*. An interpretation of the different geotechnical units is given in Figure 3 and the derived values to be considered in this document are presented in Figure 42 to Figure 44.

As recommended in EN1997-1, 4.2.4 (3) and Table 4.7, an internal quality control of the data can be done by a critical review '*to identify inconsistencies and anomalies*.' Some data can be disregarded if some problems during their gathering are reported in the GIR. Furthermore, the representative values should consider data that are representative for the geotechnical unit.

In this case, few (single) values evaluated from the CPT were excluded from the provided values.

### II.3.2 Selection of 'Nominal Values'

The provided values from three TG-C1 members (#1–3) are presented in Figure 45 and Figure 46.

Member #1 and 2 presented their evaluated values based on a plot of undrained shear strength vs level (Figure 45) and member #3 presented the evaluated values based on a plot of undrained shear strength vs depth (Figure 46). Notably, member #1 combined the selection of a nominal value with the calculation of a characteristic value. The evaluated value given by member #1 is presented in the section of nominal values although it should not be regarded as a 'pure' nominal value.

Each member provided comments on their evaluated values which are given in the following bullet list.

<u>#1 (combination of nominal and characteristic procedure):</u>

Member #1 calculated the characteristic value according to EN 1997-1, Annex A, Table A.3 (Type A and Case 2 'Vx assumed') each 0.2 m of depth.

- Only some of the data has been disregarded from the analysis, from the first 5 meters of 19WS61-CPT, where the obtained values were very large compared to the other values. The rest of the data has been processed based on depth and a statistical analysis using Table A.3 in EN1997-1, Annex A.
- It was observed that there is some dependence with depth between the interval of elevations of approximately 0.0 m and 15.0 m.
- It was noticed that the characteristic value obtained mathematically is unreasonably small due to the high variance of the values. Therefore, manual processing was conducted, where the values in the chart below are recommended, roughly in the following way:
  - 35 kPa down to approximately 15.4 m;
  - Linear variation from the previous point down to 60 kPa at approximately 0.0 m;
  - Constant value of 60kPa below 0.0 m.

<u>#2 (nominal procedure):</u>

- In Denmark we would correlate $q_{net}$ from the CPT ($q_{net} = q_T - \sigma_v$) with the vane shear test to establish a $N_{kt}$ factor and derive $c_u$ from this as $c_u = q_T / N_{kt}$. If TX or DSS tests are available, we would most likely try to use these – but only if sufficient tests are available (which often is not the case as many tests are often needed to establish a reliable cu profile).
- In Denmark we would most likely not even consider treating these data in a statistical way.

— Assuming that $c_u$ derived from CPT on the $c_u$ vs. level plot is derived as described above I would establish a representative profile from the nominal profile $X_{rep} = X_{nom}$ (4.1.a) as a cautious estimate, properly using a cautious estimate of the average value of the undrained shear strength in the volume involved in the limit state.

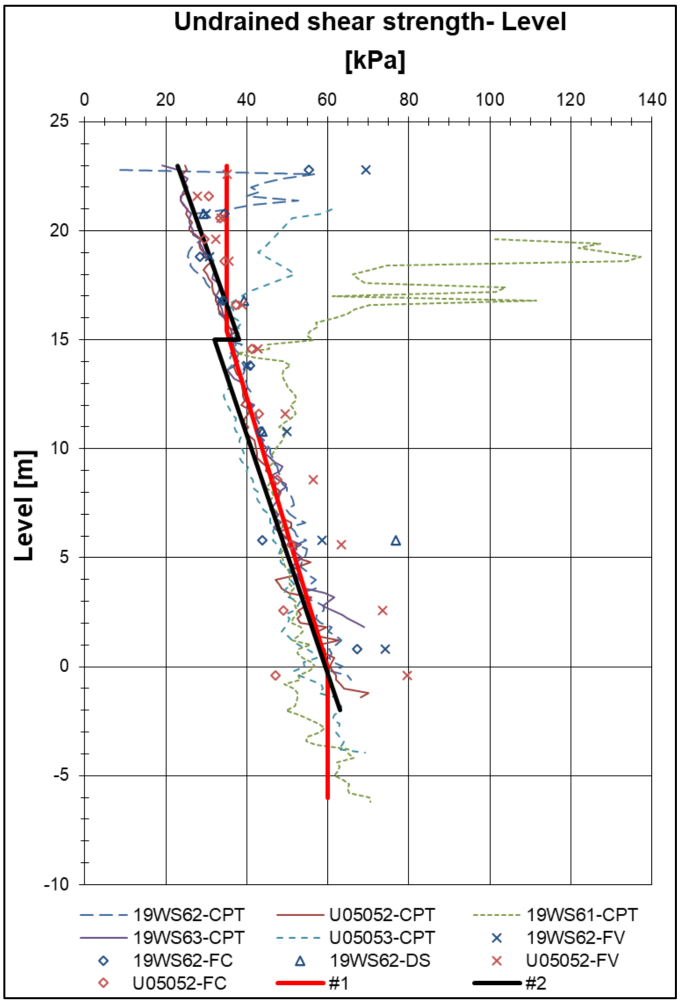
#3 (nominal procedure):

— Suggestion for nominal value should be compared with normal SHANSEP-values. It depends on sample quality of lab specimens. Deep samples in quick clay (seems to be quick deeper than 12 meters) are often subjected to sample disturbance. Therefore, low FC-values below 20 meters can be unreliable. DS on 20 meters seems to correlate badly with CPTU's.

— Uncertainty due to quality should be taken into account when estimating nominal values

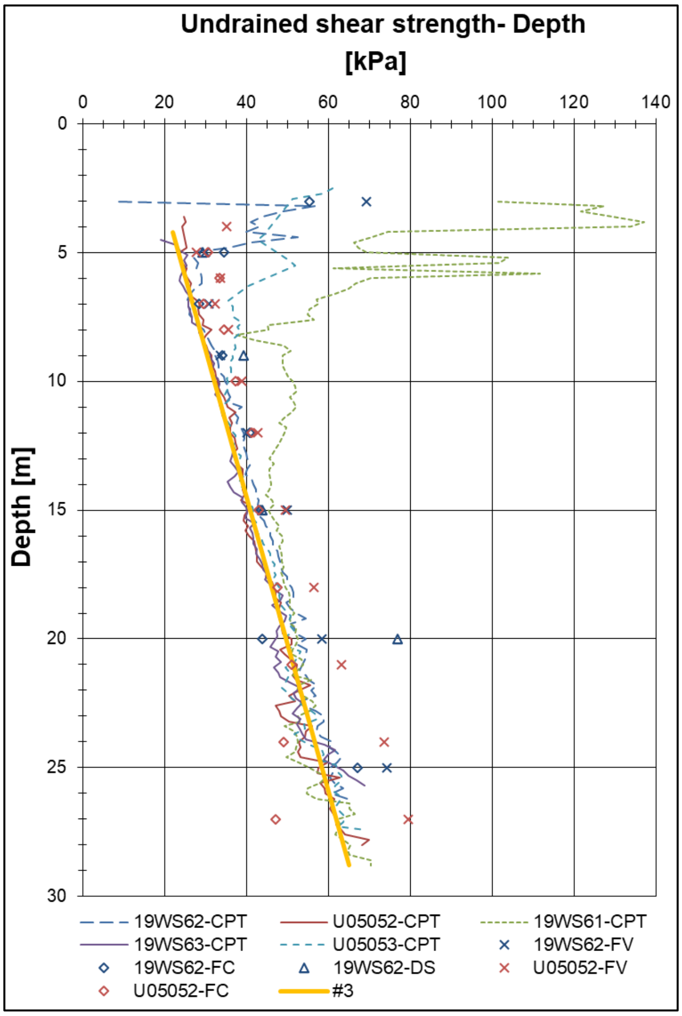
Regarding zone of influence. What to choose in the bottom of slope, where we don't have any investigations might be very important in this case.

**Figure 45** Evaluated characteristic/nominal values by members #1 and 2.




*Source: Developed by the authors.*

**Figure 46** Evaluated nominal value by member #3.




*Source: Developed by the authors.*

### II.3.3 Evaluation of 'Characteristic Values'

Notably, TG-C1 members were invited to send their evaluated representative values (nominal or characteristic). One member (#1) provided an answer based on a combination of a nominal and characteristic procedure which is presented in the section above. As a benchmark for evaluation of this case example the main drafter (denoted as member #4) of this case report provided a calculation of the characteristic value which is based on the Designers' Guide to Eurocode 7: Geotechnical design (Frank et al, 2004).

Notably, the procedure described by Frank et al (2004) is not provided in the draft of EN 1997-1:2022. The procedure given in Annex A provides guidance on how to evaluate the characteristic value on stationary data. However, in Annex A clause (11) and Note 3, the following is stated: '*There are determination procedures, different from the one described in this Annex, that can be used to determine the characteristic relation line (for example least squares using regression analysis) of the values of a ground property that varies with depth (z) or the characteristic values of dependent properties (e.g. cohesion and friction angle).*'

According to the ULS under verification, the representative values must correspond to the estimate of the average value of the property in the volume affected by the limit state. In this case, calculation of the characteristic value was done by estimation of the 95% confident mean value (denoted as Type A in EN 1997-1). The characteristic value was calculated given a homogeneous soil (Unit 2), with local sampling and a linear trend towards the depth (non-stationary) according to Equations 57 to 59. Hence, values that were considered to represent Unit 1 (dry crust) were excluded from the analysis:

- 19WS62 – values between 0 and 5 m;
- 19WS61 – values between 0 and 13 m;
- U05053 – values between 0 and 7 m.

**Equation 57** Determination of characteristic mean trend.

$$x_{\mathrm{k}}^{\{non-stationary\}}(\bar{x}|z) = [\bar{x} + b(z - \bar{z})] - t_{n-2}^{0.95} s_{\bar{x}|z}$$

**Equation 58**. Determination of the variable $s_{\bar{x}|z}$.

$$s_{\bar{x}|z} = \sqrt{\left(\frac{1}{n} + \frac{(z - \bar{z})^2}{\sum_{i=1}^{n}(z_i - \bar{z})^2}\right) s_X^2}$$

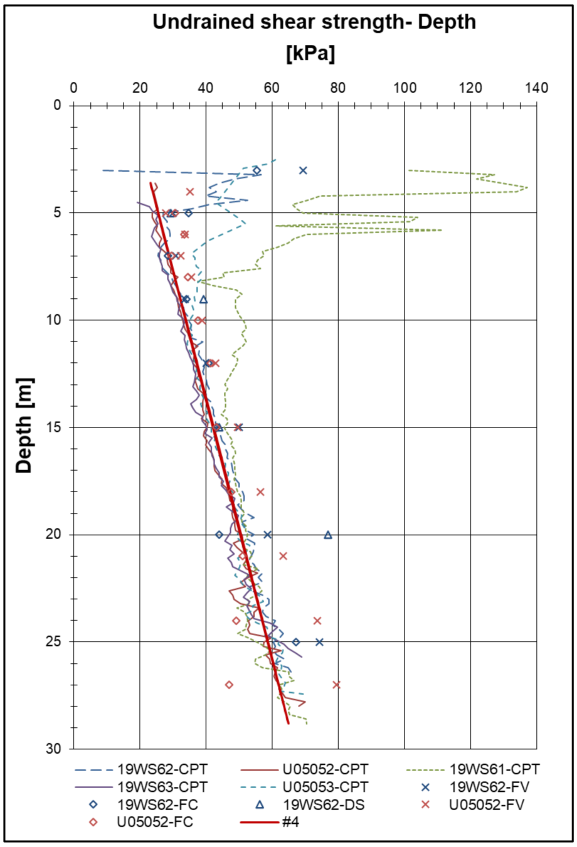
**Equation 59.** Determination of the variable $s_X^2$.

$$s_X^2 = \frac{\sum_{i=1}^{n}(x_i - \bar{x})^2 - b^2 \sum_{i=1}^{n}(z_i - \bar{z})^2}{n - 2}$$

Where $b$ is the slope of the depth dependent regression line, $\bar{z}$ can be interpreted as the centre of gravity of the measurements $\{x_1, x_2 \dots x_n\}$, $t_{n-2}^{0.95}$ is the t-factor of student's distribution and $s_{\bar{x}|z}$ is the standard deviation of the variability in $\bar{x}$.

Following Equations A2.3 to A2.5, using a Student's t-factor ($t_{n-2}^{0.95}$) of 1.645 and a total of n=559 data points, the characteristic value for the 95% confident mean value (Type A) is presented Figure 47.

**Figure 47** Evaluated nominal value by member #4




*Source: Developed by the authors*

II.3.4 Evaluation of results: determination of representative values

The representative value of a ground property $X_{rep}$ shall be determined from either Equation 60 or 61:

**Equation 60**. Determination of representative values as nominal value [Taken from EN 1997-1/Formula (4.2)].

$$X_{rep} = X_{nom}$$

**Equation 61**. Determination of representative values as characteristic value [Taken from EN 1997-1/Formula (4.1)].

$$X_{rep} = X_k$$

where $X_k$ is the characteristic value of the ground property and $X_{nom}$ is the nominal value of the ground property.

## II.4 Summary and conclusions

This case report presents evaluated nominal and/or characteristic values of four TG-C1 members. It can be concluded that there is a discrepancy between the members in the selection of nominal values in the top and in the bottom of the ground layer that is referred to as Unit 2 (see

**Figure** 45 and Figure 46). Where member #1, compared to member #2–4, has selected a higher value in the top of the soil layer and a lower value in the bottom of the soil layer.

Members #1 and #4 have both relied on a statistical analysis to evaluate the characteristic value (member #4 for a restricted part of the soil layer). Based on calculation of the characteristic value as a 95% confident mean value it is concluded that the value is basically very similar to the mean value, given the large amount of data point. The high amount of data points is highly related to the large amount of CPT-measurements (registered every 0.2 m), and hence, it may be relevant to reduce the amount of data points from CPT by local averaging based on the soil vertical scale of fluctuation. Comparing the pure nominal values given by member #2 and 3 with the characteristic values given by member #1 and 4, the selected nominal values tend to be more 'cautious' (lower than the calculated 95% confident mean value). Another factor that has influenced the results given by the contributing members is that member #1 and 2 performed their evaluation based on level and member #3 and 4 based on depth below the ground surface.

According to EN 1997-1, Annex A, A.3 and Equation A1, the sources of uncertainty should be related to the observed variability value through Equation A2.8.

**Equation 62**. Determination of sources of uncertainty [Taken from EN 1997-1/Formula (A.1)].

$$V_x = \sqrt{V_{x,inh}^2 + V_{x,quality}^2 + V_{x,trans}^2}$$

Where $V_x$ is the coefficient of variation of the observed property value, $V_{x,inh}$ is the coefficient of variation of the property due to inherent ground variability, $V_{x,quality}$ is the coefficient of variation of the measurement error and $V_{x,trans}$ is the coefficient of variation of the transformation error. Notably, when calculating the characteristic value according to EN 1997-1 and the coefficient of variation of the ground property is calculated from the provided measurements, uncertainty due to the quality of measurements and transformation error that is biases will not appear in the variability of the measurements. Therefore, given the results presented in section 3.3, i.e., that the 95% confident mean value is basically the same as the mean value, one may argue whether the evaluated characteristic value take proper account of the quality of measurement ($V_{x,quality}$) and the transformation error ($V_{x,trans}$).

## References

Frank, R., Bauduin, C., Driscoll, R., Kavvadas, M., Krebs Ovesen, N., Orr, T., Schuppener, B.Gulvanessian, H., 2004. *Designers' Guide to EN 1997-1 Eurocode 7: Geotechnical Design - General Rules* (Gulvanessian, H. (ed.)). Thomas Telford, London, UK, DOI: 10.1680/dgte7.31548.

# Annex III: Case 3 – Embankment on sensitivity clay (Perniö, Finland)

## III.1. Description of the case

### III.1.1. General features

Table 40 collects the main general features of Case 3 relevant for the evaluation of representative values.

**Table 40.** General features of Case 3.

| | |
|---|---|
| **Geotechnical structure involved** | Embankment on soft clay |
| **Design situation** | Temporary |
| **Limit state under verification** | Overall stability for future constructions on top |
| **Calculation model** | Stability analysis by limit equilibrium method |
| **Ground property to be determined** | Undrained shear strength |
| **Type of representative value estimate** | Type A: estimate of the mean value (see 4.3.3.3.3) |

*Source: Developed by the authors*

In 2009, the Finnish Transport Agency carried out an embankment failure experiment in Perniö, Western Finland, in collaboration with Tampere University of Technology. The main goal was to study the performance of an embankment built on a very soft clay simulating the situation of a train coming to a standstill. A shallow instrumented embankment, 2.5 m wide and 60 m long, was built on top of a soft sensitive 3-5 m-thick clay deposit. Containers were used to simulate train cars Figure 48). Loading was applied by filling up the containers with sand until failure. Failure occurred two hours after the last loading step. The experiment is described in detail by Lehtonen et al. (2011). A numerical analysis is presented by D'Ignazio et al. (2017).

**Figure 48.** Containers simulating train loading at Perniö before (left) and after (right) failure.

*Source: Lehtonen at al. 2011.*

### III.1.2. The Ground Model

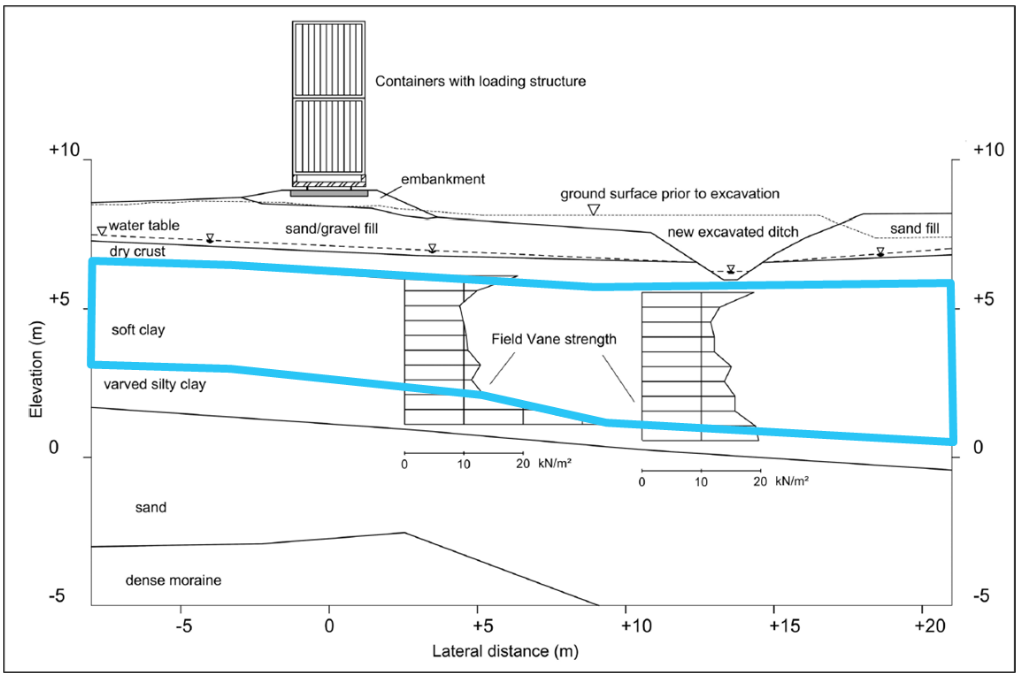
<u>Ground profile</u>

Figure 49 shows the soil stratigraphy at Perniö site, as part of the Ground Model.

From top to bottom, five geotechnical units can be identified:

- Unit 1- Sand/gravel fill;
- Unit 2- Dry (clay) crust:
- Unit 3- Soft clay;
- Unit 4- Varved silty clay;
- Unit 5- Sand/Moraine.

**Figure 49.** Soil stratigraphy at Perniö site.




*Source: D'Ignazio et al. (2016).*

Characteristics of Perniö soft clay (Unit 3):

- Liquid limit = 38-82%;
- Water content = 48-109%;
- Sensitivity, $S_t$ = 23-69;
- $S_u$ remolded < 0.5 kPa;
- OCR = 1.3-1.5 kPa;
- $S_u$ (Field Vane) = 9-12 kPa increasing 1.15 kPa/m with depth;
- Clay content = 48-81%.

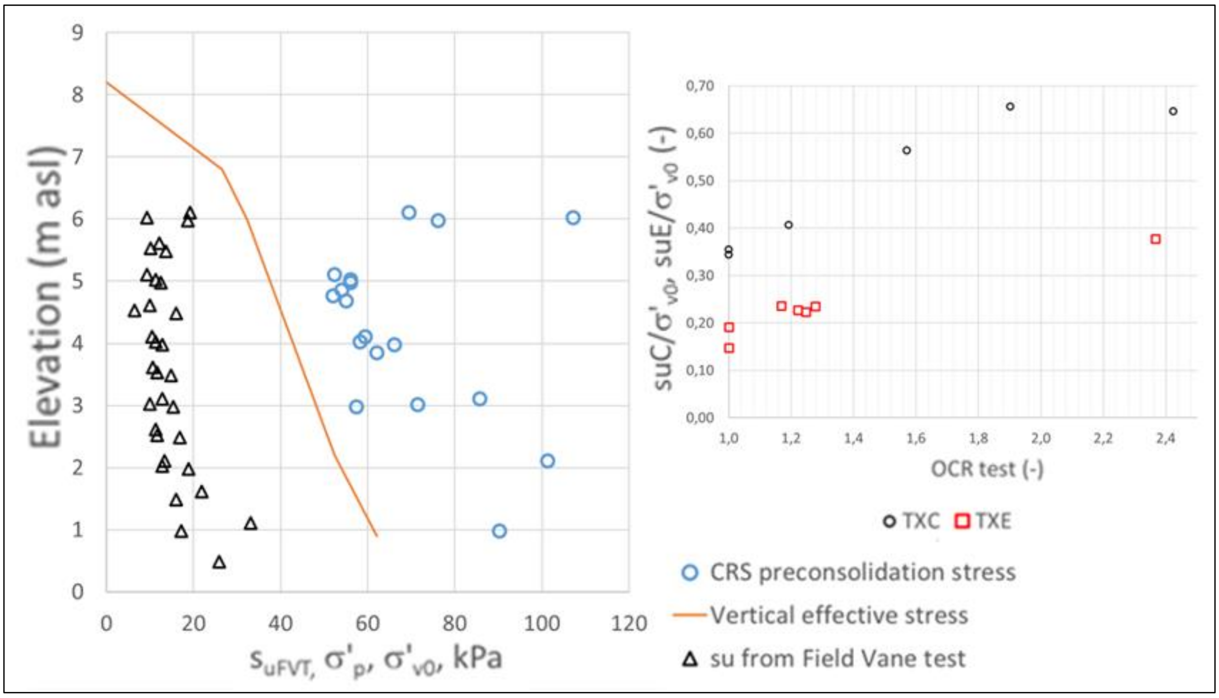
Measured and derived values

For this document, the characteristic and representative value determination of undrained shear strength will be determined only for Unit 3.

To determine the strength characteristic of the soft sensitive clay material, a comprehensive set of in-situ (Field Vane) and laboratory tests (index, consolidation, undrained triaxial compression and extension) was made available.

Figure 50 illustrates the general relevant measured and derived properties of Perniö clay. These include undrained shear strength from Field Vane test and inferred preconsolidation stress from constrain-rate-of-strain (CRS) oedometer tests. Moreover, measured normalized undrained shear strength from undrained triaxial compression (TXC, $s_{uC}$) and extension (TXE, $s_{uE}$) tests vs over consolidation ratio (OCR) from samples taken approximately 50 m off the test area are available. Triaxial samples were reconsolidated to the in-situ stress state. No preloading was applied to the samples in order to preserve the soil's structure. The $S_{uC}$ and $S_{uE}$ values were selected from test results as peak values (if visible) or plateau values from the stress-strain curves**Error! Reference source not found.** and Table 42 summarize the values illustrated in Figure 50.

**Figure 50.** Measured undrained shear strength from Field Vane test and inferred preconsolidation stress from constrain-rate-of-strain (CRS) oedometer tests; (Right) Measured normalized undrained shear strength from undrained triaxial compression (TXC, $su_C$) and extension (TXE, $su_E$) tests vs over consolidation ratio (OCR).




*Source: Developed by the authors.*

**Table 41.** Preconsolidation stress, vertical effective stress and Field Vane undrained shear strength (test area)

| **Preconsolidation stress** | | **Vertical effective stress** ***estimated from unit weight and GWT*** | | **Field Vane test (FVT)** | | |
|---|---|---|---|---|---|---|
| **Elevation** | **CRS preconsolidation stress** | **Elevation** | **Vertical effective stress** | **Elevation** | **$S_u$ vane (no correction factors applied)** | **Vertical effective stress** |
| **m asl** | **kPa** | **m asl** | **kPa** | **m asl** | **kPa** | |
| 5.981 | 76.1 | 8.2 | 0.0 | 5.981 | 18.6 | 32.3 |
| 4.981 | 55.9 | 6.8 | 26.6 | 5.481 | 13.6 | 35.0 |
| 3.981 | 66.1 | 6.0 | 32.2 | 4.981 | 12.5 | 37.7 |
| 2.981 | 57.3 | 2.2 | 52.3 | 4.481 | 16.0 | 40.4 |
| 0.981 | 90.1 | 0.9 | 62.10 | 3.981 | 12.8 | 43.1 |
| 6.109 | 69.4 | | | 3.481 | 14.8 | 45.8 |
| 5.109 | 52.3 | | | 2.981 | 15.4 | 48.5 |
| 4.109 | 59.3 | | | 2.481 | 16.8 | 51.2 |
| 3.109 | 85.7 | | | 1.981 | 18.9 | 53.9 |
| 2.109 | 101.2 | | | 1.481 | 16.0 | 56.6 |
| 4.859 | 54.0 | | | 0.981 | 17.1 | 59.3 |
| 4.689 | 55.0 | | | 0.481 | 25.8 | 62.0 |
| 3.859 | 62.0 | | | 6.109 | 19.1 | 31.6 |
| 6.024 | 107.1 | | | 5.609 | 12.2 | 34.3 |
| 5.024 | 55.9 | | | 5.109 | 9.3 | 37.0 |
| 4.024 | 58.2 | | | 4.609 | 9.9 | 39.7 |
| 3.024 | 71.4 | | | 4.109 | 10.4 | 42.4 |
| 4.774 | 52.0 | | | 3.609 | 10.7 | 45.1 |
| | | | | 3.109 | 12.8 | 47.8 |
| | | | | 2.609 | 11.3 | 50.5 |
| | | | | 2.109 | 13.3 | 53.2 |
| | | | | 1.609 | 21.8 | 55.9 |
| | | | | 1.109 | 33.1 | 58.6 |
| | | | | 6.024 | 9.3 | 32.1 |

| | | |
|---|---|---|
| 5.524 | 10.2 | 34.8 |
| 5.024 | 11.3 | 37.5 |
| 4.524 | 6.4 | 40.2 |
| 4.024 | 11.3 | 42.9 |
| 3.524 | 11.6 | 45.6 |
| 3.024 | 9.9 | 48.3 |
| 2.524 | 11.6 | 51.0 |
| 2.024 | 12.8 | 53.7 |

*Source: Developed by the authors*

**Table 42.** OCR vs normalized undrained shear strength with respect to vertical effective stress from undrained triaxial compression and extension tests (50 m off the test area).

| **OCR test** | **$s_{uC}/\sigma'_{v0}$ (TXC)** |
|---|---|
| 1.19 | 0.41 |
| 1.00 | 0.34 |
| 1.90 | 0.66 |
| 1.57 | 0.56 |
| 1.00 | 0.36 |
| 2.42 | 0.65 |

| **OCR test** | **$s_{uE}/\sigma'_{v0}$ (TXE)** |
|---|---|
| 1.00 | 0.19 |
| 1.22 | 0.23 |
| 1.00 | 0.15 |
| 2.37 | 0.38 |
| 1.17 | 0.24 |
| 1.28 | 0.23 |
| 1.25 | 0.22 |

*Source: Developed by the authors*

## III.2. General considerations

The determination of the representative value of undrained shear strength was done applying different procedures by different TG-C1 members, as illustrated in the following chapter:

- Based on engineering judgment and verification by means of literature correlations.
- Based on engineering experience and statistical considerations, according to EN 1997-1/Annex A.

## III.3 Proposed solutions

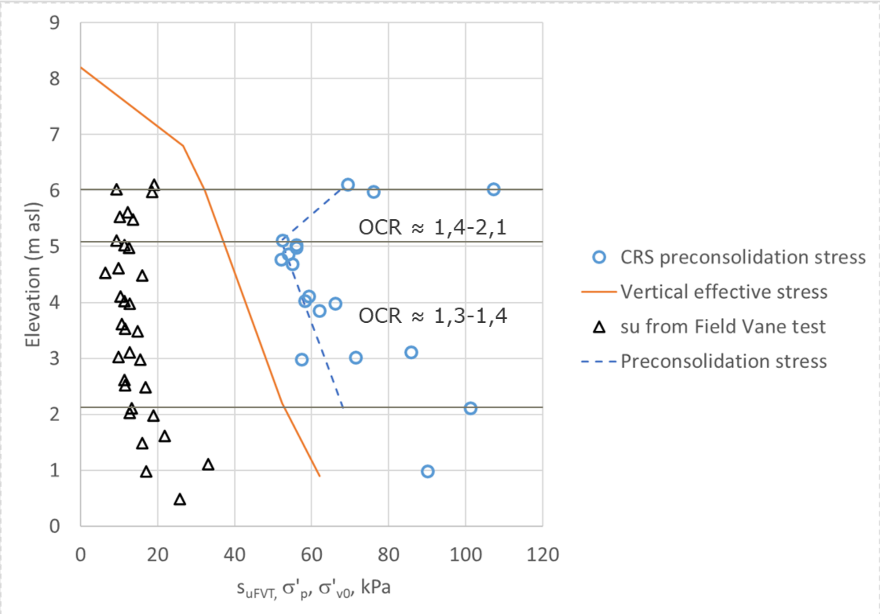
### III.3.1 Solution by TG-C1 Member #1

In this chapter, the determination of nominal and representative value of undrained shear strength of Unit 3 is presented.

The engineering approach used relates the $S_u$ to a more fundamental parameter, i.e. the preconsolidation stress $\sigma'_p$. The values of $S_{uFVT}$, $S_{uC}$, $S_{uE}$ are compared with literature correlations for Su and preconsolidation stress (vertical yield stress) measured from CRS oedometer tests.

Figure 51 illustrates the interpreted preconsolidation stress profile based on engineering judgment. The profile is based on pre-overburden pressures (POP=$\sigma'_p$-$\sigma'_v$) values of 15-35 kPa in the upper clay, located right below the dry crust, and POP=15 kPa in the lower soft clay. Based on POP values, the OCR varies in the range from 1.4 to 2.1 in the upper clay and from 1.3 to 1.4 in the lower clay.

**Figure 51.** Interpreted preconsolidation stress of Perniö clay based on engineering judgment.

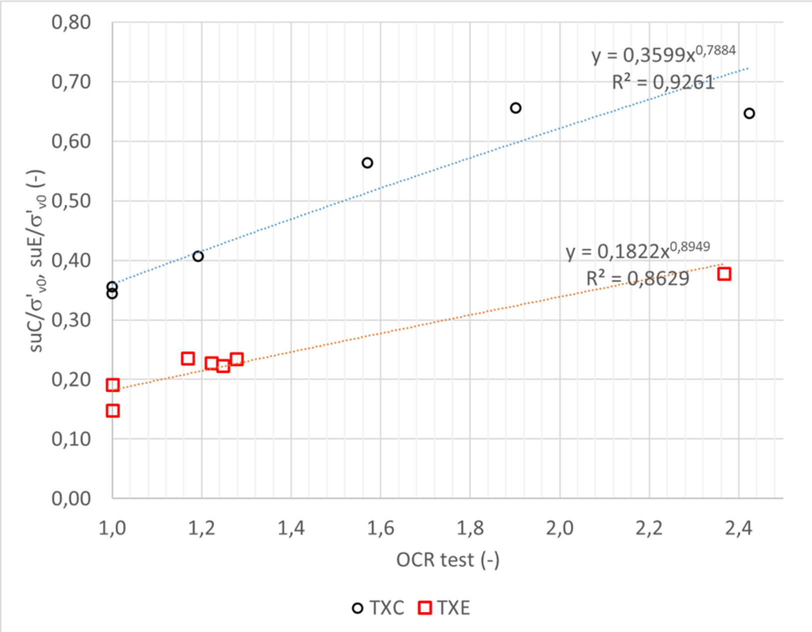



*Source: Developed by the authors.*

Figure 52 illustrates the best fit lines for normalized $S_u$ from TXC ($S_{uC}/\sigma'_{v0}$) and TXE ($S_{uE}/\sigma'_{v0}$) vs OCR, while Figure 53 illustrates the best fit line for normalized mobilized $S_u$ from Field Vane ($S_{u(mob)}/\sigma'_{v0}$) test vs OCR.

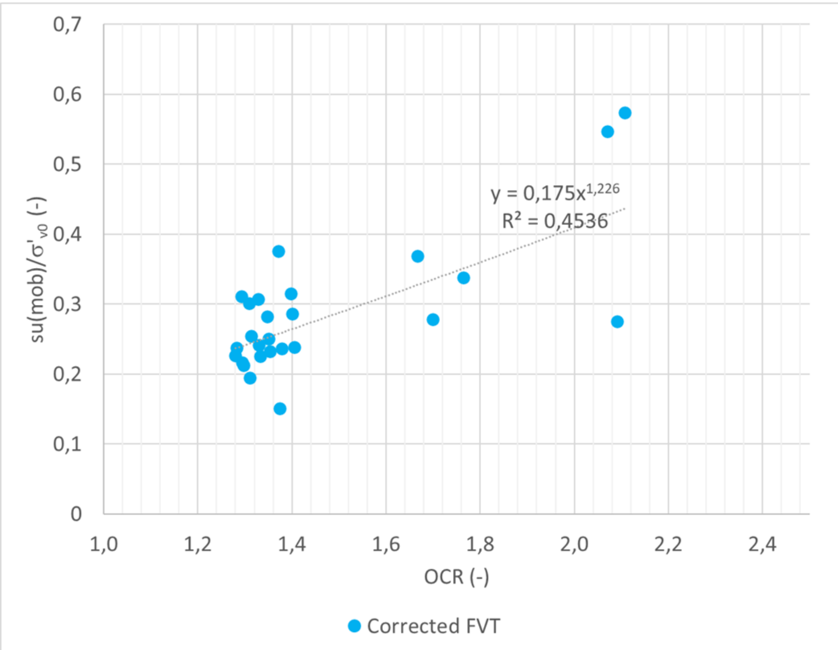
The $S_{u(mob)}$ is defined as the corrected Field Vane strength value. The correction is done by multiplying $S_{uFVT}$ by a factor $\mu$ that is a function of the liquid limit ($w_L$) of the soil, as defined in the Finnish Embankment Stability Guidelines as $\mu$ = 1.5/(1+$w_L$) (Liikennevirasto 2018). For Perniö, an average $\mu$≈0.95 is found. The $S_{u(mob)}$ is assumed to be representative of direct simple shear (DSS) conditions, hence $S_{u(mob)}$≈$S_{uDSS}$.

**Figure 52.** Best-fit lines for suC/s'v and suE/s'v vs OCR.




*Source: Developed by the authors.*

**Figure 53.** Best-fit line for su(mob)/σ'$_{v0}$.




*Source: Developed by the authors.*

It can be noticed how the Field Vane test results show a larger scatter (lower $R^2$) compared to the triaxial data. Therefore, the $S_{u(mob)}$ is compared with literature correlations for Finnish clays.

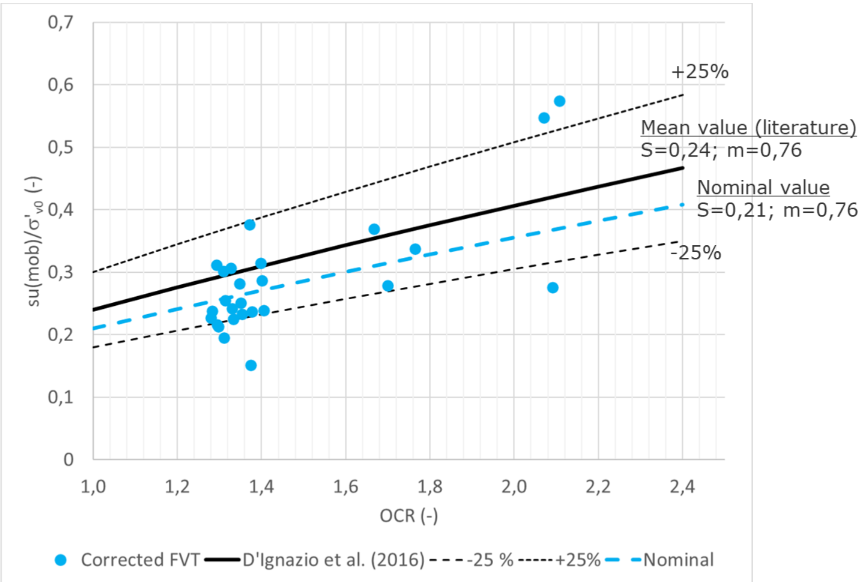
D'Ignazio et al. (2016) proposed, for Finnish clays:

**Equation 63.** Determination of undrained shear strength, for Finnish clays

$$\frac{s_{u(mob)}}{\sigma'_v} = 0.24OCR^{0.76}\,; V_X \approx 0.25$$

It can be observed how most of the $S_{u(mob)}$ data from Perniö clay fall into the lower bound of the correlation by D'Ignazio et al. (2016). A nominal value of $S_{u(mob)}$ vs OCR is established based on engineering judgment, following the model by D'Ignazio et al. (2016).

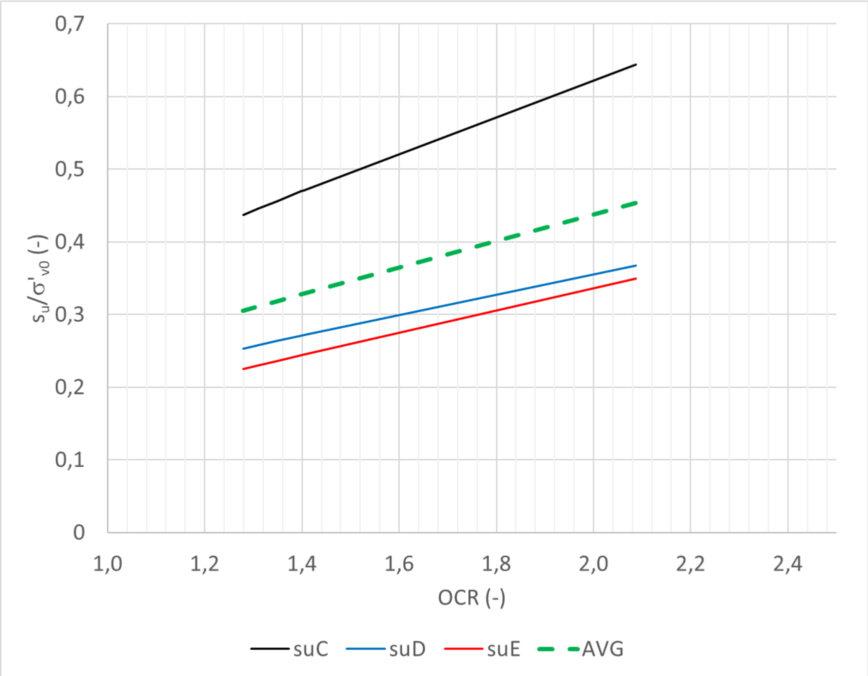
**Figure 54.** Comparison of su(mob)/s'$_v$ vs OCR of Perniö clay with a literature correlation for Finnish clays.




*Source: Developed by the authors.*

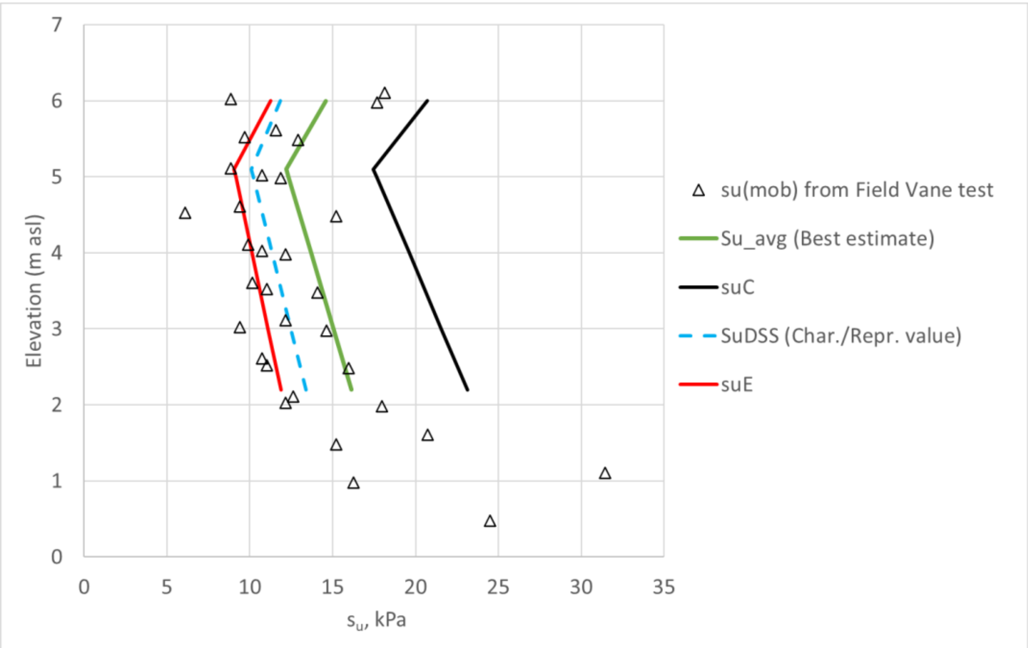
The undrained shear strength profile of Unit 3 at Perniö is then established as the average of $S_{uC}$, $S_{uDSS}$ (or $S_{u(mob)}$) and $S_{uE}$, as illustrated in Figure 55 and Figure 56. This approximation is compatible with the shape of the slip surface observed in the failure test (Lehtonen et al. 2015; D'Ignazio et al. 2017).

**Figure 55.** $s_u/s'_{v0}$ vs OCR of Perniö clay.




*Source: Developed by the authors.*

**Figure 56.** $s_u$ vs elevation of Perniö clay (Unit 3).

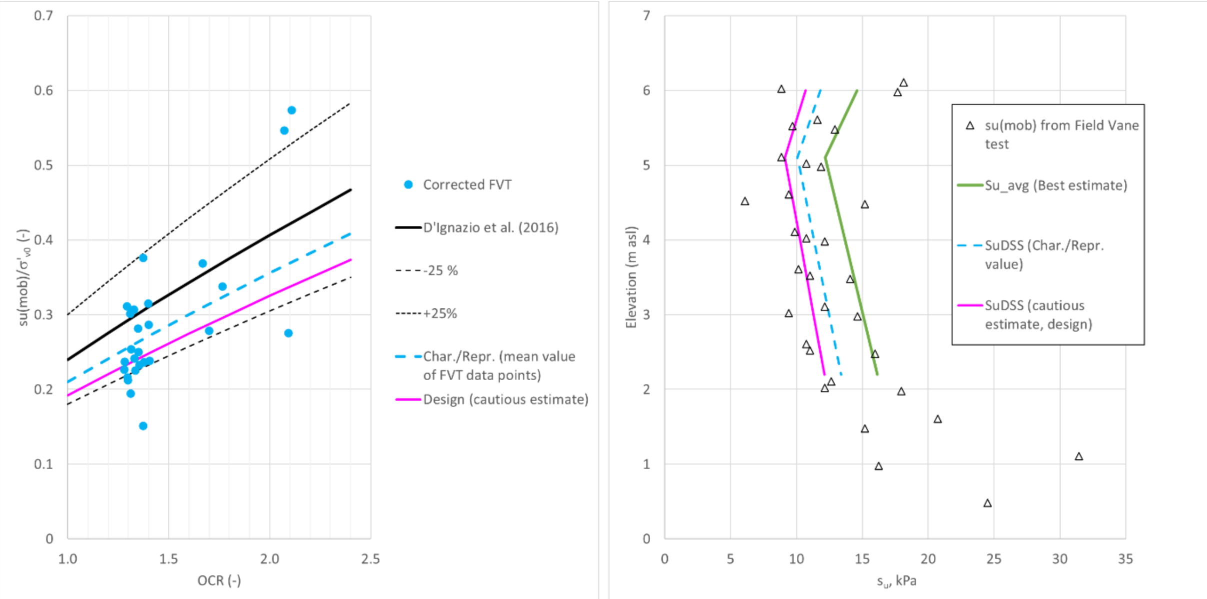



*Source: Developed by the authors.*

The $S_{uAVG}=(s_{uC}+s_{uDSS}+s_{uE})/3$ appears to represent of a Best Estimate value of the data. The $S_{uDSS}$ profile appears to be more in line with a characteristic value of $S_{uAVG}$ of Unit 3, here coinciding with *a)* the mean value of the site-specific field vane points based on engineering judgment and *b)* the nominal value of $S_{uDSS}$ of Unit 3. The judgment-based characteristic $S_{uDSS}$ value roughly corresponds to 0.82 $S_{uAVG}$. In this case, nominal and characteristic values are coincident and correspond to $s_{uDSS}$.

The selected *characteristic* $S_u$ here represents what is current practice in Finland, i.e. determine $S_u$ according to corrected Field Vane test results. The characteristic value is then taken as representative value. In design, the designer might select a more cautious characteristic value, as illustrated in Figure 57.

**Figure 57.** Evaluation of cautious (design) $S_u$ profile (Unit 3).




*Source: Developed by the authors.*

## III.3.2 Solution by TG-C1 Member #2

<u>Nominal values</u>

For the determination of nominal values of undrained shear strength, the Designer made the following considerations on the available data. The designer based the evaluation on triaxial test results and evaluated the anisotropic undrained shear strength ($c_u$=$S_u$) as a function of OCR and vertical effective stress (p'0=s'v0) based on literature for Norwegian clays, leading to the following expressions in Equation 64.

**Equation 64**. Determination of anisotropic undrained shear strength.

- $c_{uCk} = 0{,}38 * OCR^{0{,}70} * p_0^{'}$
- $c_{uEk} = 0{,}19 * OCR^{0{,}8} * p_0^{'}$
- $c_{uDSSk} = 0{,}29 * OCR^{0{,}75} * p_0^{'}$

Table 43 illustrates the effective vertical stress ($p'_0$), preconsolidation stress ($p'_c$), OCR and the nominal values of anisotropic undrained shear strength at different elevations based on the expressions above.

**Table 43.** Effective vertical stress (p'0), preconsolidation stress (p'c), OCR and nominal values of anisotropic undrained shear strength at different elevations.

| Elevation (m) | $p'_0$ | $p'_c$ | OCR | $c_{uCk}$ (kPa) | $c_{uEk}$ (kPa) | $c_{uDSSk}$ (kPa) |
|---|---|---|---|---|---|---|
| 2.2 | 52.3 | 78 | 1.49 | 26 | 13.6 | 20 |
| 6 | 32.2 | 69 | 2.14 | 20 | 11 | 17 |

*Source: Developed by the authors*

Field Vane test results were not considered in the assessment. The quality of the data is unknown, although it is assumed to be good.

Characteristic values

For the determination of characteristic values of undrained shear strength, FV tests and triaxial tests were considered. The OCR was assumed to be constant within the geotechnical unit. A statistical method was then used to find the characteristic normalized shear strength, $c_{uk}/p_0'$.

The dataset includes 32 FV and 13 TX tests (45 in total), leading to $k_n$ = 0,26. The cu was assumed to be in the lower range for the normalized parameter. Both FV-tests and triaxial tests are assumed equally valid and normal distribution is assumed, leading to:

**Equation 65.** Determination of characteristic normalized shear strength

$$C_{uk}/p_0' = c_{u\_mean}/p0' \ (1\text{-}k_n V_x) = 0{,}33 \ (1 - 0{,}26 \ . \ 0{,}3) = 0{,}30$$

Values of $c_{uk}/p'_0$ are summarized in Table 44.

**Table 44.** Effective vertical stress ($p'_0$), and characteristic value of undrained shear strength at different elevations.

| Elevation (m) | $p'_0$ (kPa) | $c_{uCk}$ (kPa) |
|---|---|---|
| 2.2 | 52.3 | 15.7 |
| 5 | 37.5 | 11.4 |

*Source: Developed by the authors*

To determine the characteristic anisotropic undrained shear strength, the ratios used to derive nominal values shall be used.

## III.4 Analysis of the values

Nominal and characteristic values

Table 45 and Table 46 summarize the nominal and characteristic values. respectively. of undrained shear strength of Unit 3 at two different ground elevations selected by different TG-C1 members.

**Table 45.** Nominal values selected by different TG-C1 members.

| TG-C1 member | Elevation (m) | $S_{uC}$ (kPa) | $S_{uE}$ (kPa) | $S_{uDSS}$ (kPa) |
|---|---|---|---|---|
| #1 | 2.2 | 23.1 | 11.9 | 13.4 |
| | 6 | 20.7 | 11.3 | 11.8 |
| #2 | 2.2 | 26 | 13.6 | 20 |
| | 6 | 20 | 11 | 17 |

*Source: Developed by the authors.*

**Table 46**. Characteristic values selected by different TG-C1 members.

| TG-C1 member | Elevation (m) | $S_{uk}$ (kPa) |
|---|---|---|
| #1 | 2.2 | 13.4 |
| | 5 | 10.2 |
| #2 | 2.2 | 15.9 |
| | 5 | 11.4 |

*Source: Developed by the authors*

It can be seen that the *nominal* values for $S_{uDSS}$ by Member 1 are lower compared to those by Member 2. since Member 2 did not take Field Vane values into account and defined $S_{uDSS}$ as the average of $S_{uC}$ and $S_{uE}$. On the other hand. Member 1 evaluated $S_{uDSS}$ as the mean value of the corrected Field Vane data points.

The *characteristic* value proposed by Member 1 are slightly lower that those proposed by Member 2. in line with the more conservative ratio (0.82 vs 0.92) between characteristic and mean value.

Representative values

The representative values are determined from Equations 11 and 12. So, the last step is to choose the representative value from either the nominal or the characteristic values. Both TG-C1 members selected $X_{rep}$=$X_k$. Nevertheless, designers might make a different choice or even select a more cautious representative value according to their experience, their knowledge of the site and any other constrain from the project.

## III.5 Summary and conclusions

Case 3 illustrated the derivation of nominal and characteristic undrained shear strength at Perniö site in Finland. Available site-specific data consisted of field vane, triaxial compression, triaxial extension and oedometer test results.

Two TG-C1 members proposed nominal and characteristic values, following different approaches.

Member 1 followed a more empirical approach, where nominal and characteristic values were determined based on engineering judgement. For this specific case, nominal and characteristic values coincided and identified with the derived $S_{uDSS}$ profile at the site which is roughly 80% of the average undrained shear strength from different test results.

Member 2 adopted an empirical approach based on triaxial data to derive nominal values and introduced statistics to derive characteristic values.

The nominal values for $S_{uDSS}$ by Member 1 are lower compared to those proposed by Member 2, since Member 2 did not take Field Vane values into account and defined $S_{uDSS}$ as the average of $S_{uC}$ and $S_{uE}$. On the other hand. Member 1 evaluated $S_{uDSS}$ as the mean value of the corrected Field Vane data points. The *characteristic* value proposed by Member 1 are slightly lower than those proposed by Member 2, in line with the more conservative ratio (0.82 vs 0.92) between characteristic and mean value. Finally, both members assumed representative and characteristic values to coincide.

## References


D'Ignazio, M., *Undrained shear strength of Finnish clays for stability analyses of embankments. Ph.D. Thesis*. Tampere University of Technology. Finland,2016, Publication 1412, p. 182, ISBN 978-952-15-3804-9, https://trepo.tuni.fi/handle/10024/114414

D'Ignazio, M., Phoon, K. K., Tan,. S. A., and Länsivaara, T. T. 'Correlations for undrained shear strength of Finnish soft clays', *Canadian Geotechnical Journal*, 53(10), 2016, pp.1628-1645.

Lehtonen, V., 'Instrumentation and analysis of a railway embankment failure experiment', *Research report of the Finnish Transport agency*, 29/2011, Finnish Transport Agency, Helsinki, Finland 2011, p. 62,https://ava.vaylapilvi.fi/ava/Julkaisut/Liikennevirasto/lts_2011-29_instrumentation_and_web.pdf

'Penkereiden stabiliteetin laskentaohje *Liikenneviraston ohjeita* 14/2018' (In Finnish). Title in English: Guidelines for embankment stability calculations, *Instructions from the Finnish Transport Agency,* 14/2018, Finnish Transport Agency, Helsinki, Finland, 2018, p. 55, ISBN 978-952-317-629-4, https://julkaisut.vayla.fi/pdf8/lo_2018-14_penkereiden_stabiliteetin_web.pdf

# Annex IV: Case 4 – Settlements of an embankment on clay peat (Puurs. Belgium)

## IV.1. Description of the case

### IV.1.1. General features

Table 47 collects the main general features of Case 4 relevant for the evaluation of representative values.

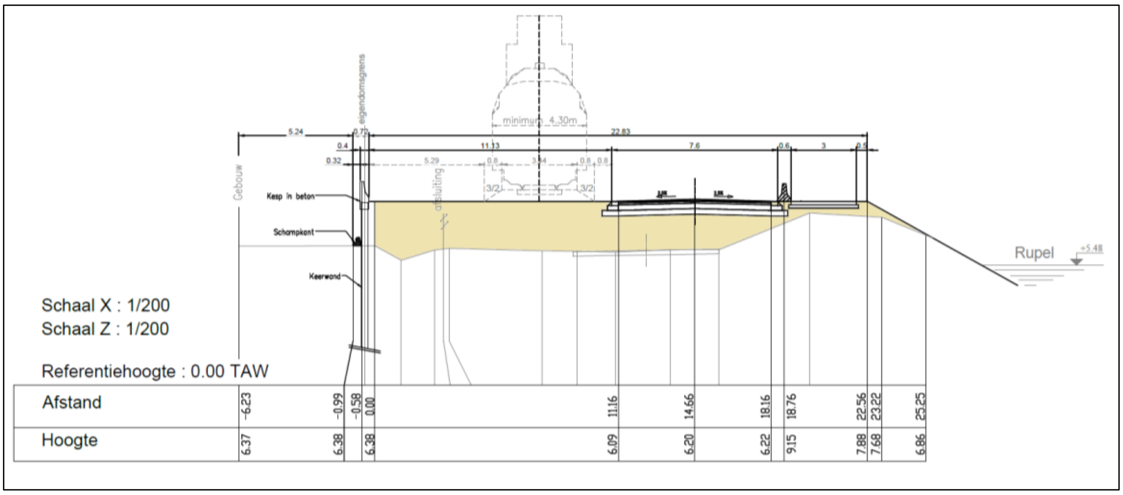
**Table 47.** General features of Case 4.

| | |
|---|---|
| **Geotechnical structure involved** | Embankment |
| **Limit state under verification** | SLS |
| **Calculation model** | Analytical method.<br>one dimensional consolidation by Terzaghi |
| **Ground property to be determined** | Hydraulic conductivity or consolidation coefficient<br>In this case: hydraulic conductivity |
| **Type of representative value estimate** | Type A: estimate of the mean value (see 4.3.3.3.3) |

*Source: Developed by the authors*

The objective of the project is to increase the height of an embankment next to the river Rupel with 2.3 m. For this Annex, the aim is to predict the duration of the settlement of a soft alluvial clay and peat. To do so, it is needed to determine the **representative value of the hydraulic conductivity** to calculate the duration of the settlement (or the residual settlement after a certain period of time). According to the ULS under verification, the representative values must correspond to the estimate of the average value of the property in the volume affected by the limit state (Type A estimate, see 4.3.3.3.3).

**Figure 58.** Cross section of the embankment




*Source: Developed by the authors*

## IV.1.2. The Ground Model

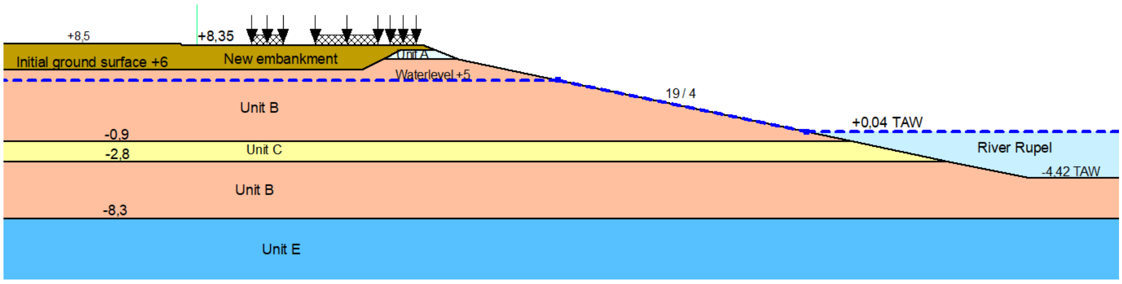
Ground profile

Figure 59 shows the ground profile and Figure 60 shows the results of the most unfavourable CPT.

From top to bottom, the following geotechnical units can be identified:

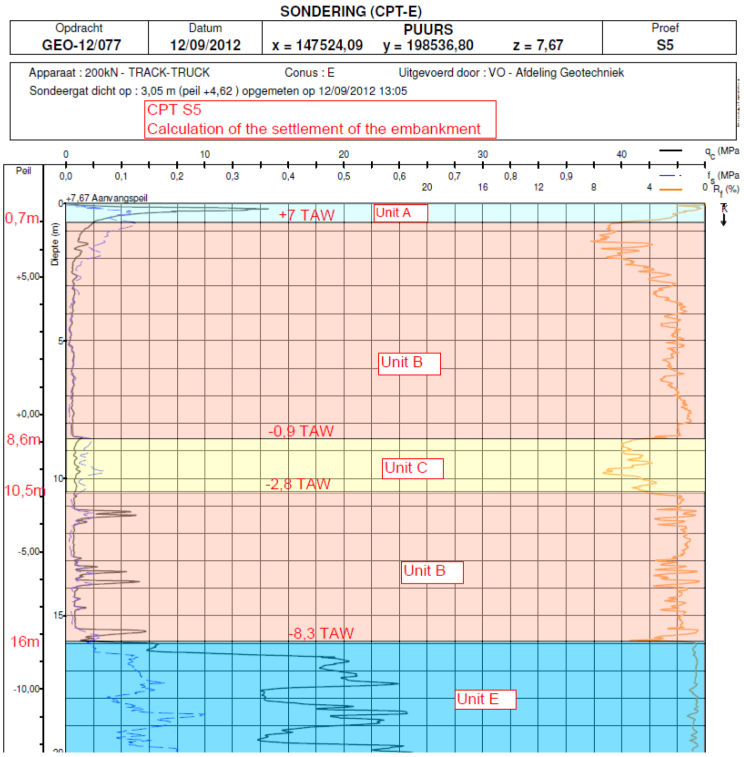
- Unit B is an alluvial clay layer;
- Unit C is an alluvial peat layer;
- Unit E is a sand layer.

**Figure 59.** Cross section with indication of the different geotechnical units.




*Source: Developed by the authors.*

**Figure 60.** Results of the most unfavourable CPT (at the current highest point of the embankment in the Cross section of Figure 59*).*




*Source: Developed by the authors.*

Derived values

For this document, the determination of characteristic and representative values will be done for the Unit B (alluvial clay layer) and Unit C (alluvial peat layer).

The provided derived values originate from:

- site specific back calculations based on the settlement of a smaller embankment;
- site specific dissipation field tests;
- site specific falling head permeability laboratory tests;
- site specific incremental loading oedometer laboratory tests;
- incremental loading oedometer laboratory tests on samples from the same clay unit (Unit B) originating from a nearby site.

**Table 48.** Results for the hydraulic conductivity of the clay and peat unit; derived values vary between 10-7 m/s and 10-10 m/s.

| Data number | **$k_v$ from back calculation of a smaller embankment (Units B & C)** | |
|---|---|---|
| | effective stress (kPa) | kv.BC (m/s) |
| 1 | 40.2 | 1.1E-08 |
| 2 | 40.2 | 6.7E-09 |

| | **, from dissipation tests on clay (Unit B)** | |
|---|---|---|
| | effective stress (kPa) | kv.disB (m/s) |
| 3 | 44.5 | 4.8E-09 |
| 4 | 42.4 | 7.0E-09 |
| 5 | 62 | 1.3E-08 |
| 6 | 90 | 3.3E-09 |
| 7 | 41 | 1.7E-08 |
| 8 | 69 | 7.4E-08 |

| | **$k_v$ from dissipation tests on peat (Unit C)** | |
|---|---|---|
| | effective stress (kPa) | kv.disC (m/s) |
| 9 | 80 | 6.8E-09 |
| 10 | 80 | 3.5E-09 |
| 11 | 84 | 3.3E-08 |

| | $k_v$ from falling head permeability tests on clay (Unit B) | | |
|---|---|---|---|
| | effective stress (kPa) | kv.fhB (m/s) | e () |
| 12 | 36 | 1.2E-10 | 0.9 |
| 13 | 36 | 1.7E-10 | 1.1 |
| 14 | 36 | 3.8E-10 | 2.0 |
| 15 | 36 | 3.2E-10 | 1.2 |
| 16 | 36 | 2.8E-10 | 0.9 |
| 17 | 36 | 5.7E-10 | 1.2 |
| 18 | 36 | 3.3E-10 | 2.4 |
| 19 | 36 | 2.2E-10 | 1.2 |
| 20 | 36 | 9.8E-10 | 1.2 |
| 21 | 36 | 1.1E-08 | 2.4 |
| 22 | 36 | 2.9E-10 | 1.1 |

| | kv from falling head permeability tests on peat ( Unit C) | | |
|---|---|---|---|
| | effective stress (kPa) | kv.fhC (m/s) | e () |
| 23 | 36 | 5.2E-10 | 4.6 |
| 24 | 36 | 1.2E-09 | 3.8 |
| 25 | 36 | 1.2E-09 | 5.1 |

| | kv from IL-oedometer tests on clay (Unit B) | | |
|---|---|---|---|
| | effective stress (kPa) | kv.ilB (m/s) | e (-) |
| 26 | 60 | 3.1E-10 | 0.70 |
| 27 | 120 | 2.0E-10 | 0.65 |
| 28 | 240 | 5.1E-10 | 0.60 |
| 29 | 60 | 2.9E-10 | 0.86 |
| 30 | 120 | 2.0E-10 | 0.81 |
| 31 | 240 | 1.1E-10 | 0.73 |
| 32 | 60 | 2.3E-10 | 0.86 |

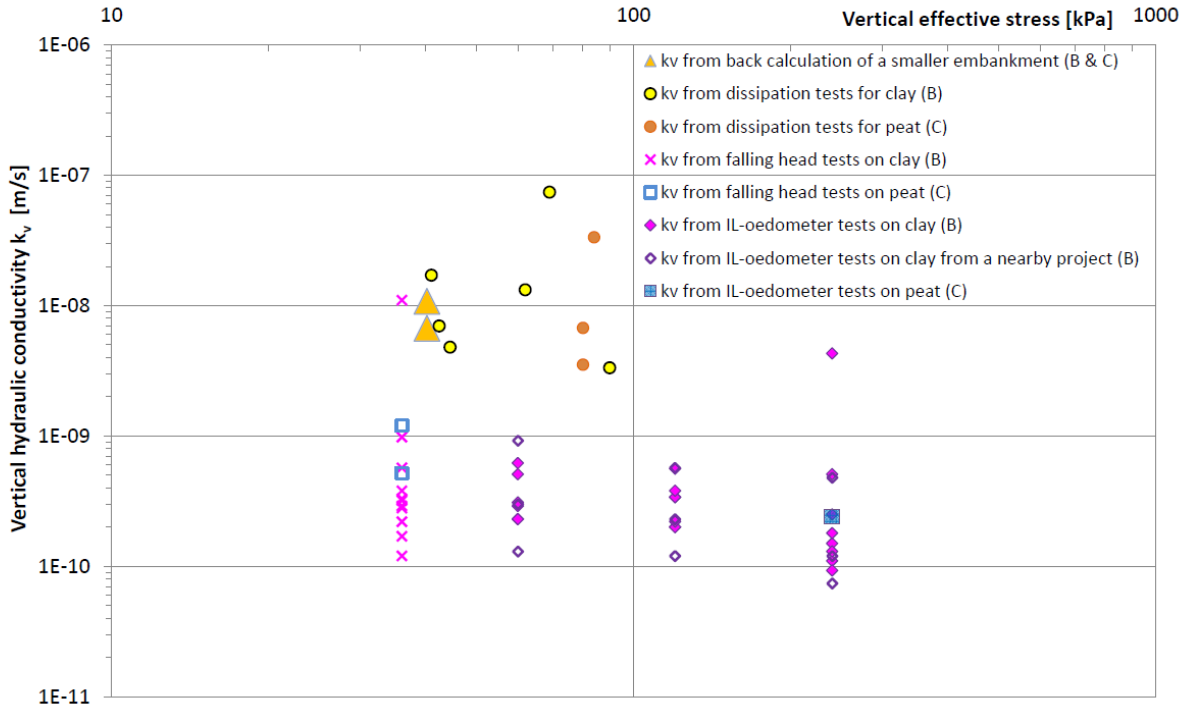
| | | | |
|---|---|---|---|
| 33 | 240 | 9.3E-11 | 0.64 |
| 34 | 60 | 5.1E-10 | 1.13 |
| 35 | 120 | 3.4E-10 | 1.06 |
| 36 | 240 | 1.5E-10 | 0.93 |
| 37 | 60 | 6.2E-10 | 1.11 |
| 38 | 120 | 5.6E-10 | 1.07 |
| 39 | 240 | 1.8E-10 | 0.98 |
| 40 | 60 | 2.3E-10 | 0.95 |
| 41 | 120 | 2.2E-10 | 0.87 |
| 42 | 240 | 1.3E-10 | 0.78 |
| 43 | 120 | 3.4E-10 | 0.83 |
| 44 | 240 | 2.5E-10 | 0.78 |
| 45 | 120 | 3.8E-10 | 2.23 |
| 46 | 240 | 4.3E-09 | 0.96 |

| | **kv from IL-oedometer tests on peat (Unit C)** | | |
|---|---|---|---|
| | effective stress (kPa) | kv.ilC (m/s) | e (-) |
| 47 | 240 | 2.4E-10 | 7.28 |

| | **kv from IL-oedometer tests on clay from a nearby project (Unit B)** | | |
|---|---|---|---|
| | effective stress (kPa) | kv.ilB (m/s) | e (-) |
| 48 | 60 | 9.2E-10 | 1.70 |
| 49 | 120 | 5.7E-10 | 1.57 |
| 50 | 240 | 4.8E-10 | 1.37 |
| 51 | 60 | 1.3E-10 | 1.26 |
| 52 | 120 | 1.2E-10 | 1.17 |
| 53 | 240 | 7.4E-11 | 1.05 |
| 54 | 60 | 3.0E-10 | 1.43 |
| 55 | 120 | 2.3E-10 | 1.29 |
| 56 | 240 | 1.2E-10 | 1.13 |

*Source: Developed by the authors*

**Figure 61.** Derived values for the vertical hydraulic conductivity $k_v$ of Unit B (clay) and Unit C (peat) as a function of vertical effective stress for various test methods.




*Source: Developed by the authors.*

## IV.2. General considerations

This example is primarily intended for illustration purposes, rather than as recommendation. Depending on the design situation and the local boundary conditions, other assumptions may be more suitable for a specific application. To determine one representative value based on a combination of different sources that derive the same ground property, certain assumptions should be made. In particular, the weight of each derived value requires careful consideration. In this case, the determination of the characteristic values of hydraulic conductivity $k_v$ was done considering every geotechnical unit and every test method as a separate data set.

**Table 49.** Overview and comments of the provided derived values.

| **Test** | **#Data points clay** | **#Data points peat** | **Comment** |
|---|---|---|---|
| Back calculation | 2 | | These values represent an average value from a certain soil volume and contains uncertainties in assumption on drainage distance and model uncertainty.<br>Back calculation rather gives a **best estimate value** for Units B and C together. In this case, the selected representative value should be smaller than the best estimate value. |

| | | | |
|---|---|---|---|
| CPTU dissipation tests | 6 | 3 | For CPTU values, it is generally assumed that dissipation is dictated by horizontal seepage, and thus derived values refer to $k_h$ in horizontal layered ground. In this case, a $k_h/k_v$ conversion factor is needed, which could vary significantly.<br>To derive the values for $k_v$, a ratio of $k_h/k_v$ equal to 2 was assumed; however, $k_h/k_v$ could potentially be larger. The results imply that the hydraulic conductivity is 10-100 times higher in the dissipation test compared to laboratory tests. |
| Falling head permeability tests | 11 | 3 | These measured values are the most direct measurements of vertical conductivity in this case (derived using Darcy's law), but they should be corrected for effects of permeability. Values show a scatter of about ±80% from the mean value. One outlier, at approximately 1E-8 m/s, is assumed to be influenced by sample disturbance (or, perhaps, for the presence of silt lens). |
| Incremental loading-oedometers | 21 + 9 | 1 | Test results appear to be consistent with falling head permeability tests. Data from a nearby project is included in the dataset. However, the results are assumed to be influenced by sample disturbance (or perhaps. for the presence of silt lens). |

*Source: Developed by the authors*

Evaluation

In this case, there is a large difference between derived values from field tests and laboratory tests. Photos of the samples tested in the laboratory could provide more information whether or not there was a focus on 'most quality' clay samples in the laboratory, withdrawing silt lenses which may have resulted in (overly) cautious small values for $k_v$.

Consideration should be given to the mechanism of the limit state, which may contribute to an averaging of ground properties or which is characterised as a local problem caused by heterogeneity. In a consolidation case, lower permeabilities have a more severe effect on the total behaviour (compared to a seepage problem where higher permeabilities are dominating).

On other hand, the coefficient of variation Vx is unknown and the occurrence of the limit state in study is insensitive to the spatial variability of the ground property (possible regions of higher permeability are small in relation to compressed region. and most probably closed around their perimeter by soil of lower permeability).

If the aim is to accurately find the lowest characteristic $k_v$ in both the clay and peat units, the laboratory tests should be the only ones used, since back calculation and dissipation tests will also reduce heterogeneity effects and yield relatively rather higher values.

Tables B.2 B3 and B.4 in EN 1997-2. Annex B 'Applicability of field investigation and laboratory tests' do not include any references to the determination of hydraulic conductivity, as developed in EN1997-2, Clause 11.

## IV.3 Analysis of the values

### IV.3.1 Management of the derived values

A correction for temperature and viscosity should be applied (if not already included in the derived value). However, when conducting an overall assessment of the case, consideration must be given to scale effects, model uncertainty, uncertainty of real vertical drainage distances in situ, and the impact of horizontal drainage.

Due to the small value range that is likely to yield negative values, the log-normal distribution seems the most convenient in this case. A log-transformation is used to transform the skewed data to approximately conform to normality. Subsequently, the method for depth- or stress-dependent normally distributed parameters described in Chapter 5.2 can be applied on the transformed data.

### IV.3.2 Selection of 'Nominal Values'

Five different TG-C1 members selected the nominal value for this case; it can be seen in Table 50.

**Table 50.** Nominal values selected by different TGC1 members.

| **TG-C1 member** | **Hydraulic conductivity (m/s)** | | |
|---|---|---|---|
| | **Unit B** | **Unit C** | **Units B & C** |
| M 1 | --- | --- | $k_{v.max}$ = 1 E-7; $k_{v.min}$ = 1 E-10 |
| M 2 | 2 E-10 | 5 E-10 | --- |
| M 3 | Upper part:6 E-9<br>Lower part: 5E-9 | 1 E-8 | --- |
| M 4 | --- | --- | 1 E-9 |
| M 5 | --- | --- | 1.5 E-9 |

*Source: Developed by the authors*

The main difference lies in the weight given to various data sources and the assessment of how cautiously the value should be chosen.

### IV.3.3 Evaluation of 'Characteristic Values'

Determination procedure

As the clay unit (Unit B) and the peat unit (Unit C) are identified as different geotechnical units in the Ground Model of this case, the results are evaluated separately. The determination procedure that will be followed is to calculate a separate characteristic value for each test method and then weight these characteristic values based on engineering judgement to choose a nominal value as representative value.

Results from falling head permeability laboratory tests

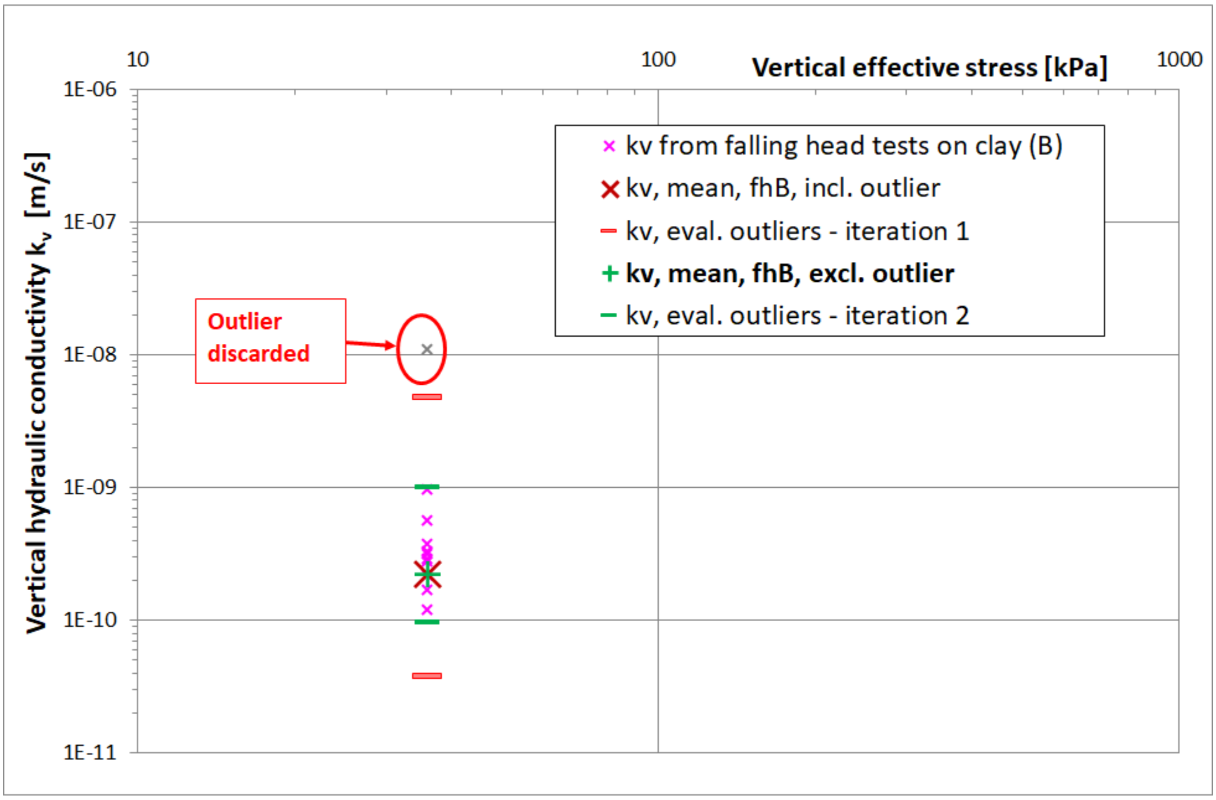
Table 51 collects the values when formulas A.5 to A.7 in EN 1997-1. Annex A (for log-normal distribution) are applied on the falling head permeability test results for the clay layer (Unit B). In a first iteration, outliers are evaluated based on the calculated mean value ± 2 times the standard deviation. Datapoint 21 is identified as an outlier which is then discarded in the second iteration. Excluding the outlier, the characteristic value did not change in this case: $\mathbf{k_{v.\,mean.\,fhB}}$ **= 2.2E-10 m/s**.

**Table 51.** Calculation of the characteristic value based on falling head permeability test results for the clay layer (Unit B).

| **Results for the clay layer (Unit B).** | **Iteration 1 including outlier(s)** | **Iteration 2 outlier excluded** |
|---|---|---|
| **Number of data (n)** | 11 | 10 |
| $\mathbf{t_{n-1}^{95\%}}$ | 1.812 | 1.833 |
| $\mathbf{k_n}$ | 0.546 | 0.580 |
| $\mathbf{s_{Y.ln}}$ | 1.213 | 0.590 |
| $\mathbf{V_X}$ | 750% | 80% |
| $\mathbf{V_Y}$ | 5.6% | 2.7% |
| $\mathbf{k_{v.\,mean.\,fhB}}$ **(m/s)** | **2.2E-10** | **2.2E-10** |

*Source: Developed by the authors*

**Figure 62.** Characteristic mean value for hydraulic conductivity $k_v$ of unit B based on falling head permeability tests.




*Source: Developed by the authors.*

Although there are only three derived values for the peat layer (Unit C), the same formulas $A$.5 to A.7 in EN 1997-1. Annex A are applied, obtaining the following characteristic value: **$k_{v. mean. fhC}$ = 4.0E-10 m/s**, as shown in Table 52.

Because of the low number of derived values, this characteristic value should be used very carefully. Due to this fact, it was considered better to choose a more cautious nominal value instead.

**Table 52.** Calculation of the characteristic value based on falling head laboratory tests for the peat [Unit (C)].

| **Results for the peat (Unit C)** | **Falling head permeability test** |
|---|---|
| **Number of data (n)** | 3 |
| **$t_{n-1}^{95\%}$** | 2.920 |
| **$k_n$** | 1.686 |
| **$s_{Y.ln}$** | 0.483 |
| **$V_x$** | 43% |
| **$V_Y$** | 2.3% |
| **$k_{v. mean. fhB}$ (m/s)** | 4.0E$^{-10}$ |

*Source: Developed by the authors*

Results from incremental loading oedometer tests

The derived values at different stress levels can be statistically evaluated in accordance with the method for depth or stress dependent parameters, as developed in Sections 5.2 and 5.3. Data of a nearby project is included in the dataset.

**Equation 66**. Variation of the ground property over depth (taken from Equation 39 of this document).

$$X(z) = a_0 + a_1 z + \varepsilon$$

Substituting *X* by log($k_v$) and *z* by log($\sigma'_v$)

**Equation 67**. Variation of the ground property over depth for log normal data.

$$\log k_v(\log \sigma'_v) = a_0 + a_1 \cdot \log \sigma'_v + \varepsilon$$

**Equation 68**. Evaluation of the parameters $a_1$, $a_0$ and *se* (by Equation 40 and Equation 43 of this document).

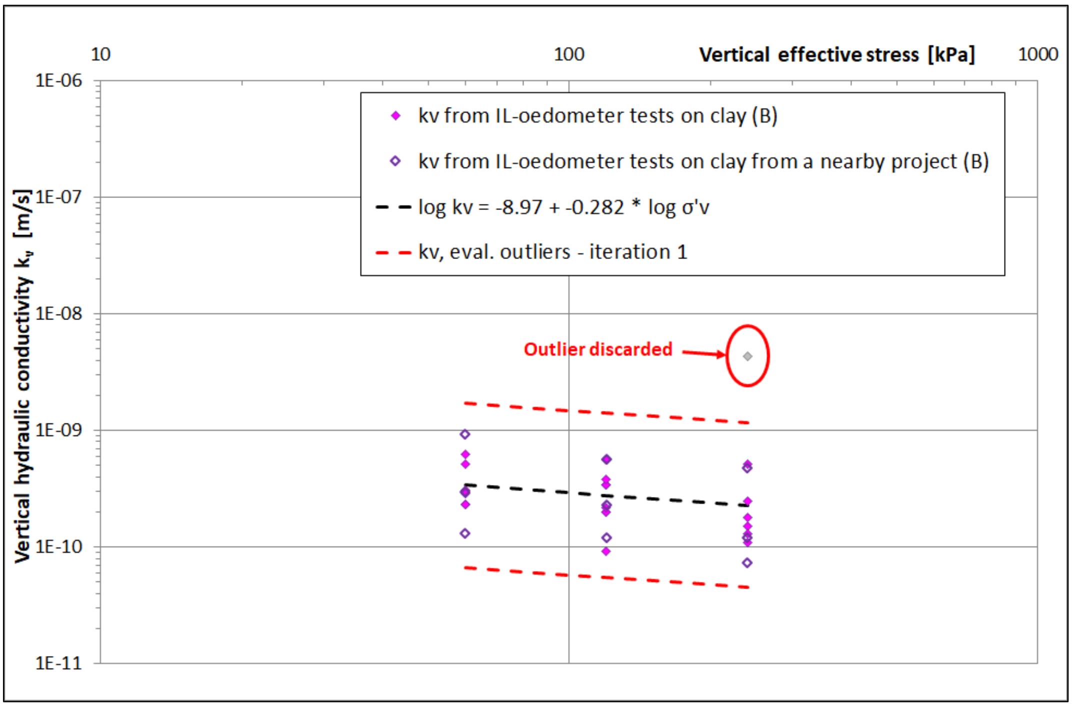
$$a_1 = \frac{\sum_{i=1}^{n}(z_i - \bar{z})(x_i - \bar{x})}{\sum_{i=1}^{n}(z_i - \bar{z})^2} = \frac{-0.508}{1.800} = -0.282$$

$$a_0 = \bar{x} - a_1\bar{z} = -9.56 - (-0.282) \cdot 2.10 = -8.97$$

$$se = \sqrt{\frac{1}{n-2}\sum_{i=1}^{n}[(x_i - \bar{x}) - a_1(z_i - \bar{z})]^2} = 0.352$$

Values outside ±2 *se* can be discarded as outliers (see Figure 63). Unless these outliers are representative (for example, representative silt lenses in the clay layer), they should be discarded, and a new iteration should be made.

**Figure 63.** Evaluation of outliers for incremental loading oedometer test results on clay layer (Unit B).




*Source: Developed by the authors.*

In this case, datapoint 46 is discarded and a new evaluation on the n = 29 remaining datapoints is made.

**Equation 69**. Evaluation of the parameters $a_1$, $a_0$ and *se* (by Equation 40 and Equation 43 of this document).

$$a_1 = \frac{\sum_{i=1}^{n}(z_i - \bar{z})(x_i - \bar{x})}{\sum_{i=1}^{n}(z_i - \bar{z})^2} = \frac{-0.855}{1.719} = -0.497$$

$$a_0 = \bar{x} - a_1\bar{z} = -9.60 - (-0.497) \cdot 2.09 = -8.56$$

$$se = \sqrt{\frac{1}{n-2}\sum_{i=1}^{n}[(x_i - \bar{x}) - a_1(z_i - \bar{z})]^2} = 0.252$$

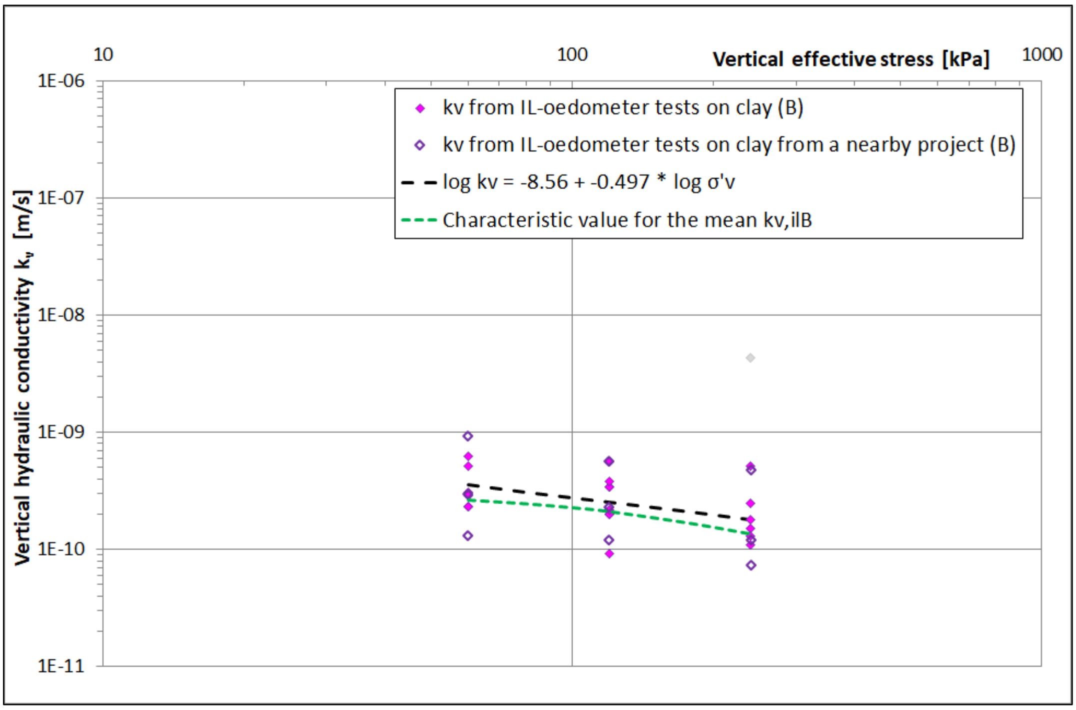
Based on these *n* values, $a_0$, $a_1$ and *se*, the characteristic mean value can be calculated for $k_{v.10°C}$ for clay layer (Unit B) using Equation (41) of Section 5.2.3.

**Equation 70.** Determination of the characteristic mean trend.

$$X_k(z) = \bar{x} + a_1(z - \bar{z}) \pm t_{(n-2)}^{95\,\%} \cdot se\sqrt{\left[\frac{1}{n} + \frac{(z-\bar{z})^2}{\sum_{i=1}^{n}(z_i - \bar{z})^2}\right]}$$

The characteristic curve for the mean, indicated as a green hyperbolic function in Figure 64, gives a cautious value for $k_v$ based on the incremental loading oedometer tests on clay layer (Unit B).

**Figure 64.** Characteristic value for the mean IL-results on clay layer (Unit B).




*Source: Developed by the authors.*

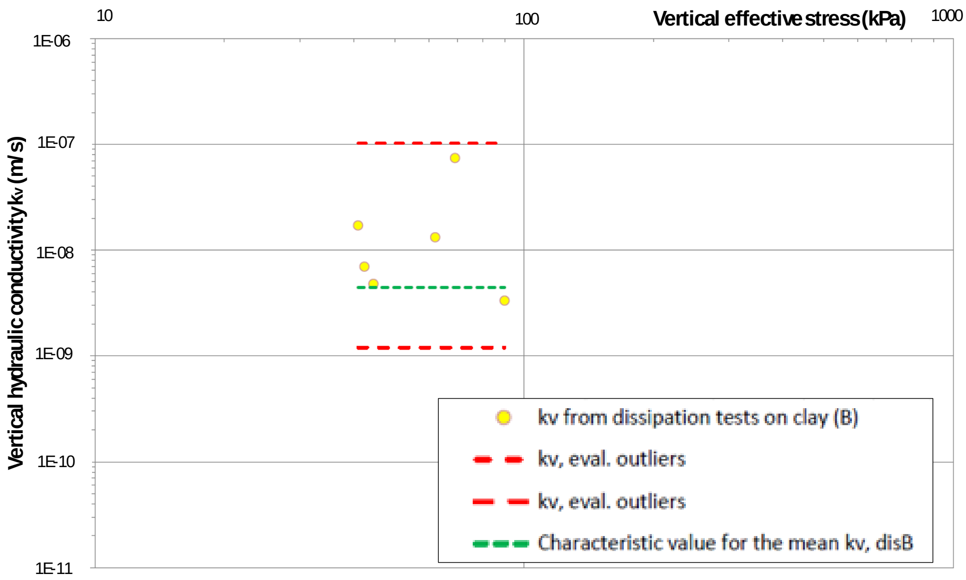
As there is only one result from an incremental loading oedometer test on the peat layer (Unit C). it is not possible to calculate a characteristic value for this unit using this method.

<u>Results from CPTU dissipation tests</u>

It is interesting to compare the characteristic value of 2.2E-10 m/s for Unit B obtained from the laboratory tests with the data from the dissipation tests on Units B and C. Based on the CPTU dissipation tests, the characteristic mean value can be calculated as illustrated in Figure 65 and Figure 66. As there is no clear trend with depth or vertical effective stress, the formulas *A*.5 to A.7 in EN 1997-1. Annex A for lognormal distribution are applied.

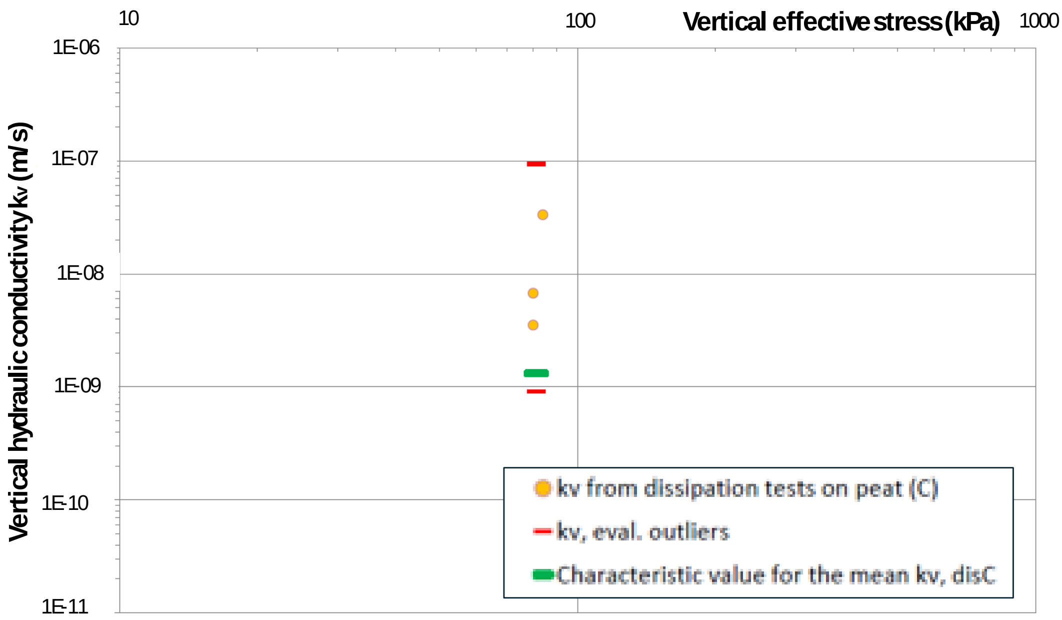
The calculated characteristic value for the mean for Unit B based on six derived values is 4E-9 m/s, but because of a very large Vx equal to 356% and because of the uncertainty on the ratio $k_h/k_v$, it may be advisable to choose a nominal value which is smaller than this calculated value.

**Figure 65.** Characteristic value for the mean dissipation test-results on clay layer (Unit B).




*Source: Developed by the authors.*

**Figure 66.** Characteristic value for the mean dissipation test-results on peat layer (Unit C).




*Source: Developed by the authors.*

Although the CPTU dissipations tests on Units B and C are in line with each other, the characteristic value of Unit C is smaller due to the smaller dataset. Because of the very low number of derived values, this characteristic value of 1E-9 m/s should be used very carefully.

### IV.3.4 Evaluation of results: determination of representative values

Due to the limited number of datapoints for some measurement methods on Unit C, it is questionable to calculate a characteristic value for each test method. However, since the derived values of Unit C correspond well to the regressions on Unit B, it may be acceptable to include Unit C in Unit B for the purpose of determining the representative value of hydraulic conductivity, thus simplifying the Ground Model. Nevertheless, this approach is not further elaborated upon.

Table 53 shows an overview of the values calculated based on the different procedures and data sets.

**Table 53.** Overview of the calculated values.

| Test | Unit B | Unit C | Comment |
|---|---|---|---|
| Back calculation | 1E-8 to 7E-9 m/s | | Back calculation rather gives a best estimate value for Units B and C together. In this case, the selected representative value should be smaller than the best estimate value. |
| CPTU dissipation tests | 4E-9 m/s | 1E-9 m/s | The ratio $k_h/k_v$ was assumed to be 2, however, $k_h/k_v$ could be larger. The number of derived values is small and the values Vx are large. It may therefore be better to choose a smaller nominal value instead. |
| Falling head permeability tests | 2E-10 m/s | 4E-10 m/s | One outlier at appx. 1E-8 m/s assumed to be influenced by sample disturbance (or for example a silt lens).<br>The number of derived values for Unit C is small. |
| Incremental loading-oedometers | 2E-10 m/s | - | Data of a nearby project is included in the dataset. Assumed to be influenced by sample disturbance (or for example a silt lens). The number of derived values for Unit C is too small to calculate a characteristic value. |

*Developed by the authors*

Given the large deviations and to avoid false semblance of accuracy, the characteristic value for geotechnical Unit B based on the laboratory tests can be rounded to 2E-10 m/s, depending on the stress range of interest. Depending on how the laboratory samples were selected, this value can be overly cautious. For instance, in the presented design case, the laboratory may have chosen to take a subsample in a more homogeneous part of the sample, although the samples of the entire unit are heterogeneous.

Compared to the laboratory tests, the results of the CPTU dissipation tests show higher values for $k_v$ and a designer could, for example, choose a nominal value equal to 1E-9 m/s based on these results. If photographs from the laboratory tests would indicate that samples were selected overly cautious, less weight could be given to the value of 2E-10 m/s from the laboratory tests and more weight to the values of the back calculation and the dissipation tests when selecting an overall nominal value. If the aim is to find the mean value of $k_v$ in the clay and peat layers (Units B and C), the back calculation and dissipation tests can be given more weight as they also consider the regions of higher permeability.

A sensitivity analysis using values ranging between 2E-10 m/s (cautious estimate based on laboratory tests) and 1E-8 m/s (best estimate from back calculations) can assist in assessing the effects of $k_v$ on the specific design case. Characteristic values, nominal values and qualitative information from the sensitivity analysis can be combined to one nominal value using engineering judgement. This nominal value can be selected as the representative value.

It is out of the scope of this case to make a preference since its selection is in the hands of the designers, according to their experience, their knowledge of the site and any other constraint from the project. It is however stressed that the representative value should be smaller than the best estimate value from the back calculation as outlined in Chapter 4.7.

Note that when selecting a representative value for consolidation problems, other ground properties can be just as significant for the design as the hydraulic conductivity. For instance, the consolidation coefficient is an additional ground property required to understand time-settlement.

## IV.4 Summary and conclusions

This case report presents values for the hydraulic conductivity of clay and peat layers (Units B and C) derived from various sources.

The coefficient of variation for the hydraulic conductivity is large, however, by detecting possible outliers per data source, Vx values were identified that correspond to the order of magnitude as listed in Table A.1 in EN1997-1, Annex A.

In case a data source has a low number of derived values and/or large Vx, it may be advisable to select a cautious nominal value instead of a characteristic value. Nevertheless, for other sources with a larger number of available derived values, a characteristic value can still be calculated, and outliers can be evaluated within the method.

Significant differences between laboratory tests and field tests were found but possible causes of this difference were discussed in this case. Sufficient information on how the derived values were obtained must be available to make a proper assessment. The characteristic and nominal values for each data source can finally be combined based on engineering judgement to choose one nominal value as representative value for the hydraulic conductivity of a unit.

# Annex V: Case 5 - Stability of a retaining wall (Bucharest, Romania)

## V.1. Description of the case

### V.1.1. General features

The case study described deals with the retaining wall of an excavation pit in Bucharest city. Romania.

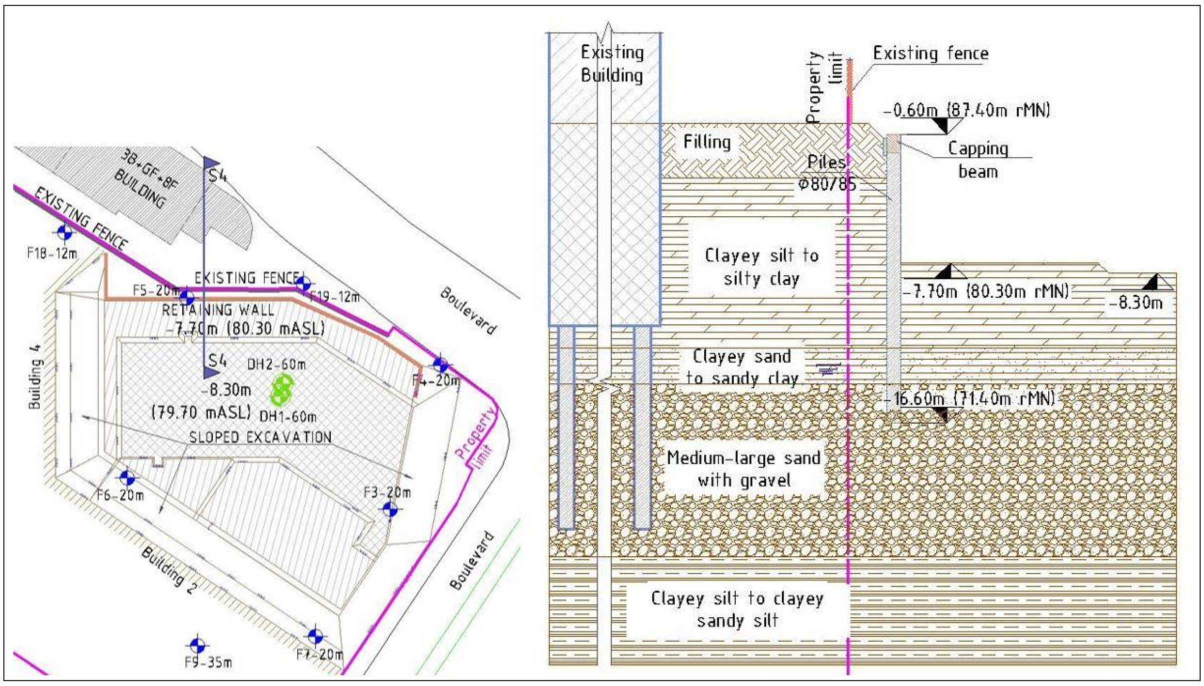
Table 54 collects the main general features of Case 5 relevant for the evaluation of representative values, while Figure 67 shows a layout and a section of the geotechnical structure under study and Figure 68 includes a photograph of the construction site.

**Table 54.** General features of Case 5.

| | |
|---|---|
| **Geotechnical structure involved** | Self-supported (cantilever) pile retaining wall |
| **Design situation** | Transient |
| **Limit state under verification** | SLS – deformation. stability & wall resistance |
| **Calculation model** | FEM analysis |
| **Ground property to be determined** | Unit weight. deformation modulus and total strength (undrained cohesion and friction angle) |
| **Type of representative value estimate** | Type A: estimate of the mean value (see 4.3.3.3.3) |

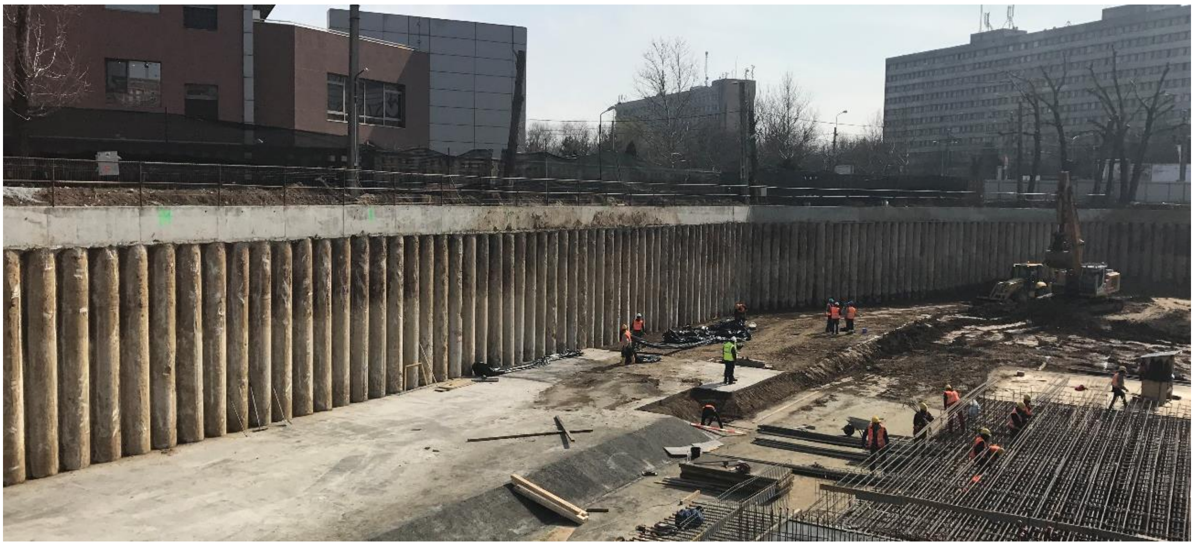
*Source: Developed by the authors*

**Figure 67.** Excavation and retaining system layout (left) and characteristic section (right).




*Source: Developed by the authors.*

**Figure 68.** Picture of the constructed retaining wall of the excavation.



*Source: Picture by Alexandra Ene*

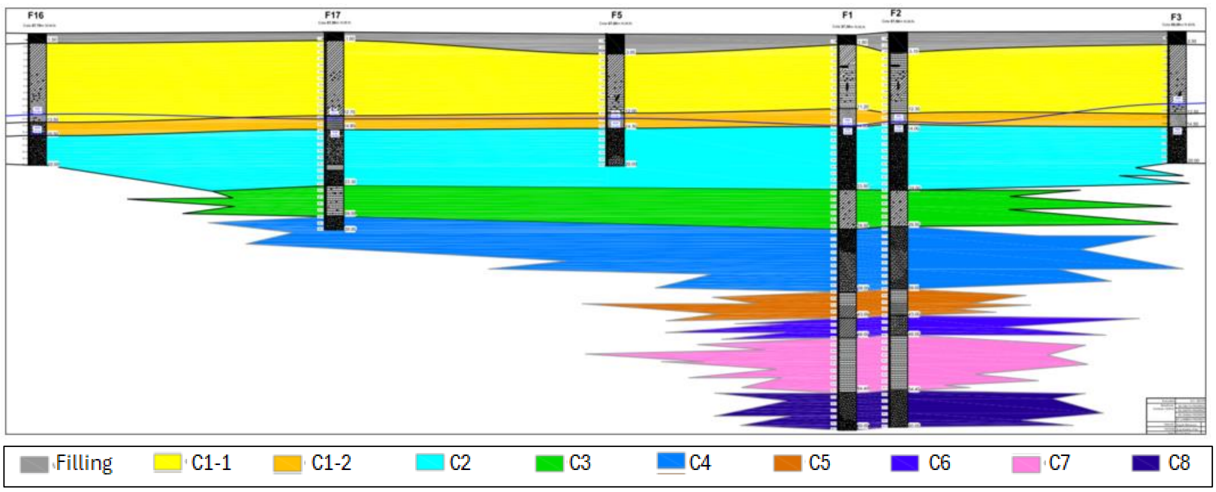
The calculations for the retaining wall were performed by 2D FEM analysis with the total stress assumptions, using undrained shear strength properties and the 'Hardening Soil with small stiffness behaviour Model'.

### V.1.2. The Ground Model

The site ground investigations consisted of seven geotechnical boreholes with depths varying between 20 m and 60 m, with soil sampling and laboratory tests performed on these samples and some SPT (Standard Penetration Test) in the cohesionless layers.

Based on the obtained result, a typical sedimentary deposits terrace structure was identified. as seen in the geological section in Figure 69, for the eastern side of Building 1 (which was supported by retaining wall), which also includes the site ground investigations performed.

**Figure 69.** Geological section as part of the Ground Model.




*Source: Developed by the authors.*

The various geotechnical units identified are the following:

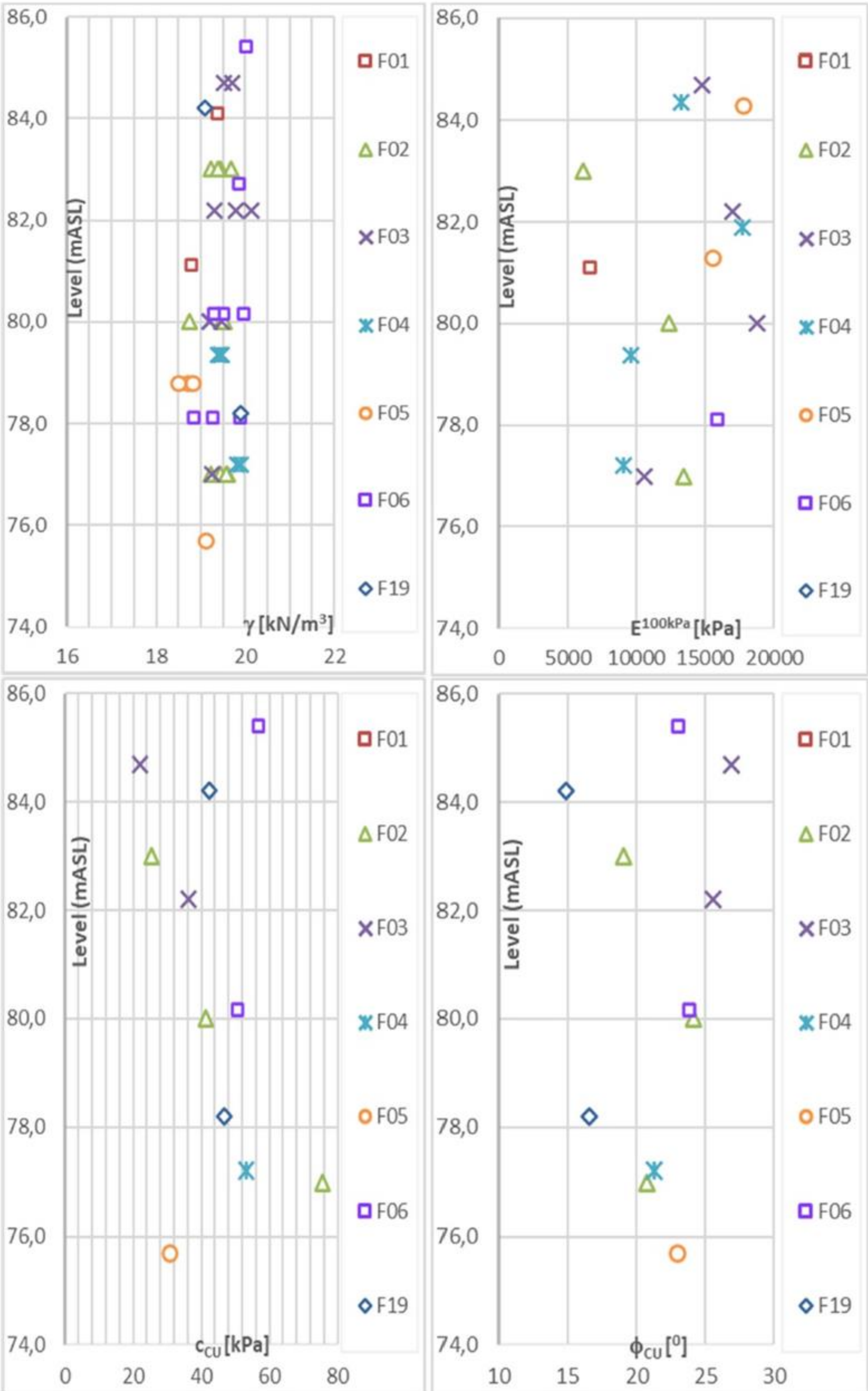
- FILLING (with thickness varying between 1.50 m and 3.10 m), consisting of construction waste material (underground foundations. concrete and reinforced concrete slabs. steel. bricks. charred timber etc.), cemented or incorporated in a stiff clay. sometimes with increased consistency;
- COHESIVE LAYER 1 (C-1). composed of two sub-layers:
  - (C1-1) the upper sub-layer consisting of clayey silt and silty clay, yellowish – brown and reddish – brown, with manganese oxides and iron oxides and limestone concretions, stiff and with increased consistency in some areas (its thickness varying between 9.00 m and 10.80 m); This was assimilated with the typical ‘Bucharest Loam’ layer;
  - (C1-2) the bottom sub-layer consisting of sandy silty clay, sandy clayey silt with few gravel elements, stiff (its thickness varies between 1.70 m and 2.80 m).
- COHESIONLESS LAYER 2, consisting of fine - medium, medium – coarse sand, coarse sand with fine gravel and silty sand, grey – brown, micaceous, submerged, in dense state (its thickness varies between 9.50 m and 10.00 m);
- COHESIVE LAYER 3, consisting of clayey silt and sandy clayey silt, mostly stiff, and stiff too soft at the bottom, grey- brown and grey – brown, with weathered calcareous concretions and manganese oxides (its thickness varies between 5.50 m and 6.00 m);
- COHESIONLESS LAYER 4, consisting of medium – fine sand or fine – medium sand with various sandy silt intercalations, clayey sandy silt or sandy silty clay, grey – yellow, in dense state (about 9.50 m thick);
- COHESIVE LAYER 5 (about 4.00 m thick), composed of two sub-layers:
  - (C5-1) the upper sub-layer consisting of clay and silty clay. grey – purple, with limestone concretions, stiff (about 2.00 m thick);
  - (C5-2) the bottom sub-layer consisting of clayey sandy silt and silty sandy clay, stiff (about 2.00 m thick).
- COHESIONLESS LAYER 6, consisting of fine granular deposits, silty sand and sandy silt, greenish – grey, submerged (about 3.00 m thick);
- COHESIVE LAYER 7,. consisting of silty clay, grey – purple, stiff to very stiff, with limestone concretions and rock fragments, embedded in the structure of the layer, especially at the bottom (about 8.40 m thick);
- COHESIONLESS LAYER 8, consisting of fine – medium sand, gray – greenish, submerged (about 5.60 m thick).

The groundwater level was found at depths varying between 13.20 and 14.50 m, which is below the final excavation level, so it does not involve any uplift concerns.

The statistical description of the geotechnical properties of the ‘Bucharest Loam’ layer (cohesive layer C1-1) is presented here since this was considered as dominant for the case study.

The values of the main geotechnical properties considered for the calculation of the retaining wall are given by depth within the following graphs: unit weight, deformation modulus and cohesion and internal friction angle. The properties for the 'Bucharest Loam' are represented in Figure 70 per geotechnical borehole within the area of the project.

**Figure 70.** Geotechnical properties versus depth per geotechnical borehole.




*Source: Developed by the authors.*

## V.2. General considerations

The following statistical distributions were considered reasonable for the present case study, based on reliability analysis and on physical and mathematical justifications (Ene, 2021):

- Normal distribution with and without prior knowledge assumption - as given in EN 1997-1;
- Lognormal distribution for the deformation modulus (determined based on correlations with the oedometric modulus) – because of the higher coefficient of variation of the sample data, without any prior knowledge assumption;
- Normal distribution resulted from linear regression analyses for the shear resistance properties (tangent of the internal friction angle and cohesion) – justified by the high dependence of the two parameters that are determined from the same test, without any prior knowledge assumption.

According to the ULS under verification, the representative values must correspond to the average value of the property in the volume affected by the limit state (Type A estimate. see 4.3.3.3.3). Thus, all the evaluations were performed for spatial averaging assumption and considering statistical uncertainty of the mean value by Student-t value, by Equation 71.

**Equation 71**. Value of the statistical parameter $k_n$.

$$k_n = t_{n-1}^{95\%}\sqrt{\frac{1}{n}}$$

In a parallel way, some TG-C1 members sent their selected nominal and characteristic values which allows making a comparison with the characteristic values determined through the procedures.

## V.3 Analysis of the values

### V.3.1 Management of the derived values

Part of the soil samples taken from 'Bucharest Loam' layer presented low unit weight as compared to the vast majority. While these are supposed to be real properties of the soil samples confirmed by other indices, they were disregarded as not being representative. The low unit weight soil samples are due to local areas of dry soil, which is also slightly water sensitive and represent a different soil behaviour.

For all the other properties, the layer was considered as statistically homogenous, so it was considered a 'geotechnical unit'. Furthermore, it does not show any trend with depth.

### V.3.2 Selection of 'Nominal Values'

As mentioned before, some TG-C1 members sent their selected nominal values which allows making a comparison among them and with the characteristic values determined in next section. Table 55 collects the values selected.

**Table 55.** Nominal values selected by different TG-C1 members.

| TG-C1 member | Unit weight (kN/m³) | Deformation modulus (kPa) | Cohesion (kPa) | Friction angle (°) |
|---|---|---|---|---|
| 1 | - | - | 25 + 8z ≈ 65 | 0 |
| 2 | - | - | 35 | 20 |
| 3 | - | - | 30 | 25.5 |
| 4 | 19.5 | 12 000 | 40 | 22 |

*Source: Developed by the authors.*

It can be seen that the nominal values are spread with values between 30 kPa and 40 kPa for cohesion and between 20° and 25.5° for friction angle, or a value of 65 kPa with zero internal friction angle.

## V.3.3 Evaluation of 'Characteristic Values'

<u>Normal distribution</u>

For normal distribution, details are given in EN 1997-1. Annex A. For the present case study, where the limit state is governed by the spatial average values of the ground properties, Type A 'Estimate of the mean value' is considered. This comes with two alternatives: Case 1 '$V_{X,known}$' or Case 2 '$V_{X,assumed}$' – considered as prior knowledge assumption and Case 3 '$V_{X,unknown}$' respectively – without any prior knowledge.

Table 56 collects the values of the statistical parameters used in the calculation and the characteristic values obtained, that are represented in Figure 71 together with the derived values.

**Table 56.** Statistics for 'Bucharest Loam' layer considering normal distribution.

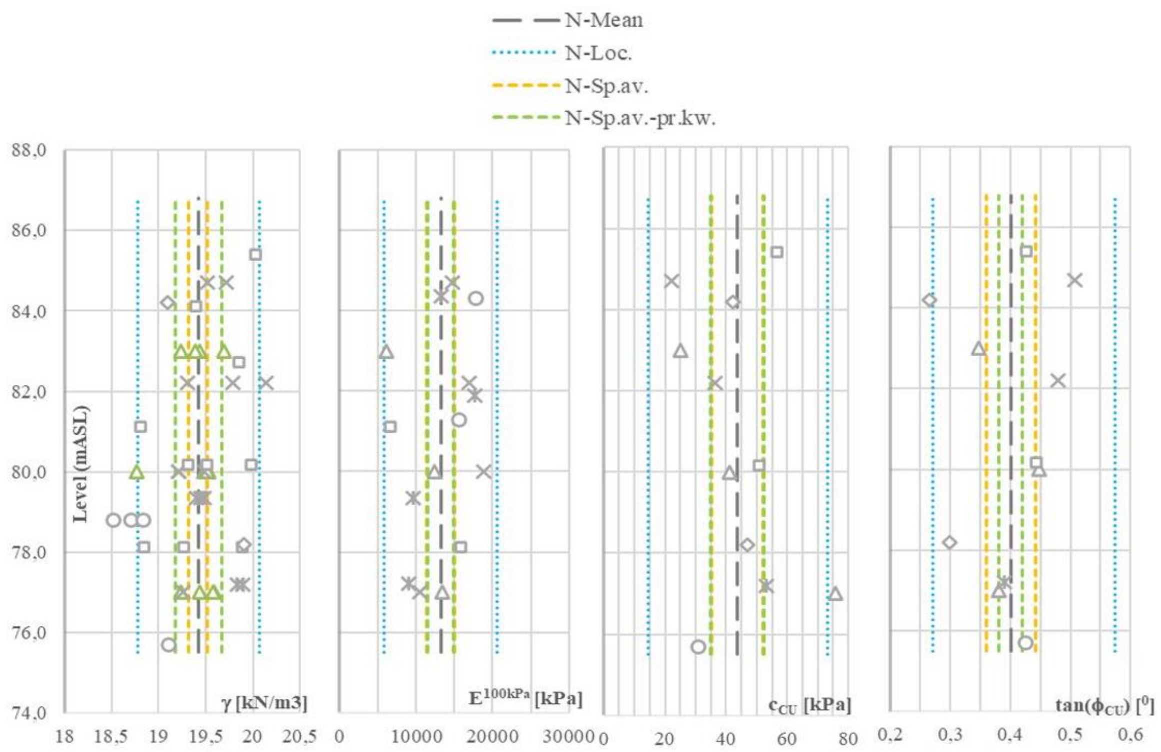
| Statistic values | Geotechnical property | | | |
|---|---|---|---|---|
| | $\gamma$ [kN/m³] | $E^{100kPa}$ [kPa] | $c_{CU}$ [kPa] | $\tan(\varphi_{CU})$ [-] |
| Sample mean, $m_x = \mu_x$ | 19.42 | 13 238 | 43.8 | 0.401 $\tan(21.8^0)$ |
| Sample Standard Deviation, $s_x$ | 0.378 | 4 081 | 15.5 | 0.074 |
| Coefficient of variation, ($V_x$) | 0.02 | 0.31 | 0.35 | 0.18 |
| Number (count), n | 42 | 15 | 11 | 11 |
| Statistical Coefficient for spatial average without 'prior knowledge', $k_n = t_{n-1}^{95\%} \cdot \sqrt{\frac{1}{n}}$ – Case A.3 | 0.26 | 0.45 | 0.55 | 0.55 |
| Superior characteristic value for spatial average with statistical uncertainty, without 'prior knowledge' – Case A.3 **(1)** | 19.52 | 15 094 | 52.2 | 0.441 $\tan(23.8^0)$ |
| Inferior characteristic value for spatial average with statistical uncertainty, without 'prior knowledge' – Case A.3 **(1)** | 19.32 | 11 382 | 35.3 | 0.360 $\tan(19.8^0)$ |

| Statistic values | Geotechnical property | | | |
|---|---|---|---|---|
| | γ [kN/m³] | E100kPa [kPa] | $c_{CU}$ [kPa] | tan($\varphi_{CU}$) [-] |
| 'Assumed' coefficient of variation, $V_{x.assumed}$ | 0.05 | 0.3 | 0.4 | 0.1 |
| Statistical Coefficient for spatial average with 'prior knowledge', $k_n = t_{\infty}^{95\%} \cdot \sqrt{\frac{1}{n}}$ - Case A.2 | 0.25 | 0.42 | 0.50 | 0.51 |
| Superior characteristic value for spatial average with statistical uncertainty, and with 'prior knowledge' - Case A.2 (1) | 19.67 | 14 925 | 52.4 | 0.421 tan(22.8$^0$) |
| Inferior characteristic value for spatial average with statistical uncertainty, and with 'prior knowledge' - Case A.2 (1) | 19.18 | 11 551 | 35.1 | 0.381 tan(20.9$^0$) |
| Correlation factor | - | - | -0.222 | |

1 This is taken as recommended in the Romanian Norm (NP 122:2010 Romanian norm on selecting the characteristic and design values of geotechnical parameters, 2011).

*Source: NP 122:2010 Romanian norm on selecting the characteristic and design values of geotechnical parameters, 2011.*

**Figure 71.** Variation of geotechnical properties with depth and the characteristic values resulted from the statistical description based on normal distribution assumptions.




*Source: Developed by the authors*

## Log-normal distribution

For the deformation modulus, the cohesion, and the tangent of the friction angle, due to the relatively large coefficient of variation, e.g. larger than 0.2, a lognormal distribution was considered to be more appropriate to be used to determine the characteristic values.

The characteristic values for 95% confidence level, for spatial average, with statistical uncertainty. are determined by Equation 72:

**Equation 72**. Determination of characteristic values for log-normal distribution.

$$x_k = e^{(\lambda \pm t_{n-1}^{95\%} \cdot \zeta \cdot \sqrt{\frac{1}{n}})} = e^{(\lambda \pm k_{n.unknownn} \cdot \zeta)}$$

Where the parameters $\lambda$ and $\zeta$ of the lognormal distribution are related to the mean ($\mu_x$) and the standard deviation ($\sigma_x$) of the variable $x$ using the following equations:

**Equation 73**. Determination of parameter λ

$$\lambda = ln\,\mu_x - \frac{1}{2}\zeta^2$$

**Equation 74**. Determination of parameter ζ

$$\zeta = \sqrt{ln(1 + \frac{\sigma_x}{\mu_x})}$$

The statistics for the same properties and the resulting characteristic values (in shadow rows) considering lognormal distribution are given in Table 57.

**Table 57.** Statistics for 'Bucharest Loam' layer considering lognormal distribution.

| **Statistic values** | **Geotechnical property** | | |
|---|---|---|---|
| | **$E^{100kPa}$ [kPa]** | **$c_{CU}$ [kPa]** | **tan($\varphi_{CU}$) [-]** |
| Sample mean, $m_x = \mu_x$ | 13 368 | 44.1 | 0.402 tan(21.9$^0$) |
| Sample Standard Deviation, $s_x$ | 4 896 | 16.7 | 0.081 |
| Mean for spatial average with statistical uncertainty | 12 613 | 41.6 | 0.395 tan(21.5$^0$) |
| Standard Deviation for spatial average with statistical uncertainty | 1 240 | 5.1 | 0.026 |
| Superior characteristic value for spatial average with statistical uncertainty | 14 750 | 50.4 | 0.439 tan(23.7$^0$) |
| Inferior characteristic value for spatial average with statistical uncertainty | 10 682 | 33.8 | 0.353 tan(19.5$^0$) |

Source: Developed by the authors

<u>Linear regression for dependent properties</u>

For estimating the statistics and characteristic values for the cohesion and the tangent of the friction angle, which are dependent properties, the liner regression model by least square method was used to statistically describe such dependence.

For determining the characteristic values for correlated variables, which are linearly dependent. $\hat{x}_1$ and $\hat{x}_2$ the following formula is used (Calle and van Duinen 2016):

**Equation 75**. Determination of characteristic value for correlated variables.

$$t_k = \hat{x}_1 + \hat{x}_2 \cdot s \pm t_{n-2}^{95\%} \cdot \sqrt{\sigma^2(\hat{x}_1) + s^2 \cdot \sigma^2(\hat{x}_2) + 2 \cdot \rho(\hat{x}_1.\hat{x}_2) \cdot s \cdot \sigma(\hat{x}_1) \cdot \sigma(\hat{x}_2) + {S_t}^2}$$

The first three terms under the square root represent the variance of the uncertainty of the mean of $t$. (spatial average) while the last term is the variance of the spatial distribution of $t$ (so. for estimating the local values).

The values for $\hat{x}_1$ and $\hat{x}_2$ are:

**Equation 76**. Determination of parameter $\hat{x}_2$.

$$\hat{x}_2 = \frac{\sum_{i=1}^{n} T_i \cdot (s_i - \bar{s})}{\sum_{i=1}^{n} (s_i - \bar{s})}$$

**Equation 77**. Determination of parameter $\hat{x}_1$.

$$\hat{x}_1 = \frac{\sum_{i=1}^{n} (T_i - \hat{x}_2 \cdot s_i)}{n}$$

Where:

**Equation 78**. Determination of parameter $\bar{s}$.

$$\bar{s} = \frac{\sum_{i=1}^{n} s_i}{n}$$

**Equation 79**. Determination of parameter ${S_t}^2$.

$${S_t}^2 = \frac{R^2}{(n-2)}$$

Estimations of the variance of the regression parameters $\hat{x}_1$ and $\hat{x}_2$ are calculated as follows:

**Equation 80**. Determination of the regression parameters $\hat{x}_1$.

$$\sigma^2(\hat{x}_1) = {S_t}^2 \cdot \left[\frac{1}{n}\left(1 + \frac{(\sum_{i=1}^{n} s_i)^2}{n \sum_{i=1}^{n} (s_i - \bar{s})^2}\right)\right]$$

**Equation 81**. Determination of the regression parameters $\hat{x}_2$.

$$\sigma^2(\hat{x}_2) = \frac{{S_t}^2}{\sum_{i=1}^{n} (s_i - \bar{s})^2}$$

The correlation coefficient is defined by the following formula:

**Equation 82**. Determination of the correlation coefficient.

$$\rho(x_1.x_2) = \frac{cov(x_1.x_2)}{\sigma(x_1) \cdot \sigma(x_2)}$$

Where $cov(x_1.x_2)$ is the covariance of two variables and is defined as follows:

**Equation 83**. Determination of the covariance of two variables.

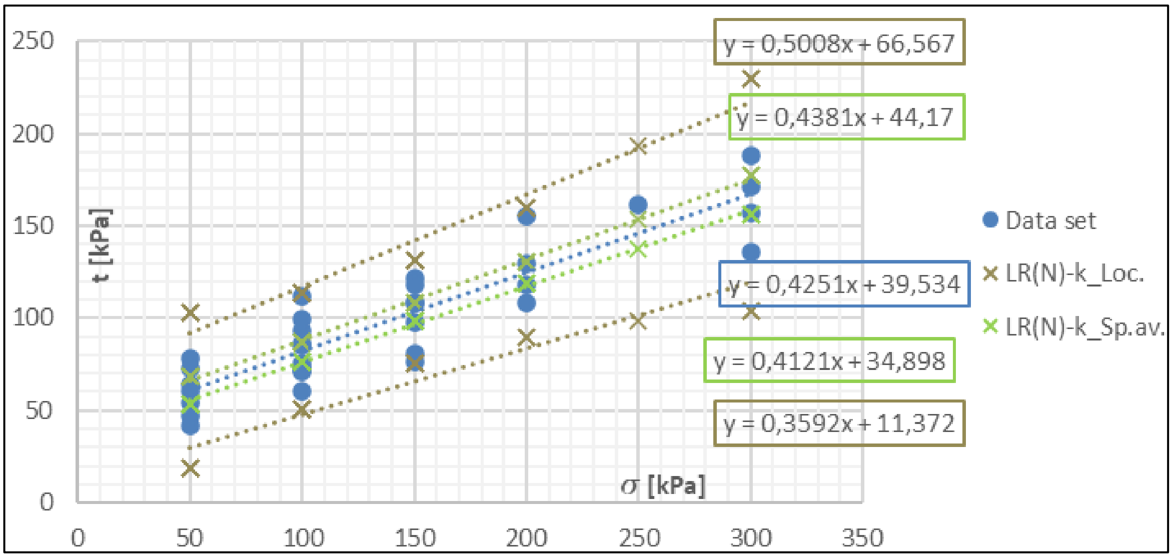
$$cov(x_1.x_2) = -\frac{\bar{s}}{\sum_{i=1}^{n}(s_i - \bar{s})^2} \cdot {S_t}^2$$

**Table 58.** Statistics for the cohesion and the tangent of the friction angle for 'Bucharest Loam' layer considering linear regression between them and results (in shadow rows).

| **Statistic values** | **Geotechnical property** | |
|---|---|---|
| | **$c_{CU}$ [kPa]** | **tan($\varphi_{CU}$) [-]** |
| Sample mean, $m_x = \mu_x$ | 39.5 | 0.425 tan(23.9$^0$) |
| Sample Standard Deviation, $s_x$ | 5.78 | 0.036 |
| Standard Deviation for spatial average with statistical uncertainty | 5.95 | 0.037 |
| Superior characteristic value for spatial average with statistical uncertainty | 44.2 | 0.438 tan(23.7$^0$) |
| Inferior characteristic value for spatial average with statistical uncertainty | 34.8 | 0.412 tan(22.4$^0$) |
| Correlation factor | -0.874 | |

*Source: Developed by the authors*

**Figure 72.** Linear regression model including characteristic values.




*Source: Developed by the authors.*

## V.3.4 Evaluation of results: determination of representative values

The characteristic values determined by different statistics presented resulted as follows:

**Table 59.** Values for the geotechnical properties considered.

| Statistical description | $\gamma$ [kN/m$^3$] | $E_{50}^{ref}$ [kPa] | tan(ϕ) [-] | $c_{ref}$ [kPa] |
|---|---|---|---|---|
| **Norm_assumed** | 19.67 | 11 551 | 0.381 tan(20.9$^0$) | 35.1 |
| **Norm_unknown** | 19.52 | 11 382 | 0.360 tan(19.8$^0$) | 35.3 |
| **Lognorm** | - | 10 682 | 0.353 tan(19.5$^0$) | 33.8 |
| **Linear regression** | - | - | 0.412 tan(22.4$^0$) | 34.8 |

*Source: Developed by the authors*

Also, other statistics performed by some TG-C1 members led to the following characteristic values for the undrained shear strength properties:

**Table 60.** Characteristic values determined by different TG-C1 members.

| TG-C1 member | Cohesion (kPa) | Friction angle (°) | Short description |
|---|---|---|---|
| 1 | 32.1 + 7.9z ≈ 71.6 | 0 | Case A EN 1997-1 (assuming zero friction angle and depth dependency) |
| 2 | 14.5 | 22.5 | Gaussian with shear tangent |
| | 18.5 | 21.5 | Lognormal with shear tangent |
| | 38.5 | 20.5 | Gaussian with shear properties |
| | 36.0 | 20.0 | Lognormal with shear properties |
| 3 | 30 | 19.8 | Characteristic value for friction angle with chosen nominal value for cohesion |

*Source: Developed by the authors*

In the case analysed, for the unit weight the 'prior knowledge' assumption considered does not lead to a lower standard deviation (or a set of characteristic values closer by the mean value). This is because the coefficient of variation of the data set is lower than the one given as a rough guidance in lack of a specific database.

For the deformation modulus and the cohesion, the difference between the situations 'without prior knowledge' (unknown) and 'with prior knowledge' (assumed) is not significant, while for the tangent of the internal friction angle the 'prior knowledge' assumption leads to a set of characteristic values closer by the mean value with about 4-5%.

The difference between the characteristic values from lognormal distribution and the inferior characteristic values from normal distribution, following the same assumption (spatial average with statistical uncertainty and no prior knowledge) is between 2% and 6%, while the mean values from

lognormal distribution for spatial average has lowered with about 5% for the deformation modulus and the cohesion and about 1% for the friction angle.
For the case analysed, the characteristic value of the cohesion is about 1% lower from linear regression analysis, while the characteristic value of the tangent of the internal friction angles is about 14% higher compared to the normal distribution. Thus, the shear strength ($\tau$) for $\sigma$ = 100 kPa is about 6% lower from normal distribution of the shear strength properties as independent variables than from linear regression.

## V.4 Summary and conclusions

This case study presented several statistical methods applied to determine characteristic values of to some geotechnical properties for the design of a retaining wall using FEM calculations. Following the different statistics for determining the characteristic values, the following representative values were chosen for the present case study:

— Characteristic value from normal distribution for the unit weight for unknown coefficient of variation – since the coefficient of variation resulted from the data set is lower than the one recommended as a rough guiding value for this parameter;

— Characteristic value from lognormal distribution for the deformation modulus, because of high coefficient of variation (which can also help avoid negative values for characteristic values for some data sets) - this approach leads to smaller mean value and characteristic values. so it is more conservative;

— Characteristic values from linear regression analysis for shear strength properties, which are correlated - it is more appropriate to account for the relationship between the variables and avoid incompatible set of values for the variables.

Furthermore, from the analyses performed and nominal values proposed, linear regression for the shear strength properties leads to pertinent set of data.

Alternatively, nominal values can be selected as representative values as those proposed by some TG C-1 members.

## V.5 References

Ene, A., 'Determinarea valorilor caracteristice ale parametrilor geotehnici prin diferite metode statistice pentru calculul peretelui de susținere a unei excavații din Bucureşti', *Lucrarile celei de-a XIV-a ConferintaNationala de Geotehnica si Fundatii Bucuresti*, Bucharest 2021 (in Romanian)

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