TU Graz
Graz University of Technology

Comparison of correlations from CPTu, SPT, DP

Masterarbeit zum Erwerb des
akademischen Titels Diplomingenieur der
Studienrichtung Bauingenieurwesen

Michael Sachsenhofer

Verfasst am Institut für
Bodenmechanik und Grundbau
der Technischen Universität Graz

Betreuer der Masterarbeit:
Ao.Univ.-Prof. Dipl.-Ing. Dr.techn. M.Sc. tit.Univ.-Prof. Schweiger Helmut,
Dipl.-Ing. Tschuchnigg, Franz;

Graz, August, 2012

Eidesstattliche Erklärung

Ich erkläre an Eides statt, dass ich die vorliegende Arbeit selbstständig verfasst, andere als die angegebenen Quellen/Hilfsmittel nicht benutzt, und die den benutzten Quellen wörtlich und inhaltlich entnommene Stellen als solche kenntlich gemacht habe.

Statutory Declaration

I declare that I have authored this thesis independently, that I have not used other than the declared sources / resources, and that I have explicitly marked all material which has been quoted either literally or by content from the used sources.

Graz, am ………………………… ....

(Sachsenhofer Michael)

Danksagung

Ein Studium sowie im speziellen diese wissenschaftliche Arbeit braucht die Unterstützung Vieler, um zum Abschluss zu kommen.

An erster Stelle möchte ich mich dabei bei meinen Eltern für die Unterstützung während meines gesamten Studiums bedanken.

Mein besonderer Dank gilt den Betreuern dieser Arbeit, Herrn DI Tschuchnigg und Herrn Prof. Schweiger. Sie hatten immer ein offenes Ohr für meine Anliegen.

Des Weiteren möchte ich mich bei jenen- im Speziellen bei Herrn DI. Hoffmann und Herrn Dr. Premstaller- bedanken, die mir Daten zur Verfügung gestellt haben und mir auch in fachlicher Hinsicht sehr geholfen haben.

Zuletzt möchte ich mich noch bei Samantha für das Korrekturlesen sowie bei Daniel, Phillip und Gerhard für die unterhaltsamen Stunden neben der Masterarbeit bedanken.

Abstract

This thesis covers three types of In- situ tests (CPT/u, SPT, DP) for geotechnical investigations with a special focus on correlations of geotechnical parameters. From the time when soil investigations began, many correlations have been developed since then and few have later been modified. Moreover, new correlations, partly for specific geological areas have emerged, which cause an abundance of correlations leading to confusion by the user. The correlations (predominantly) to strength and stiffness parameters are introduced and later compared and evaluated in projects from Italy, Austria and Germany. The correlations have been developed for sands (drained behaviour) and clays (undrained behaviour) with intermediate and organic soils additionally being discussed.

Furthermore, correlations that require results from both, In- situ and laboratory tests, are mentioned and applied on projects where laboratory results are available.

To support the evaluation and to compare the results, the software NOVOCPT and NOVOSPT has been used.

The problem of standardisation of equipment and procedures is still an obstacle to overcome and will thoroughly be examined. Standards and guidelines (International Reference Test Procedure 1989 from the ISSMFE) are compared and differences are highlighted. Proposed corrections on the N-value (for the SPT) and reduction due to skin friction (for DP) are shown and its impact on results will be compared.

Finally, suggestions on the applicability of certain correlations are presented.

Kurzfassung

In dieser Arbeit werden drei In- situ Tests (CPT/u, SPT, DP) für geotechnische Erkundungen behandelt. Der Fokus der Arbeit liegt dabei auf den Korrelationen zu geotechnischen Parametern. Seit dem Beginn der Bodenuntersuchungen wurden viele Korrelationen erstellt, von denen später einige modifiziert wurden. Zudem entstanden neue Korrelationen, teilweise für spezielle geologische Gebiete. Eine Fülle an Korrelationen war entstanden, die für den Benützer nur schwer überschaubar ist. In dieser Arbeit werden Korrelationen zu (vorwiegend) Festigkeits- und Steifigkeitsparametern vorgestellt und danach in Projekten aus Italien, Österreich und Deutschland, verglichen und beurteilt. Die meisten Korrelationen wurden für Sand (drainiertes Verhalten) oder Ton (undrainiertes Verhalten) erstellt. Hier jedoch sollen auch zwischenliegende Böden (Schluff, Organischer Boden etc.) behandelt werden.

Des Weiteren werden auch Korrelationen, die außer diesen In- situ Tests noch Ergebnisse von Laboruntersuchen benötigen, vorgestellt.

Um die Evaluierung der Resultate zu unterstützen wird zusätzlich eine Software (NOVOCPT und NOVO SPT) eingesetzt.

Die Standardisierung der eingesetzten Geräte sowie die Vorgehensweise ist noch immer ein großes Problem in der Anwendung und soll untersucht werden. Richtlinien („International Reference Test Procedure 1989 from the ISSMFE”) und Normen einiger Staaten werden verglichen und die Unterschiede hervorgehoben. Vorgeschlagene Korrekturen für den N- Wert (SPT) und die Abminderung aufgrund der Mantelreibung (DP) sowie dessen Auswirkung werden aufgezeigt.

Content

1. Introduction

Alongside the common practise of testing soils in the laboratory, In- situ testing is a very valuable tool to gain information on the subsoil and its physical properties.

Information on the subsoil may be obtained *In- situ* in three different ways (Clayton 1990):

- using geophysical techniques (seismic techniques),
- using in situ soil testing techniques,
- making measurements using field instrumentation;

This thesis will cover three In- situ soil testing techniques that are:

- Standard penetration test (SPT),
- Cone penetration test (CPT),
- Dynamic probing (DP);

These in- situ tests are more expensive compared to standard laboratory tests and less experience is available. However the factor of obtaining reliable and accurate laboratory results depends mainly on the process of sampling. If the quality of the sample is good, laboratory testing offers the best method to provide accurate information of the subsoil and its properties. For example, clays may readily be sampled.

In contrast, Clayton 1990 specifies soil types where In- situ testing has significant advantages over laboratory testing due to the sampling effects:

- very soft or sensitive clays,
- stoney soils,
- sands and gravels,
- weak, fissile or fractured rock;

Advantages and disadvantages of laboratory testing (versus In- situ testing):

+ full control of the test conditions
+ simple repeatability
+ low costs
- sample disturbance
- unrealistic boundary conditions

A combination of both, In- situ and laboratory tests may provide a complete description of the subsoil and will be executed in large extend at bigger projects, where an accurate soil parameter determination is required.

Applicability of correlations from In- situ tests:

Within one In-situ test, differences occur in the applicability of a correlation to one specific parameter. Furthermore, some parameters can be determined with high accuracy, whereas others are difficult to obtain (compare Figure 1).

For example, the undrained shear strength may be more accurately determined from a shear vane test, rather than from a CPT or a SPT. Thus, the use of multiple In-situ tests for one specific site may be useful (although higher costs).

		Soil Parameters												
Group	Device	Soil type	Profile	u	$^*\phi'$	s_u	I_D	m_v	c_v	k	G_o	σ_h	OCR	σ-ε
Penetrometers	Dynamic	C	B	–	C	C	C	–	–	–	C	–	C	–
	Mechanical	B	A/B	–	C	C	B	C	–	–	C	C	C	–
	Electric (CPT)	B	A	–	C	B	A/B	C	–	–	B	B/C	B	–
	Piezocone (CPTU)	A	A	A	B	B	A/B	B	A/B	B	B	B/C	B	C
	Seismic (SCPT/SCPTU)	A	A	A	B	A/B	A/B	B	A/B	B	A	B	B	B
	Flat dilatometer (DMT)	B	A	C	B	B	C	B	–	–	B	B	B	C
	Standard penetration test (SPT)	A	B	–	C	C	B	–	–	–	C	–	C	–
	Resistivity probe	B	B	–	B	C	A	C	–	–	–	–	–	–
Pressuremeters	Pre-bored (PBP)	B	B	–	C	B	C	B	C	–	B	C	C	C
	Self boring (SBP)	B	B	A[1]	B	B	B	B	A[1]	B	A[2]	A/B	B	A/B[2]
	Full displacement (FDP)	B	B	–	C	B	C	C	C	–	A[2]	C	C	C
Others	Vane	B	C	–	–	A	–	–	–	–	–	–	B/C	B
	Plate load	C	–	–	C	B	B	B	C	C	A	C	B	B
	Screw plate	C	C	–	C	B	B	B	C	C	A	C	B	–
	Borehole permeability	C	–	A	–	–	–	–	B	A	–	–	–	–
	Hydraulic fracture	–	–	B	–	–	–	–	C	C	–	B	–	–
	Crosshole/downhole/ surface seismic	C	C	–	–	–	–	–	–	–	A	–	B	–

Figure 1: Applicability and usefulness of in-situ tests, A= high, B= moderate, C= low, -= none; (Clayton 1990);

Most correlations have been developed empirically, whereas just a few have a theoretical background. The empirical correlations have been determined for specific local sites (or sometimes just one site) where complete information on geotechnical parameters is available. As the soil (and the soil composition) by nature varies around the globe, it is difficult to validate such a correlation for another subsoil.

All these correlations require engineering judgement. Lots of authors are very carefully on the application of such correlations- and as a result, awareness of these results should be ensured to the user. Some quotations:

“These correlations are approximate and their use requires the exercise of engineering judgment regarding the inevitable uncertainties in estimated property values”; quoted from McGregor et al.1998.

“In addition, engineers usually rely on local experience to judge the applicability and validity of such correlations “; cited from ASTM D 3441 – 05 (Standard Test Method for Mechanical Cone Penetration Tests of Soil), which may be valid for all in- situ tests.

An important note on the evaluation of correlation from In- situ tests:

For most projects only one In- situ test was picked for evaluation purposes. When comparing correlations to parameters that are suggested in the geotechnical report (for the entire project), it has to be kept in mind that those parameters represent the whole area, whereas results of one test at a specific location may slightly vary.

Moreover, for the majority of projects, laboratory tests and/or other In- situ tests (Vane test, Dilatometer test etc.) have been performed and may influence the determination of parameters. Additional information from neighbouring building sites and knowledge of the geology in that certain area may also be incorporated for the evaluation in the geotechnical report. Furthermore, suggestions of geotechnical parameters provide the base for further calculations, which tend to be more conservative.

2. Cone Penetration Test

2.1 Introduction

In the Cone penetration test (CPT), a cone on the end of a series of rods is pushed into the ground at a constant rate and continuous or intermittent measurements are made of cone resistance and the sleeve friction. The CPTu additionally measures the porewater pressure (Mayne et al. 2001).

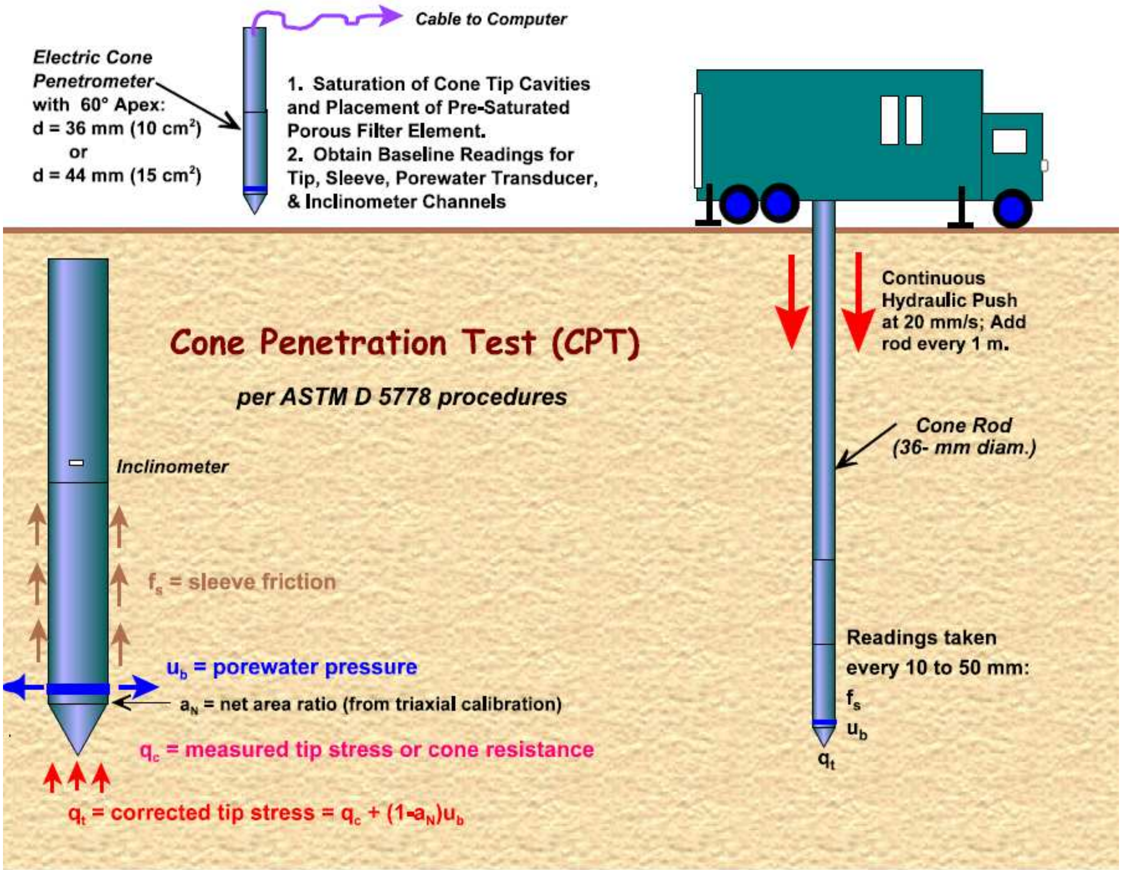


Figure 2: Procedures and components for the cone penetration test (Mayne et al. 2001)

The cone resistance (q_c) is the result of the acting force on the cone (Q_c) divided by the projected area of the cone (A_c). The sleeve friction (f_s) is the result of the total force acting on the friction sleeve (F_s) divided by the surface area of the friction sleeve (A_s) (compare Robertson 2010).

CPT- results and measurements are usually given and plotted in MPa. A typical CPTu reading is shown in Figure 3.

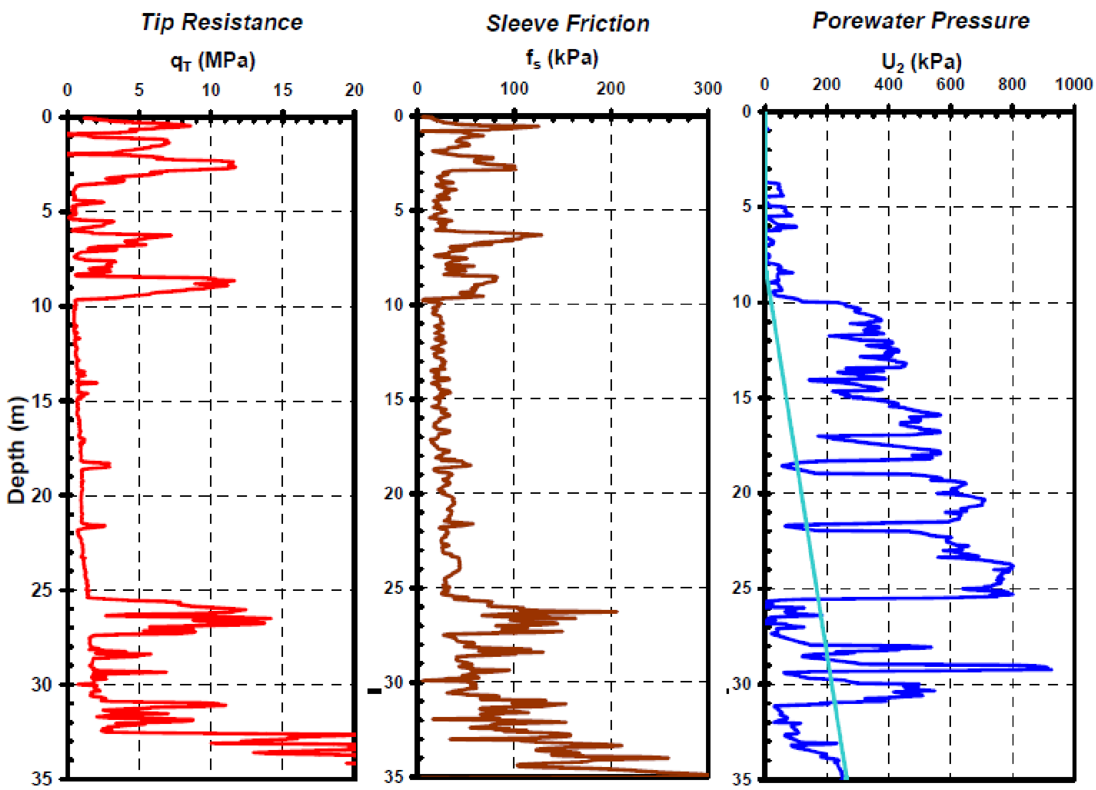


Figure 3: typical CPTu reading (Mayne et al. 2001)

2.1.1 Additional measurements

2.1.1.1 Porewater pressure

In the CPTu test, the pore water pressure is measured additionally to the sleeve friction and cone resistance. The porewater pressure with its additional correlations may provide more reliable results. The EN ISO 22476- 1 describes the difference between the CPT and the CPTU in *Types of CPT*, TE1 is performed without pore water pressure measurement, whereas for the TE2 pore water pressure has to be recorded.

2.1.1.2 Shear wave velocity

In recent years, it became increasingly popular to additionally measure the shear wave velocity. For example, the small strain stiffness modulus is a parameter often evaluated using the shear wave velocity.

The addition of a seismic sensor (usually a geophone, an accelerometer or seismometer) inside the barrel of a standard electric CPT is termed a Seismic Cone Penetrometer Test (SCPT). Such a sensor allows the measurement of the arrival of vertically propagating seismic body waves, generated from a source on the ground surface (Butcher et al. 2005).

In practise, cross-hole and down-hole methods have become the standard techniques for dynamic testing to determine the in-situ shear wave velocity (Robertson et al. 1986).

During a pause in cone penetration, a shear wave is created at the ground surface that will propagate into the ground and measurements of the time taken for the seismic wave to propagate to the seismometer in the cone are made. By repeating this measurement at another depth, one can determine (from the signal traces) the interval time and so calculate the average shear wave velocity over the depth interval between the seismometers. A repetition of this procedure with cone advancement yields a vertical profile of vertically propagating shear wave velocity (Butcher et al 2005).

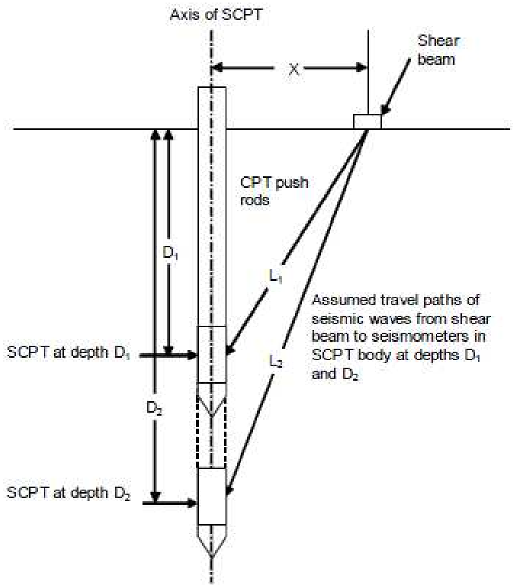


Figure 4: Schematic diagram of a seismic cone test with required dimensions, D1, D2, and X (Butcher et al. 2005).

2.1.1.3 Dissipation test

During a pause in penetration, any excess pore pressure generated around the cone will start to dissipate. The rate of dissipation depends on the coefficient of consolidation, which in turn, depends on the compressibility and permeability of the soil. The rate of dissipation also depends on the diameter of the probe. A dissipation test can be performed at any required depth by stopping the penetration and measuring the decay of pore pressure with time. It is common to record the time to reach 50% dissipation (t_{50}). If the equilibrium pore pressure is required, the dissipation test should continue until no further dissipation is observed. This can occur rapidly in sands, but may take many hours in clays. Dissipation rate increases as probe size decreases (compare Robertson 2010).

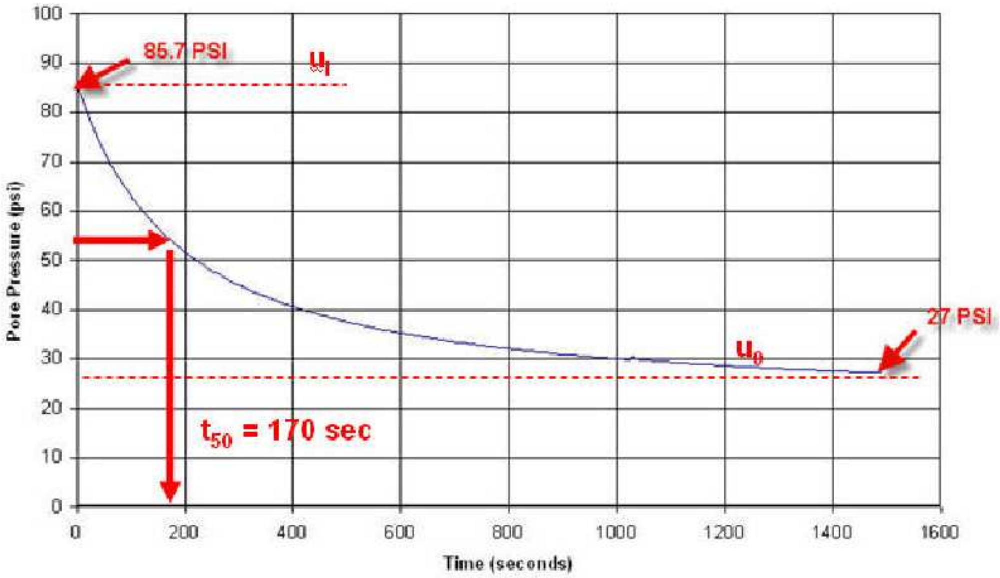


Figure 5: Example dissipation test to determine t_{50} (Robertson 2010)

2.1.2 Application of CPTu

The CPT/u has three main applications in the site investigation process (compare Lunne et al. 1997):

1. To determine subsurface stratigraphy and identify materials present,
2. To estimate Geotechnical parameters,
3. To provide results for direct geotechnical design;

The EN ISO 22476-1 introduces four application classes for CPTu. They differ in the type and composition of the soil (interbedding, compaction of layers etc.), where the CPT is conducted (EN ISO 22476- 1, page: 21). This standard also determines the "Type of CPT/U" to be performed and to what parameter can be derived from CPT results. It also provides tolerances that must be adhered to.

Mayne et al. 2001, summarizes the advantages and disadvantages of a CPT:

Advantages:

- Fast and continuous profiling,
- Economical and productive,
- Strong theoretical basis in interpretation,
- Particularly suitable for soft soils;

Disadvantages:

- High capital investment,
- Requires skilled operator to run,
- Electronic drift, noise, and calibration,

- No soil samples are obtained,
- Unsuitable for gravel or boulder deposits;*

*Note: Except where special rigs are provided and/or additional drilling support is available.

Mayne 2007 states that one clear advantage of the CPT is its ability to provide three independent and simultaneous measurements (contrary to the SPT where just one value is obtained). Additional sensors are available to produce up to four or five direct readings, with a certain depth to ascertain a more realistic evaluation of soil behaviour.

2.2 General variations of CPT

2.2.1 Standards

An overview of wide and commonly used CPT Equipment (and its variations) all over the world will be provided. The focus is on the most used and cited standards such as the ASTM D5778 and D3441 and the BS 1377 Part 9, 1990 as well as the DIN 4094- 1 and the ÖN EN ISO- 22476- 1 (Draft- 05) for Austria and Germany (Central Europe). Furthermore, the Swedish standard (SGF Report 1:93 E) that is recommended for Cone Penetration Tests by Lunne et al. 1997- CPT in Geotechnical Practice- will be introduced. A very important document concerning the standardisation of the CPT is the „International reference test procedure for cone penetration test (CPT) that was published in 1989 by the ISSMFE Technical Committee on Penetration Testing of Soils- TC 16, which will often be referred to.

2.2.2 Equipment and procedures

There are some noticeable differences in the CPT- equipment and its procedures which will be described in this section. The CPT is also far from standardized. Firstly, general variations on the equipment that are in use around the world will be introduced. Secondly the standards that are mentioned above will be compared. Furthermore, for comparing the standards some representative dimensions are picked on CPT- equipment and highlights the variations in the procedure.

2.2.3 CPT- Systems

Existing CPT systems can be divided into three main groups:

Comparison of correlations from CPTu, SPT, DP;

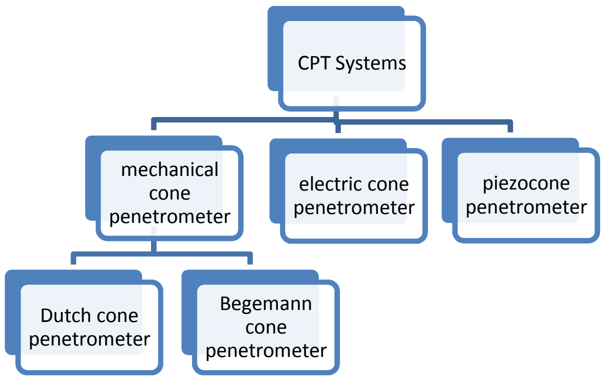


Figure 6: CPT Systems

Both electrical and mechanical penetrometer by means of measuring cone resistance and side friction are currently used. The shape of the cone differs considerably according to the method in use (compare Clayton et al. 1995).

Mechanical penetrometers operate incrementally, using a telescoping penetrometer tip, resulting in no movement of the push rods during the measurement of the resistance components. Design constraints for mechanical penetrometers preclude a complete separation of the end-bearing and side-friction components (ASTM D3441 – 05).

Because of their low costs, simplicity and robustness, mechanical cone penetrometers are still widely used. In rather homogeneous soils, without sharp variations in cone resistance, mechanical cone data can be adequate, provided by the equipment is properly maintained and the operator has the required experience. Nevertheless the quality of the data remains somewhat operator- dependant. In soft soils, the accuracy of the results can sometimes be inadequate for a qualitative analysis of soil properties. In highly stratified materials even a satisfactory qualitative interpretation may be impossible (Clayton et al. 1995).

Electric cones are more expensive, both in terms of cone manufacture and data logging and recording. They have the advantages, however, of being simpler to use, of measuring forces close to their point of application and of providing almost continuous data with respect to soil depth.

Additionally, some more measurements like the cone inclination (drifting out of vertical) or the pore pressure in the piezocone may be possible of being recorded with the electric cone (compare Clayton et al. 1995).

According to Meigh 1987, electric cones have the following advantages:

- improved accuracy and repeatability of results, particularly in weak soils,
- better delineation of thin strata (because readings can be taken more frequently),
- faster over-all speed of operation,
- the possibility of extending the range of sensors in or above the tip (see above),
- and more manageable data handling;

As mentioned, the USA published two standards, one dealing with the “Standard Test Method <u>Electronic</u> Friction Cone and <u>Piezocone</u> Penetration Testing of Soils” in the ASTM D5778 document, and another, describing the “Standard Test Method for Mechanical Cone Penetration Tests of Soil” (compare ASTM D3441). This standard covers two specific types of mechanical penetrometer, the Dutch mantle cone and the Begemann- friction- cone.

Also in Europe a standard (EN ISO 22476 Part: 12) “Mechanical cone penetration test (CPTM)” was published in 2009 and updated in 2012, however receives less consideration in literature. In Germany and in Sweden only one standard (DIN 4094- 1 and SGF Report 1:93 E) covers all systems provided.

2.2.4 Location for filter to measure the pore water pressure

Another big uncertainty is the location of measurement devices for the pore pressure that are:

- on the cone (u_1),
- behind the cone (u_2),
- behind the sleeve friction (u_3),
- also, combined measurements- dual element or triple element piezocones (additional correlations can be derived);

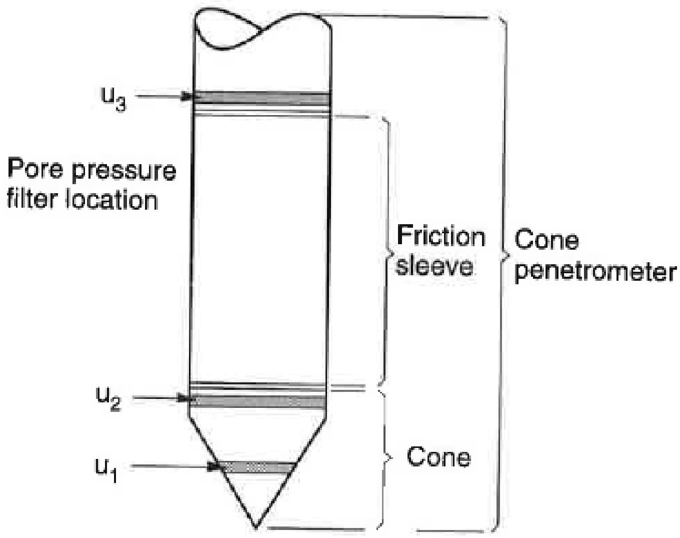


Figure 7: Possible locations for pore pressure filter (Lunne et al. 1997)

Depending on the types of soils being tested, the porous filter is usually located either at the apex or midface (termed- in accordance with the American Standard ASTM D 5778- 07- Type 1) or at the shoulder (Type 2) just behind the cone tip, else positioned behind the sleeve (Type 3) (compare: Mayne 2007).

The IRTP, 1989 suggests the position of the measurements behind the cone (u_2).

2.2.5 Variations in base area

The standard equipment consists of a cone with a 10 cm² base area. For various applications different cones with a larger or smaller base area have been developed. Other available dimensions are a cone with a 15 cm² and a 5 cm² base area. The 15 cm² cone penetrometers are widely used in China (compare: Powell 2010) and in offshore seabed tests (when additional sensors are incorporated). The 5 cm² miniature cone penetrometer was also introduced for research and consulting project (often used in laboratory model tests in order to increase the sample to cone diameter ratio and to reduce boundary effects) (compare: Lunne et al. 1997).

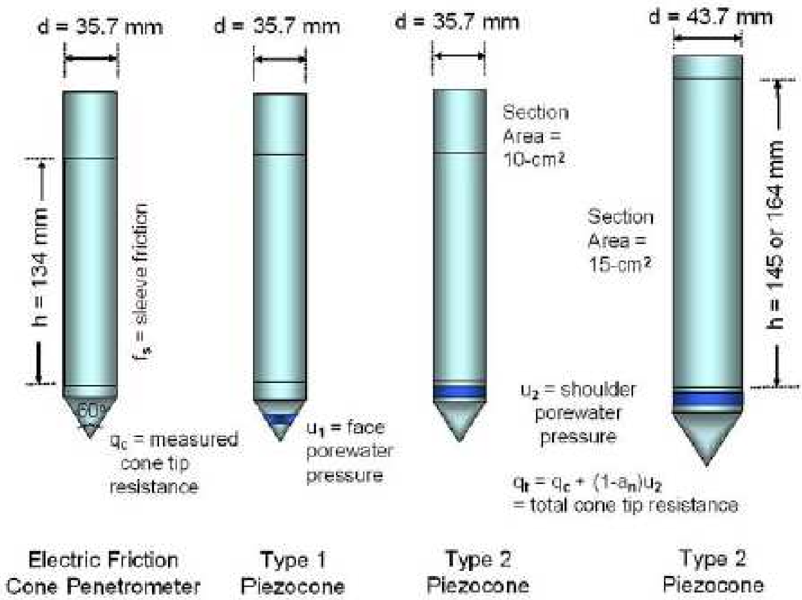


Figure 8: Variations in cone base area and location of porewater measurements (Mayne 2007)

For the user it is important to know if the correlations have to be corrected when using a non-standard diameter. Lunne et al. 1997 suggests that in practice cone penetrometers ranging in cross sections from 5 cm² to 15 cm² give very similar *corrected* cone resistance values in most materials. For dimensions outside this range it is recommended that corrections should be considered, preferably based on site specific correlations.

Note: When calculating the area ratio (used for calculation the *corrected cone resistance*) of the cone ($=A_n/A_c$) of the corrected cone resistance the user should be aware that α is ~0,676, when using a 15 cm² cone;

2.2.6 Design of the cone penetrometer

Due to various manufacturers there are three different kinds of arrangements to measure cone resistance and sleeve friction:

(1) Cone resistance and sleeve friction are measured by two independent load cells both in <u>compression</u> (see Figure 9a).
(2) Same as (a) but measured in <u>tension</u>.
(3) In the third type the sleeve friction load cell (in compression) records the summation of the loads from both- the cone resistance and sleeve friction. The sleeve friction is obtained from the difference in load between the friction and cone resistance load cells. This cone is often referred as the subtraction cone (compare: Lunne et al 1997).
Several users mentioned that this design show the best overall robustness of the penetrometer.

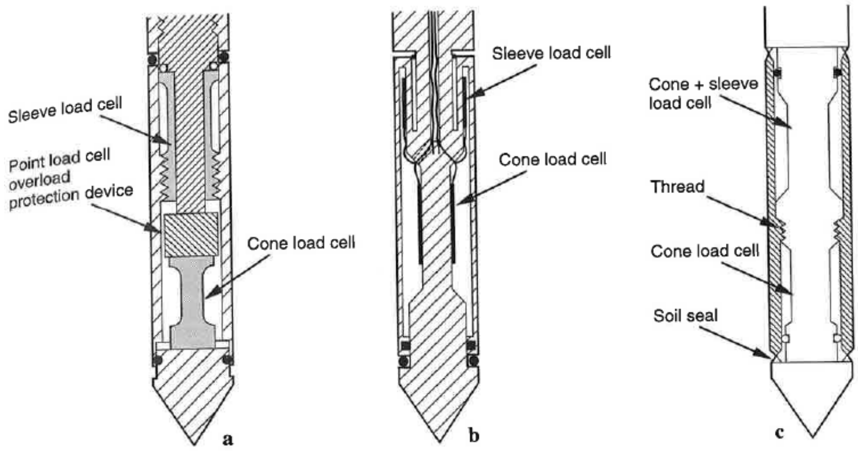


Figure 9: (a) Cone resistance and sleeve friction load cells in compression. (b) Cone resistance load cell in compression and sleeve friction load cell in tension. (c) Subtraction type cone penetrometer (Lunne et al 1997);

2.2.7 Other effects

There are some further factors that have an impact on the result of a CPT reading:

- Temperature effects (influences mechanical devices containing load cells or sensors)
- Inclination
- Calibration and resolution errors
- Wear of the cone

2.3 Hydraulic Pushing system

Figure 10: (a) Truck-Mounted and (b) Track-Mounted Cone Rigs (Mayne et al 2001)

The pushing equipment consists of push rods, a thrust mechanism and a reaction system. The rigs, used for pushing the penetrometer normally consist of hydraulic jacking and reaction systems. They are specially built for this purpose. The thrust capacity needed for cone testing generally varies between 10 and 20 tons (100 and 200kN). Screw anchors will also be used if extra reaction is needed. The equipment on land is similar to that for shallow water CPT work, but needs special modifications on the standard techniques for deepwater CPT work (compare: Lunne et al 1997).

Typical depths of penetration by CPT rigs depends upon the site-specific geologic conditions but most commercial systems are setup for up to achieve 30 m. In some special cases, onshore CPTs have reached 100 m using direct-push technology from the ground surface. Downhole CPTs can also be conducted step-wise in deep boreholes with depths up to 300 m or more (compare: Mayne 2007).

2.4 International Reference test procedure for CPT (1989)

The Reference Test Equipment consists of a 60° cone, with 10 cm² base area and a 150 cm² friction sleeve located above the cone. The position of the filter for measurement of pore pressure is not standardized but the IRTP suggests the location behind the cone (u_2). Cone resistance (q_c) and sleeve friction (f_s) are usually derived from electrical strain gauge load cells. The rate of penetration should be 2cm/s ±0,5 cm/s.

2.4.1 CPT in different countries

Powell 2010 presents existing CPT equipment and procedures in different countries. The author refers and introduces some papers that describe the domestic situation and developments in CPT technology. The papers are mainly for land-based investigations but some deal with shallow and deep water offshore applications. Powell 2010 summarizes, that there is a lack of papers dealing with equipment and procedures and outlines that techniques moving between countries and cross correlations to other national practices.

A large number of papers about certain areas have been published that describe the subsoil and locally derived correlations. These papers have been an attempt to get a better knowledge about the local ground, by developing new CPT correlations.

A good overview over the situation in northern European countries (including Austria) is given in Long 2010.

2.4.2 Comparison of standards

In the following representative dimensions and its tolerances are evaluated and listed in the table below. It has to be mentioned that all dimensions are derived from a CPT device with a cross section of 10cm. As, previously mentioned in *Variations of the base area* there are also larger or small er diameters that are used. The CPT with a 15cm² base area is only introduced in the ASTM D 5778 (with dimensions), and in the German standard DIN 4094-1. In all other standards the larger CPT device is not mentioned.

The dimensions (of a CPT with a base area of 10cm²) can be seen and compared in Figure 11.

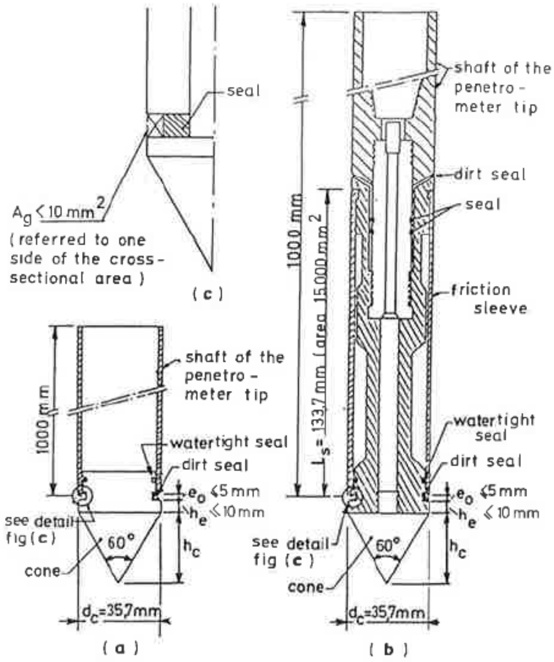


Figure 11: Dimension of Cone penetrometer in accordance to the IRTP for CPT, 1989

Table 1: Comparison of standards

Document	dc [mm]	hc [mm]	he [mm]	Sleeve area [mm²]	Penetration speed [mm/sec]	Intervals of reading [mm]
Piezocone 20±2 Without u 20±5	34,8-36,0	7- 10	24-31,2	15000	20±5	200, recommended continuous
PrEN ISO 22476- 1: 2005	35,3-36,0	As IRTP	As IRTP	As IRTP	As IRPT	For class 1 and 2: 20mm for class 3 and 4: 50mm
ASTM D 5778- 07 (electric and piezocone)	35,3-36,0	3-7	As IRTP	As IRTP	As IRTP	Recommended 100 / minimum more than 50
ASTM D 3441- 05 (mechanical)	35,7±0,4	5	30	As IRTP	10-20	-----
BS 1377 Part 9, 1990	35,3-36,0	2,0-5,0	24,0-31,2	As IRTP	As IRTP	≥200
SGF Report 1:93 E	CPT1: as IRTP	As IRTP	As IRTP	As IRTP		Pore pressure ≤ 20mm, penetration recom.: 10mm; without -u every 50mm
	CPT2&3: 35,4- 36					
DIN 4094-1: 2002-06	35,7±0,3	4,4	33	15000	As IRTP	20

2.4.3 Note on penetration rate

Remarkable is the tolerance (20±*5* mm/ sec) accepted for the penetration rate in the IRTP and adopted by almost all other standards. It is questionable if the cone resistance is the same if the rate of penetration is 15 mm/sec or 25mm/sec. A limited number of studies have been published to explain the effects of various penetration rates.

Generally it is assumed that, for clean sand, drained behaviour prevails during cone penetration at the standard rate of 20 mm/sec. In contrast, penetration in clays takes place under fully undrained conditions at the same penetration rate. In some intermediate soils, cone penetration would take place under partially drained conditions at the standard penetration rate. When the drainage condition changes from undrained to partially drained, the soil ahead of the cone starts to consolidate. Thus q_c increases due to the increase in soil strength around the cone tip (compare Kim et al. 2006).

Lunne et al. 1997 shows a summary of related data to the influence of rate of penetration on cone behaviour and the main conclusions are summarized. (Importance of the soil type: coarse grained-generally drained conditions- or fine grained soils- generally undrained conditions.)

Most testing in sands indicates that rates less than the penetration rate of 20mm/sec has little effect on the cone resistance. For faster rates of penetration an increase in cone resistance may occur as a result of dilatancy and higher negative pore water pressure (Lunne et al. 1997).

For clays, most of the CPT rate effect studies have shown an increase in q_c with an increasing rate of penetration. However, a study done by (Bemben and Myers, 1974) has shown a minimum cone resistance at a rate of 2mm/s. Bemben and Myers, 1974 argued that up to about 0,5mm/s drained conditions apply and that above about 50 mm/s undrained conditions apply (compare Lunne et al. 1997). The shape of the curve is shown in Figure 12.

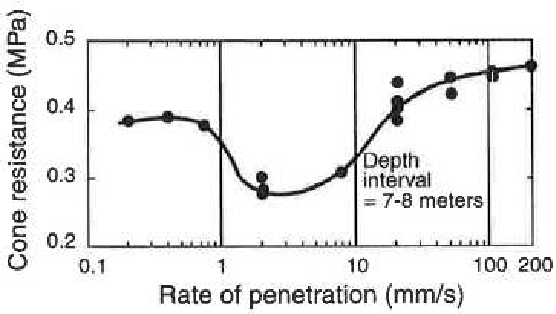


Figure 12: Influence of penetration rate on cone resistance, study by Bemben and Myres, 1974 (Lunne et al. 1997)

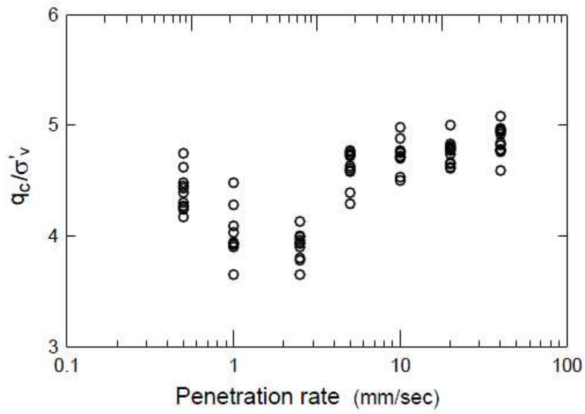


Figure 13: Study by Roy et al 1982 (Lunne et al. 1997)

For very slow rates of penetration q_c is predominantly of a drained nature. As the rate of penetration increases, q_c decreases due to the decrease in effective stress and reduction in strength. As the penetration rate increases, the viscous forces offset the strength reduction and the curve will pass through a minimum. Then viscous forces will tend to dominate the process and q_c will increase (Lunne et al 1997).

The study, done by Roy et al. 1982 showed similar results to those obtained by Bemben and Myers 1974.

According to Kim et al. 2006, in the study conducted by Roy et al 1982, the pore pressures were not observed to drop with decreasing penetration rate. Kim et al. 2006 claimed that the drainage conditions probably were undrained for all rates. Thus, the change of q_c due to the change of penetration rate in the test is probably not explained by transition from drained to undrained conditions.

Kim et al. 2006 concluded that almost all previous studies tried to investigate the rate effects in the CPT by data from either the field or calibration chambers. But the correlations between penetration rates and cone resistance were developed without full characterization of soil properties or enough consideration of drainage conditions. Kim et al. 2006, proposed a normalized penetration rate:

$$V = \frac{v * d}{c_v}$$

With: v = cone velocity;
d = cone diameter;
c_v = coefficient of consolidation;

This form of dimensionless velocity accounts for differences in velocity, soil compressibility, probe size, material type, void ratio, and stress conditions (Jaeger et al. 2010).

Kim et al. 2006 argued that the true transition point between undrained and partially drained penetration should be decided by observations of pore pressure and not q_t. According to Figure 15, this transition occurs for V≈ 10. The range between the minimum q_t in Figure 16 and V≈ 10 is an "offset range" within which q_t would tend to drop because it approaches undrained conditions but would tend to increase because loading rate effects (If undrained conditions are obtained at high penetration rates in low permeability clayey soils, soil shearing rate effects should be considered). Loading rate effects- which may be attributed to soil "viscosity"-, were studied by several researchers. It is known that the shear strength of clay increases, because loading rate effects start taking place.

From a practical standpoint, if the goal is to determine the value of V at which penetration resistance stops dropping, then the V≈ 4 read from the q_t plot may be of great interest.

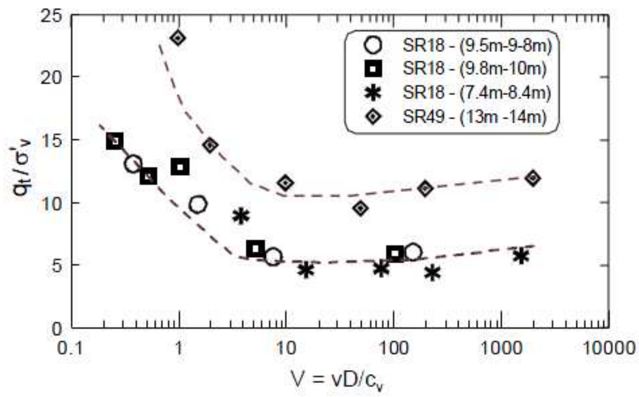


Figure 14: Normalized cone resistance vs. normalized penetration rate (Kim et al. 2006)

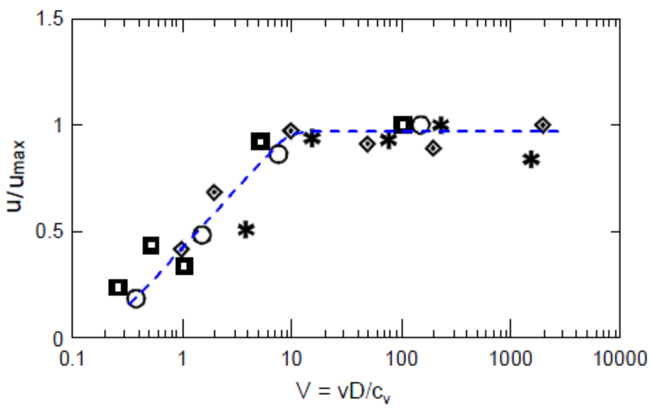


Figure 15: Normalized excess pore pressure vs. normalized penetration rate (Kim et al. 2006)

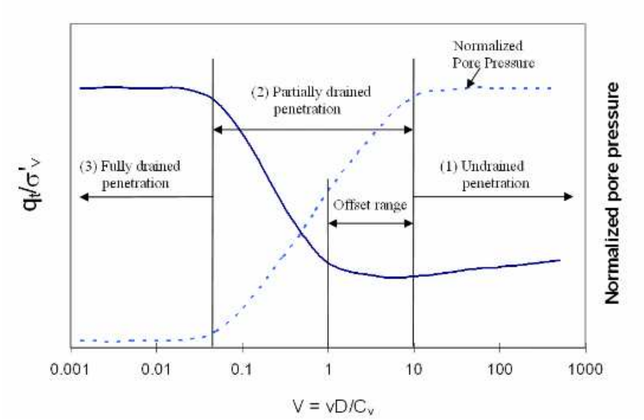


Figure 16: Effect of penetration rate on normalized cone resistance and pore pressure (Kim et al. 2006)

2.5 Correlations of CPTu results

Concerning the use of published correlations a rough soil classification (clay or sand) is useful to know beforehand, because just a limited amount of correlations are applicable to both soils - clay and sand.

In order to provide the reader an understanding in the interpretation of CPTu results, the following should be kept in mind (compare Jacobs 1996):

- Sand: high cone resistance, low friction ration, low pore pressure - quick dissipation of water (high permeability), Typical values: q_c = 8 - 12 MPa R_f = 1%
- Clay: low end resistance, high friction ratio, high pore pressure - slow dissipation of water (low permeability), Typical values: q_c = 0.7 MPa R_f = 3-3½ %.

Almost all correlations have been developed empirically and done in clean sands or clay. Some equations use coefficients that interpolate between sand and clay (silt), but most equations are only applicable to clay or sand. More information can be found in the chapter *Applicability of correlations*.

The use of probabilistic numerical and analytical models has a comparatively low proportion of papers in CPT interpretation and correlation. This may result from difficulties in accurately modelling the penetration process using numerical methods with realistic soil models, as many constitutive models do not account for soil structure and aging effects, high stress compressibility and particle crushing, stress dependent dilation, as well as stress and strain dependence on soil stiffness (Schneider 2010).

2.5.1 Applicability of correlations

Not every geotechnical parameter can be derived with CPT readings. CPT readings show significant limitations in correlating geotechnical parameters. To give the reader an overview about the applicability of the correlation to geotechnical parameters by CPT readings two figures are presented below. Figure 17 was published by Robertson et al. 2010, whereas Figure 18 is presented by Lunne et al in 1997. Both figures distinguish between Sand and clay, which is common practice. Additionally, this thesis should cover other soil types regarding CPT correlations (peat/ organic soils, silt), but still the main focus remains on clays and sands.

As mentioned in the Introduction the local experience of the subsoil is very important. Every area has its specifications and results of CPT correlations can be deceivable.

Soil Type	D_r	Ψ	K_o	OCR	S_t	S_u	ϕ'	E,G*	M	G_0*	k	c_h
Sand	2-3	2-3	5	5			2-3	2-3	2-3	2-3	3	3-4
Clay			2	1	2	1-2	4	2-4	2-3	2-4	2-3	2-3

1=high, 2=high to moderate, 3=moderate, 4=moderate to low, 5=low reliability, Blank=no applicability, * improved with SCPT

Figure 17: Applicability of CPTu for derived soil parameters (Robertson et al, 2010)

	Initial state parameter					Strength parameters		Deformation characteristics			Flow charact.	
Soil type	γD_r	ψ	K_0	OCR	S_t	s_u	$\phi'a$	E, G	M	G_o	k	c_h
Clay	3–4		4–5	2–3	2–3	1–2	3–4	4–5	4–5	4–5	2–4	2–3
Sand	2–3	2	4–5	4–5			2	2–4	2–4	2–3		

Applicability rating: 1 High reliability; 2 High to moderate reliability; 3 Moderate reliability; 4 Moderate to low reliability; 5 Low reliability.

Figure 18: Applicability of CPTu for derived soil parameters (Lunne et al 1997)

Noticeable and worth mentioning is the case, that within the two figures, significant differences appear in assessing the applicability of CPT. An argument could be that in the meantime (compare difference in year of publication: 1997 and 2010) new correlations that are better applicable have surfaced.

Interpretation methods valid for sands or clays may not be applicable for silts since penetration in this type of material is partially drained. Bugno and McNeilan (1984) suggested that undrained response will occur for the standard penetration rate of 20mm/s if the permeability of the soil is less than 10^-7 to 10^-6 cm/s. Soils having permeability between 10^-6 and 10^-3 cm/s, which would include most silts, are believe to behave in a partially drained manner (Lunne et al. 1997).

2.5.2 Other Materials

2.5.2.1 Intermediate soils (clayey sands to silts)

Correlations with materials besides sand and clay are quite difficult to handle and to assess.

The definition of silt varies in different countries. According to ASTM D 2487 – 06, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), silt is defined as soil passing the 75 µm sieve with a Plasticity Index I_P, less than 4 % or if the plot of I_P versus liquid limit falls below the “A” line. In addition, the ASTM D 2487 – 06 does not describe how to distinguish between clayed silt and silty clay since the sub-classification is given in terms of percentages of soil mass retained on the 75 µm sieve, and not on the actual clay content, wherefore "clayed silt" does not exist according to the ASTM D 2487 – 06. According to the ISO 14688-1 (2002), silt is defined as soil with a particle size between 63 µm and 2 µm. The classification of sand is based on the soil fraction predominating in terms of mass (Poulsen et al. 2011).

Hight et al. 1994 confirmed that the clay content is very important for the penetration behaviour in clayey silts and clayey sands. Penetration through the clayey sands is marked by large variations in both q_c and u which is due to variations in clay content. Hight et al. 1994 also found a relation that could be used to assess the variation in clay content and the drainage conditions during penetration (compare Figure 19) (compare Lunne et al 1997). Noticeable is that two types of piezocone were used (on the cone phase u_1 and behind the cone u_2).

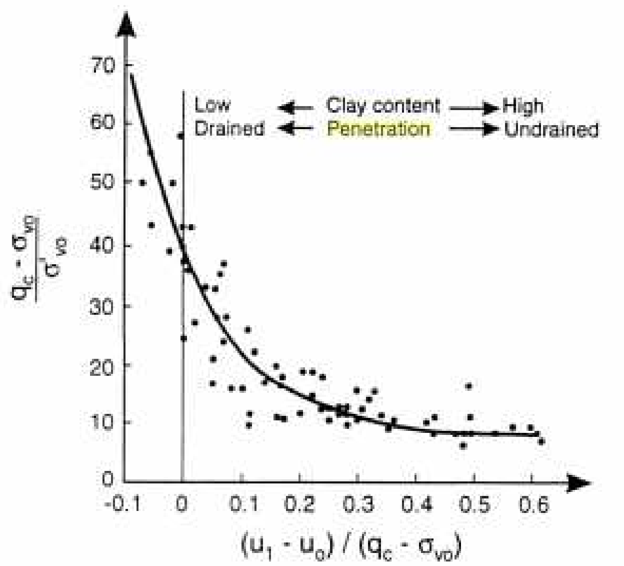


Figure 19: Assessment of Insitu variation of clay content Hight et al. 1994 (Lunne et al 1997, page 96)

For the interpretation of CPT data in silty soils it is important to identify the drainage conditions expected in the design problem and during the cone penetration. If the design problem will involve undrained shear strength and cone penetration is also undrained, the CPT data can be interpreted in a manner similar to clay. On the other hand, if the design problem is drained loading and the cone penetration is also drained, the CPT data can be interpreted in a similar manner to sand. Interpretation is more difficult if the design problem is expected to involve drained loading but the cone penetration process is undrained or partially drained. In this case effective stress strength parameters are needed for design (Lunne et al 1997).

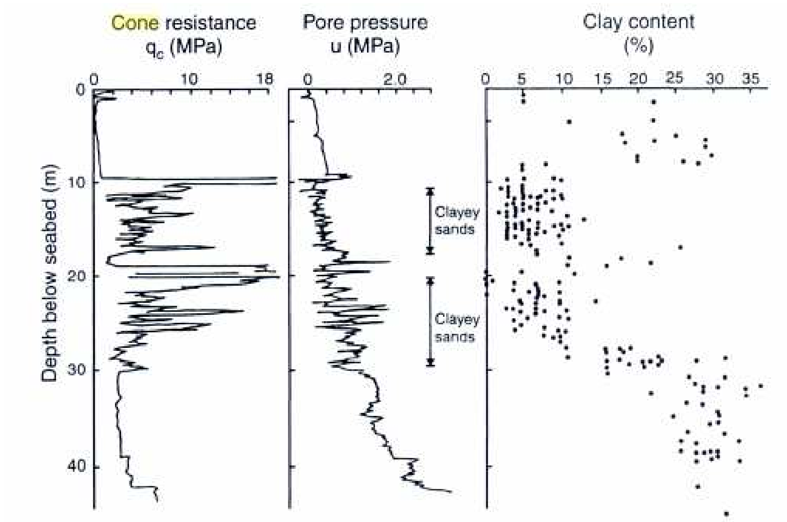


Figure 20: Typical CPTU profile and its correlation with clay content by Hight et al. 1994 (Lunne et al. 1997)

2.5.2.2 Peat and organic soils

Peats are derived from plants and can therefore be very fibrous material. As the degree of humification increases, the peat becomes less and less fibrous until it is transformed into an amorphous mass without any discernible structure (Lunne et al. 1997).
Landva et al. 1983 concludes that the effects of organic matter are probably not yet fully recognized by a geotechnical engineer. Peat fibres have a reinforcing effect and humus may act as a bounding agent. Landva (1986) reports that negative excess pore pressure would be generated at the location around the filter since the layers that are compressed by the cone point will tend to rebound as they are punctured and thus unload near the point. Landva (1986) concluded that due to the fibrous nature of peat and the frequent presence of obstructions like stumps and roots, small scale instu tests like the cone penetrometer or the vane tests are of little engineering use for the design of road embankments. However, in organic soils of non- fibrous nature, CPT and vane test can be very useful tools (compare Lunne et al. 1997).

A typical CPTu profile with peat layers shows high friction ratios, low or even negative friction ratios and low cone resistance. Mlynarek et al. 2010 conducted a study in organic soils where he concluded that the soil classification chart by Robertson (1990) leads to an erroneous interpretation of results- this is due to the organic soils that are located in the zone allocated for silty clays and sensitive soils. His CPT results showed- in contradiction to other studies conducted in organic materials- high values of cone resistance and a low friction ratio. Mlynarek et al. 2010 concluded that such a situation may be explained by the considerable admixture of the sand fraction in these soils, characteristic of soils of fluvial accumulation. Mlynarek et al. 2010 recommends the use of seismic measurements to provide a G_0 value and then use a SCPTU- classification- chart proposed by Robertson et al. 1995.

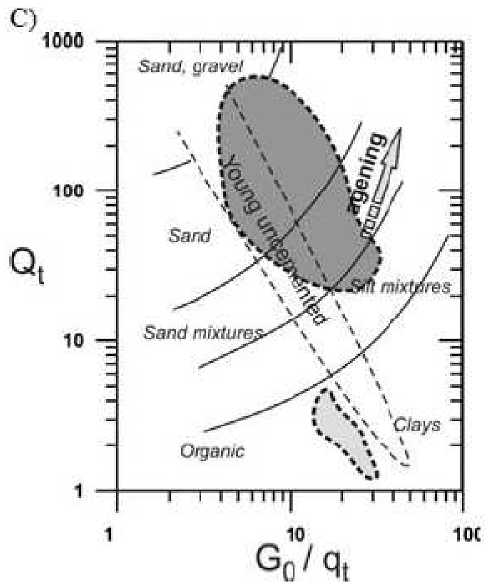


Figure 21: Soil classification chart developed by Robertson et al 1995 (Młynarek et al. 2010)

2.5.3 Intermediate Parameters

As already mentioned in the introduction, a CPT records the sleeve friction (f_s), the cone resistance (q_c) and the pore water pressure (u_2) is measured. Basically, all correlations that are introduced need intermediate or "corrected" values, which will be introduced here.

The problem is that there is not only one intermediate parameter for cone resistance or sleeve friction. When these correlations had been published, intermediate parameters were "modified", "corrected", some got normalized to provide a more reliable use in diagrams or graphs. Hence, care must to be taken when using correlations and its intermediate parameters.

For better clarification, the intermediate parameters are highlighted and the differences are outlined below:

As for the cone resistance, the following modifications are derived:

- The corrected total cone resistance (qt), that is (instead of q_c) widely used in correlations with geotechnical parameters. The cone resistance (q_c) should be corrected for porewater pressures, because of the action on unequal areas (shoulder area) of the cone tip. In soft clays and silts and in over water work, the measured q_c must be corrected for pore water pressures acting on the cone geometry, thus obtaining the corrected cone resistance, q_t :

$$q_t = q_c + u_2 * (1 - a)$$

With: q_t= corrected total cone resistance
q_c= cone resistance,
u_2= pore water pressure measured behind the cone
a= area ratio of the cone (=A_n/A_c), ratio of the cross sectional area of the load cell or shaft (An) divided by the projected area of the cone (A_c). The net area ratio is determined from laboratory calibration with a typical value between 0.70 and 0.85. In sandy soils q_c = q_t (Robertson 2010).

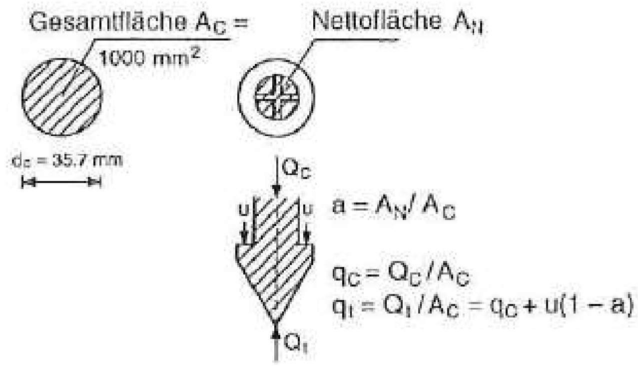


Figure 22: Definition of a CPT Cone with area ratio of the cone (Witt 2008)

Most 10 cm² commercial penetrometers have 0.75 < a < 0.82, whereas many 15-cm² cones show 0.65 < a < 0.8, yet several older models indicate values as low as a ~0.35. The value of a should be provided by the manufacturer.

- Moreover, there are other intermediate values- first introduced by Wroth (1984) that received wide recognition in literature:
  1. The normalized cone resistance (Q_t):
  $$Q_t = \frac{q_t - \sigma_{v0}}{\sigma_{v0}'} \; [-]$$
  2. A modified version of Q_t, named Q_{tn} (introduced first in Robertson 2009):
  $$Q_{tn} = \frac{q_t - \sigma_{v0}}{\sigma_{atm}} * \left(\frac{\sigma_{atm}}{\sigma_{v0}'}\right)^n \; [-]$$

With: q_t= total cone resistance
σ'_{vo}, σ_{vo} = vertical (overburden) stress (total and effective)
σ_{atm}= atmospheric pressure (0,1MPa)
n= stress exponent that varies with SBTn (Soil Behavioural Index):
n = 0.381 (I_c) + 0.05 (σ'_{vo}/p_a) – 0.15) with I_C obtained from Jefferies and Been (2006) and n ≤ 1.0 (p_a= atmospheric pressure: 0,1Mpa).

The modification with the varying stress exponent (that depends on the soil type) was first proposed by Robertson and Wride (1998) and updated by Zhang et al (2002) (compare Robertson 2009).

Most of the correlations are still using Q_t, whereas some of the newer developed ones already contain the modified normalized cone resistance (Q_{tn}).

As for the sleeve friction, several intermediate parameters are found and used in literature:

- friction ratio:
  1. with q_t (corrected total cone resistance)

$$R_f = \frac{f_s}{q_t}\ [\%]$$

2. with q_c (“uncorrected” total cone resistance)

$$R_f = \frac{f_s}{q_c}\ [\%]$$

- normalizied friction ratio:

$$F = 100 * \frac{\mathrm{f_s}}{(q_t - \sigma_{v0})}\ [\%]$$

with: f_s= sleeve friction
q_t=corrected cone resistance
σ_{vo}= total overburden stress

Another important parameter that is derived from the input values is the pore pressure parameter (only for CPTu):

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}}\ [-]$$

With: u_2= pore water pressure measured behind the cone
u_0= insitu (equilibrium) pore water pressure
q_t= corrected cone resistance
σ'_{vo}= effective overburden stress

2.5.4 Soil behavioural type

The CPT cannot be expected to provide accurate predictions of soil type based on physical characteristics, such as grain size distribution but provide a guide to the mechanical characteristics (strength and stiffness) of the soil, or the soil behaviour type (SBT) (Robertson 2010).

The first chart is derived by measurements from Furgo Company and relies on the friction ratio for evaluation of the soil type:

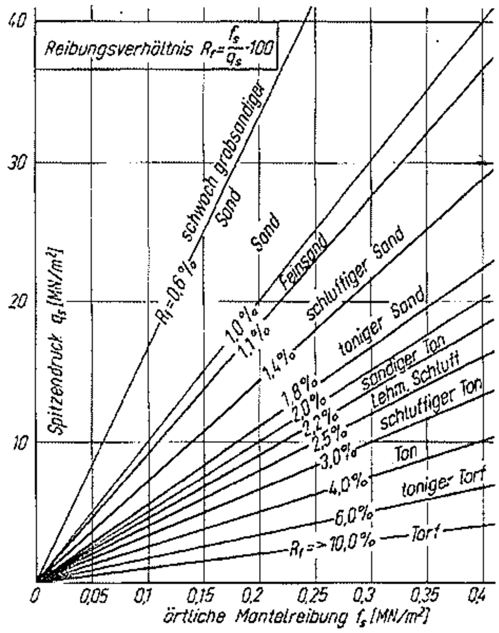


Figure 23: Soil type charts after measurements of Fugro Company

The next soil type chart needs the cone resistance (uncorrected) and the friction ratio (with q_c) as an input parameter. This figure is recommended in the DIN4094- 1, C1. It is remarkable that there is no zone for peat or other organic soils.

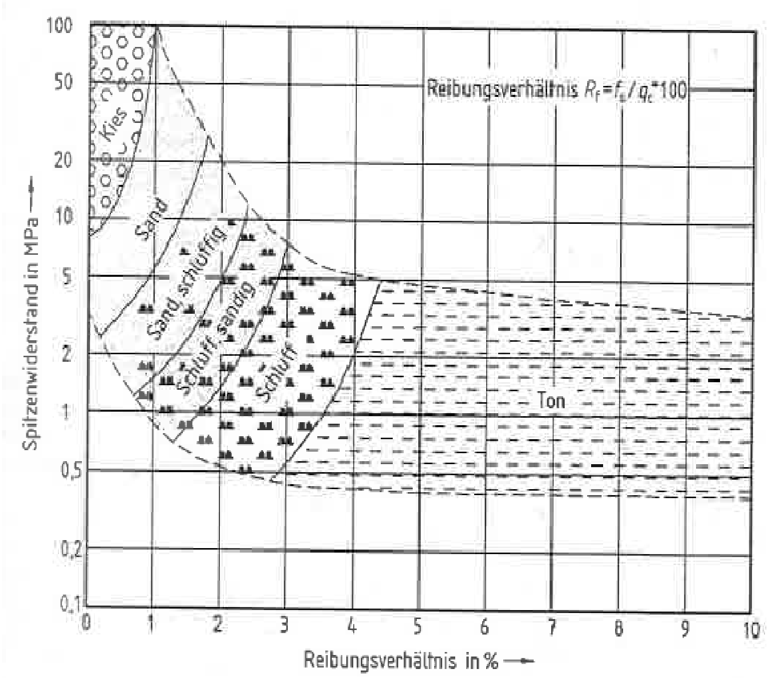


Figure 24: Soil type chart derived from measurements of GEOSOND Wollenhaupt GmbH, and as recommendation in DIN 4094- 1, C1;

The following two Soil Type charts are the most commonly used for the soil description. Robertson et al 1986 distinguishes 12 different zones, whereas the chart presented in 1990 shows 9 different zones and uses normalized parameters (Q_t, F_r).

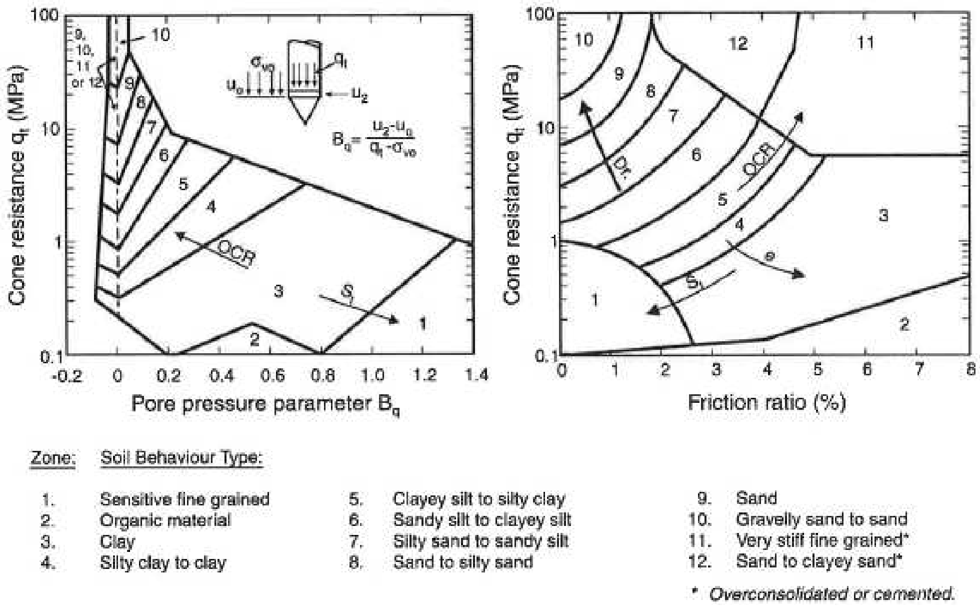


Figure 25: Proposed soil type classification for CPTu data by Robertson et al. 1986 (Lunne et al 1997)

The overburden stress and depth influence the measured penetration resistances. Therefore, it is more rigorous in the post-processing of CPT data to consider stress normalization schemes like the normalized cone resistance (Q_t) and the normalized sleeve friction (F_r) (compare Mayne 2007).

Robertson 1990 argued that in general, the normalized charts provide more reliable identification of SBT than the non-normalized charts, although when the in-situ vertical effective stress is between 50 to 150KPa there is often little difference between normalized and non-normalized SBT. The normalization was based on theoretical work by Wroth (1984). Robertson 1990 suggested two charts based on either Q_{t1} – F_r or Q_{t1} - B_q but recommended that the Q_{t1} – F_r chart was generally more reliable.

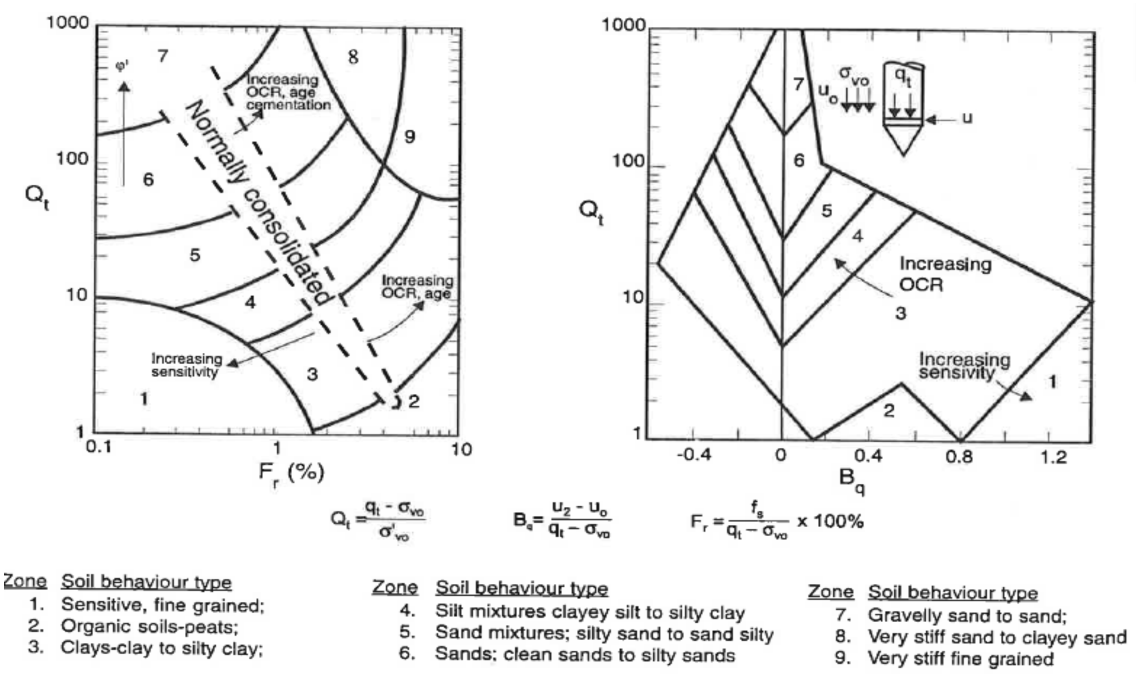


Figure 26: Soil behavioural type classification chart based on normalized CPT/ CPTu data (Robertson 1990)

Two soil type determination methods were introduced by Robertson and Wride 1998 and Jefferies and Been, 2006. Both introduced a material index (I_c or I_{cRW}) and transfer that value in a Soil Behavioral Type Zone. Jefferies and Been 2006 requires a CPTu result.

Roberton & Wride, 1998:

$$I_{CRW} = \sqrt{(3{,}47 - \log(Q)^2 + 1{,}22 + \log(F)^2)}$$

Jefferies& Been, 2006:

$$I_C = \sqrt{(3 - \log(Q + [1 - B_q] + a)^2 + (1{,}5 + 1{,}3 * \log(F))^2}$$

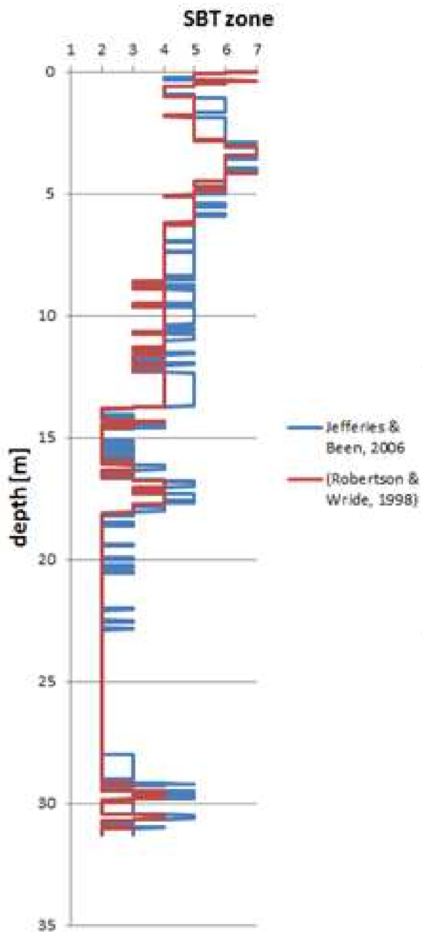


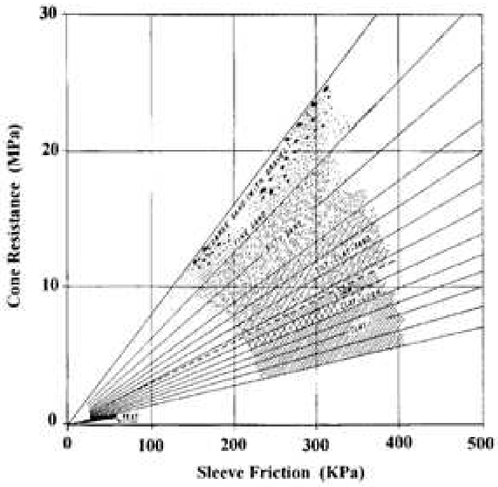
Table 4.1 Soil behavioral type by CPTu material index, I_c

Soil Classification	SBT Zone	Range CPTu Index, I_c	Range CPT Index I_{cRW}
Sands with gravels	7	$I_c < 1.25$	$I_{cRW} < 1.31$
Sands: clean to silty	6	$1.25 < I_c < 1.80$	$1.31 < I_{cRW} < 2.05$
Sandy mixtures	5	$1.80 < I_c < 2.40$	$2.05 < I_{cRW} < 2.60$
Silty mixtures	4	$2.40 < I_c < 2.76$	$2.60 < I_{cRW} < 2.95$
Clays	3	$2.76 < I_c < 3.22$	$2.95 < I_{cRW} < 3.60$
Organic soils	2	$I_c > 3.22$	$I_{cRW} > 3.60$
Sensitive soils	1	NA	NA

Notes: 1. Index I_c after Jefferies & Been (2006).
2. Index I_{cRW} after Robertson & Wride (1998).

Figure 27: Classifying the soil type, values and example from a project in Salzburg

For the sake of completeness further (older and less used) soil type charts will be presented:



Soil Type as a Function of Friction Ratio (Begemann, 1965)

Coarse sand with gravel through fine sand	1.2 %	-	1.6 %
Silty sand	1.6 %	-	2.2 %
Silty sandy clayey soils	2.2 %	-	3.2 %
Clay and loam, and loam soils	3.2 %	-	4.1 %
Clay	4.1 %	-	7.0 %
Peat			>7 %

Figure 28: Soil profiling chart (Begemann, 1965) (Fellenius and Eslami 2000)

The Begemann chart was derived from tests in Dutch soils with the mechanical cone. The chart is site-specific, i. e., directly applicable only to the specific geologic locality where it was developed. For example, cone tests in sand usually shows a friction ratio smaller than 1 %. However, the chart has important general qualitative value (compare Fellenius and Eslami 2000).

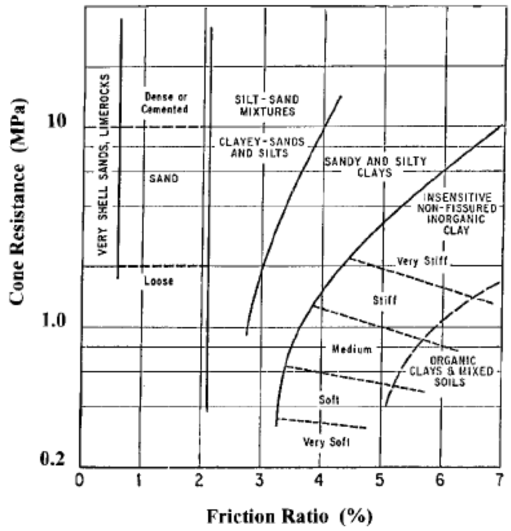


Figure 29: Profiling chart proposed by Schertmann 1978 (Fellenius and Eslami 2000)

The chart is based on results from mechanical cone data in “North Central Florida” and incorporates Begemann’s CPT data and indicates zones of common soil type. It also presents boundaries for loose and dense sand and consistency (undrained shear strength) of clays and silts, which are imposed by definition and not related to the soil profile interpreted from the CPT results (compare: Fellenius and Eslami 2000).

Douglas and Olsen (1981) were the first to propose a soil profiling chart based on tests with the electrical cone penetrometer:

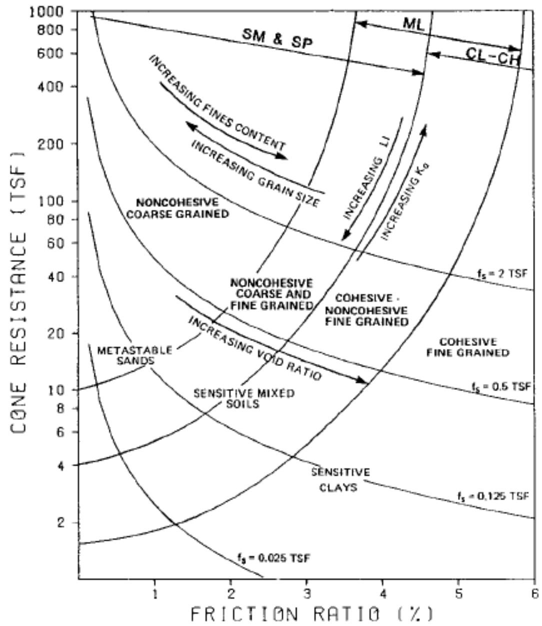


Figure 30: Soil chart by Douglas and Olsen (1981) (Fellenius and Eslami 2000)

They published a chart shown in Figure 30, which appends classification per the unified soil classification system to the soil type zones. The chart also indicates trends for liquidity index and earth pressure coefficient, as well as sensitive soils and "metastable sands" (compare: Fellenius and Eslami 2000).

Jones and Rust (1982) developed the soil profiling chart shown in Figure 31, which is based on the piezocone using the measured total cone resistance and the measured excess pore water pressure mobilized during cone advancement:

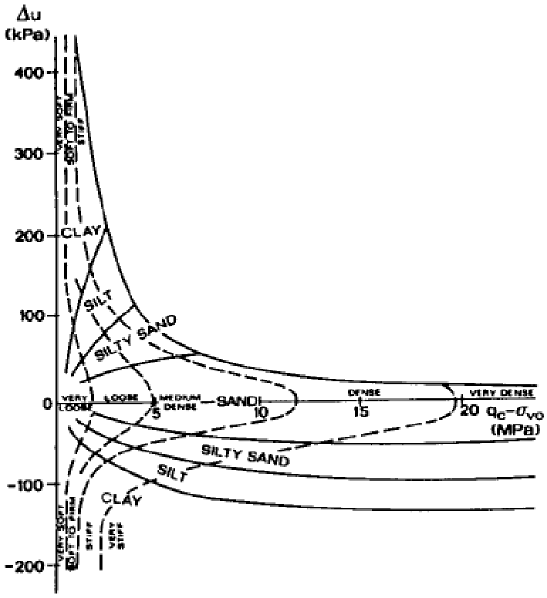


Figure 31: soil type chart by Jones and Rust (1982) (Fellenius and Eslami 2000)

The probably latest published chart is proposed by Eslami and Fellenius (1997). It is based on non-normalized parameters using effective cone resistance, q_e and f_s, where $q_e = (q_t - u_2)$. The effective cone resistance, q_e, suffers from lack of accuracy in soft fine-grained soils (Robertson, 2009).

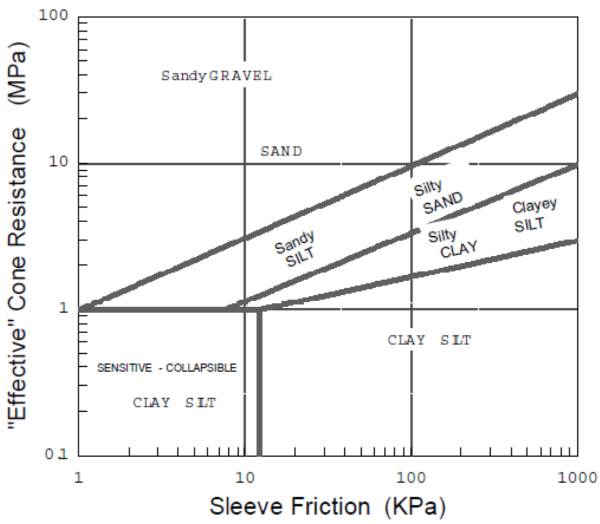


Figure 32: suggested Soil type chart by Fellenius, B. H., and Eslami 2000

According to Mayne 2007, at least 25 different CPT soil classification methods have been developed. The most recognized charts are from Begemann (1965), Schmertmann (1978a), and Robertson (1990).

For verification and application of the chart in the following project, only some charts are selected.

Used charts for soil type classification:

- Fugro
- DIN 4094-1, C- 1;
- Robertson 1986
- Robertson 1990
- Jeffrey and Beens, 2006

Projects on soil type evaluation:

- Project 3 (only comparison of Robertson 1986 and 1990, Figure 64)
- Project 5 (Figure: 80, 81, 82, 83)
- Project 6 (Figure: 86, 87, 88, 89, 90)
- Project 8 (Figure: 97, 98, 99)

Conclusion on soil type charts:

It can be seen that all soil type charts show accurate results for most soils.

The determination of peat or organic soils is problematic. All soil classification charts show wrong results when it comes to the determination of peat or organic at the Project 8 (compare: soil type charts like Fugro, DIN 4094-1, Robertson et al. 1990 and 1986). Z. Młynarek et al 2010 also highlights the problem of determining peat and organic soil. A possible approach when it comes to peat layers in the subsoil may be the soil type chart from Robertson 1995 (that requires the G_0 value as an input parameter).

In all projects Jefferies and Been 2006 shows higher SBT values than Robertson and Wride 1998 (compare Figure 27). Moreover, Jefferies and Been 2006 requires the pore pressure as an input parameter, which show some restriction on the application, as the measurement of the porewater pressure is neglected for some projects. When comparing the projects to the data from the borehole Robertson and Wride 1998 shows better results.

Comparing soil type classification after Robertson 1990 in softwares like NOVOCPT and GEOMIL show slightly inconsistent results, within the evaluation (compare: project 6).

One difference between the Robertson 1986 and the Robertson 1990 soil type chart is that a different number of zones are introduced. Especially, in Robertson 1990, zone type two includes “organic soils and clay”, which would be important to distinguish for further calculation purposes (essential for instance at Project 3and Project 8). Robertson 1986 distinguishes between clay (“zone 3”) and organic soils (“zone 2) which is significant for classification purposes.

2.5.5 Undrained shear strength

Basically there are 3 main types of approaches available to estimate the undrained shear strength (s_u) empirically:

- Using the total cone resistance
- Using the effective cone resistance
- Using excess pore pressure

Nevertheless, any researchers also tried to develop theoretical solutions based on models like the cavity expansion theory, strain path theory, conservation of energy combined with cavity expansion theory, classical bearing capacity theory as well as analytical and numerical approaches. The equation is:

$$q_c = N_c * c_u + \sigma_0$$

Note: All these approaches are aimed to determine the *N*- value. The notation differs in literature, and hence was used in accordance with Kim et al. 2006.

Since the cone penetration is a complex phenomenon, all the theoretical solutions make several simplifying assumptions regarding soil behaviour, failure mechanism and boundary conditions. The theoretical solutions need to be verified from actual field and/or laboratory test data. Theoretical solutions have limitations in modelling the real soil behaviour under conditions of varying stress history, anisotropy, sensitivity, ageing and macrofabric (Lunne et al. 1997).

However, according to Lunne et al (1997) some researchers emphasized that N_k is related to a plasticity index I_P, and plotted correlations between N_k and I_P (Lunne at el. 1976, Baligh et al. 1980, Lunne and Kleven 1981, Aas et al. 1986, Rochelle et al. 1988).

In contrast, Kim et al. 2006 concluded that, those papers do not account for the dependence of c_u on the rate of loading. Therefore, the value of N_k from theoretical solutions has to be increased to account for rate effects. The correction for "viscosity" is shown in Figure 34. As shown in Figure 33, the theoretical N_k decreases with increasing I_P. On the other hand, "viscosity" increases with increasing I_P. By way of combining these two values, N_k has a fairly constant value for soils having different I_P values, and the value is somewhat bigger than 10 (compare Figure 34).

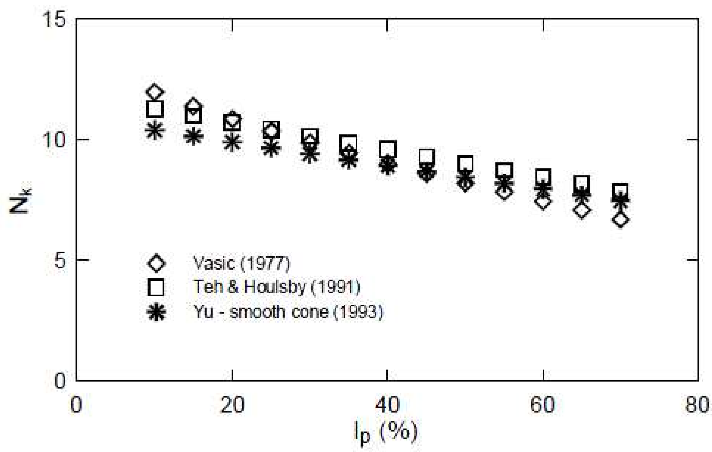


Figure 33: N_k values obtained from theoretical solutions and IP based on the correlation I_P and N_k (Kim et al 2006);

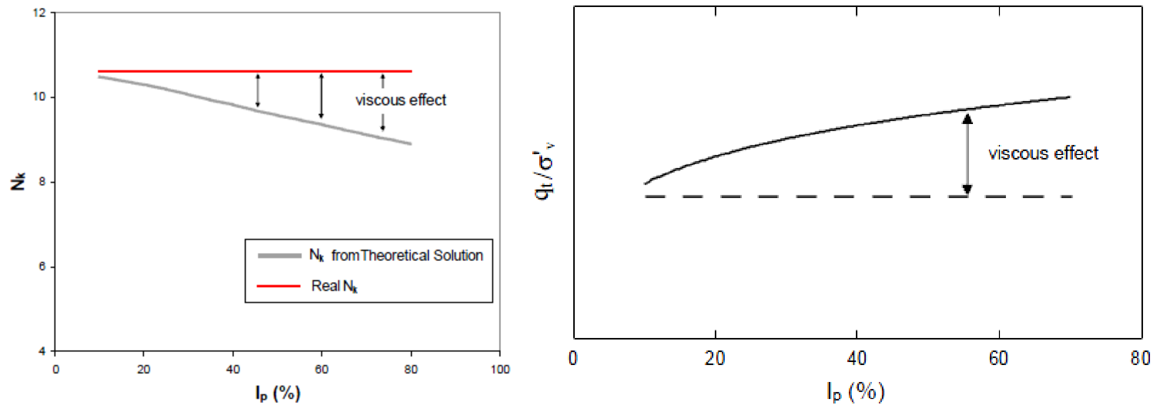


Figure 34: Left: correction of actual values of N_k and I_P, right: correction of vicious effects in q_t and I_P (Kim et al, 2006)

Kim et al. 2006 claims that all cone factors N_k derived from cavity expansion solutions depend on the rigidity index I_R (suggested values from different authors including methods of Baligh (1985) and Vesic (1975), compare Figure 35).

For clays, the rigidity index I_R (defined as $I_R = G/c_u$) can be obtained from measured triaxial stress-strain curves or pressuremeter tests. Nevertheless, Keaveny and Mitchell (1986) suggested a useful empirical correlation for I_R based on anisotropically-consolidated triaxial compression test data in terms of OCR and plasticity index (compare Figure 36) (Kim et al. 2006).

N_c	Penetration model	Reference
7.0 to 9.94	BCT	Caquot & Kerisel (1956); de Beer (1977)
$3.90+1.33\ln I_r$	CET	Vesic (1975)
$12+\ln I_r$	CET	Baligh (1975)
$\left(1.67+\frac{I_r}{1500}\right)(1+\ln I_r)+2.4\lambda_c-0.2\lambda_s-1.8\Delta$	SPM+ FEM	Teh & Houlsby (1991)
$0.33+2\ln I_r+2.37\lambda-1.83\Delta$	FEM	Yu et al (2000)
$\frac{1+A_r}{\sqrt{1+2A_r}}\ln I_r+\frac{1-A_r}{3}+R\left(1+\frac{1+A_r}{\sqrt{1+2A_r}}+0.52A_r^{1/8}(1+A_r)\right)-2\Delta$	CET+ FEM	Su & Liao (2002)
$2.45+1.8\ln I_r-2.1\Delta$	CET+ FEM	Abu-Farsakh et al (2003)

Note: $I_r = G/S_u$; $\Delta = \frac{\sigma'_{vo}(1-K_o)}{2S_u}$; λ= cone roughness; A_r=strength anisotropy ratio.

Figure 35: Theoretical cone factor solutions (Schnaid 2005)

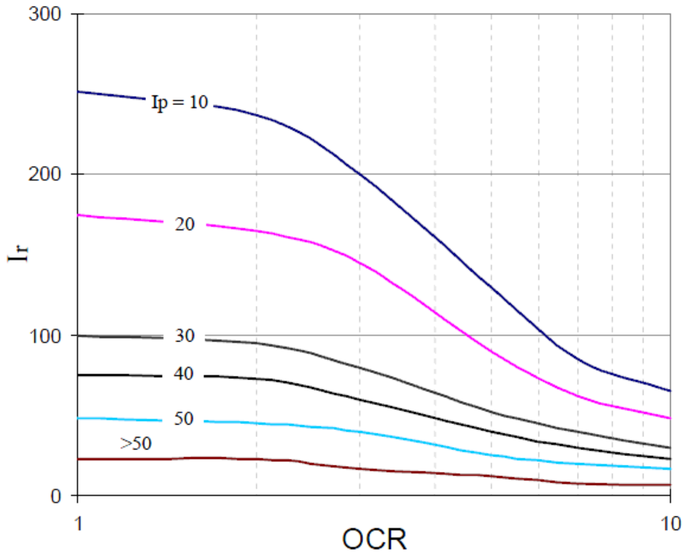


Figure 36: chart for estimating the rigidity index for fine grained soil after Keaveny and Mitchell 1996 (after Kim et al. 2006)

Kim et al. 2006 summarises that values from theoretical solutions for N_c vary between 8.5 and 13 for I_R ranging from 50 to 400 [-].

Empirical estimations using the <u>total cone resistance</u> is made from the following equation:

$$c_u = (q_t - \sigma_{v0})/N_k$$

Knowledge of the cone factor N_k is essential for reliable estimation of c_u from q_c, and numerous attempts have been made by researchers to develop accurate N_{kt} values by empirical approaches (Lunne and Kleven 1981, Aas et al. 1986, Rochelle et al. 1988, Lunne et al. 1986, Stark and Juhrend 1989). The approach to determine N_k has traditionally been to perform the CPT, recover samples, and then test them in the laboratory to obtain c_u. Alternatively, vane shear tests can be performed side-by-side with the CPT to estimate c_u (Aas et al. 1986, La Rochelle et al. 1988). The cone factor is then estimated using the equation for the total cone resistance, given that q_c, σ_v, and c_u are all known. However, as noted by Lunne et al. 1976, there are limitations on the accuracy of c_u determinations from the vane test that are related to the direction and rate of shearing. Therefore, empirical correlations between q_c and values of c_u obtained based on field vane shear tests tend to be less reliable than those based on laboratory measurement of c_u (compare: Kim et al. 2006).

To show the range of those N_k values, Aas et al. 1986 suggests a range of 8 to 16, whereas for example the DIN 4094-1 proposes the N_k value in a range of 10-20.
In most software tools, the total cone resistance is used to calculate c_u (for example *GEOMIL CPTask* calculates with Nk- values from 12- 20). *NOVOCPT* recommends a value of 12,5 but the user is able to change it.

Table 2: values and ranges of N_k (after Lunne et al. 1997)

Reference	N_k- value/ range

Aas et al (1986)	8- 16
La Rochelle et al (1988)	11- 18
Rad Lunne (1988)	8- 29
Powell and Quaterman (1988)	10- 20
Luke (1995)	8,5- 12
Software	
NOVOCPT	12,5*
GEOMIL CPTask	12- 20

*Possible to change the value manually.

Another estimation of the c_u value can be made by using the <u>effective cone resistance</u>. Senneset et al. 1982 suggested the use of the "effective cone resistance q_e to determine c_u, where q_e is defined as the difference between the measured cone resistance and pore pressure, measured immediately behind the cone (u_2). Campella et al. 1982 redefined the effective cone resistance, using the corrected cone resistance q_t, c_u can then be found using the relation (compare Lunne et al. 1997):

$$c_u = \frac{q_e}{N_{ke}} = (q_t - u_2)/N_{ke}$$

Again it is difficult to define the N_{ke} value and hence it was topic of many publications. Here, some values/ ranges are presented (after Lunne et al 1997):

Table 3: values and ranges of N_{ke}

Reference	N_{ke}- value/ range
Senneset et al (1982)	9±3
Lunne et al. (1985)	1- 13
Software	
NOVOCPT	9*
GEOMIL CPTask	----

*Possible to change the value manually.

Lunne et al 1997 concluded that for some deposits this procedure works well but, in general, it is not recommended to estimate c_u using an "effective" cone resistance q_e. In soft normally consolidated clays, the total pore pressure generated behind the cone is often approximately 90% or more of the measured cone resistance. A major disadvantage of using q_e to interpret c_u in such soils, therefore is that q_e is a very small quantity, sensitive to small errors in q_c or u measurements (in such soils, use of excess pore pressure (Δu) may be more accurate).

The third possibility of determining c_u is to use the <u>excess pore pressure</u>:

In very soft clays, where there may be some uncertainties with the accuracy in q_t, estimates of c_u can be made from the excess pore pressure (Δu) measured behind the cone (u_2) using the following (Robertson 2010):

$$c_u = \Delta u / N_{\Delta u}$$

With: $\Delta u = u_2 - u_0$

Some papers could find a dependancy of $N_{\Delta u}$ on B_q, whereas others couldn´t . Non of the available software programs use the excess pore pressure to define the undrained shear strenght.

Table 4: values and ranges of NΔu (after Lunne et al 1997)

Reference	**$N_{\Delta u}$- value/ range**
Lunne et al. (1985)	4- 10 (North seas clays)
La Rochelle et al. (1988)	7- 9 (Canadian clays)
Karlsrud et al. (1996)	6-8
Software	
NOVOCPT	----
GEOMIL CPTask	----

Comparisons of correlations for the undrained shear strength are conducted in the following projects:

- Project 3 (Figure: 54, 55, 56, 57)
- Project 4 (Figure: 69, 70)
- Project 8 (Figure: 96)
- Project 9 (Figure: 102)

Conclusion on undrained shear strength

The N- value is the decisive factor to evaluate the undrained shear strength. As already mentioned the c_u value should be determined by shear vane tests or laboratory measurements (more reliable results as vane test) and then recalculate the N value for further performances of CPT.

However, the results from empirical and theoretical approaches show good accordance. For some projects differences have been encountered when using both the total cone resistance and the effective cone resistance. In general, estimations using the total cone resistance show more reliable results.

If laboratory results (I_P) are available, the approach from Keaveny and Mitchell 1996 also shows good results.

As for the most geotechnical correlated parameters for CPTu, local experience is an important factor that should not be neglected. Eventually, Lunne et al. 1997 recommends the following:

1. For deposits where little experience is available, estimate c_u using the total cone resistance (q_t) and preliminary cone factor values (N_{kt}) from 15- 20. For a more conservative estimate, select a value close to the upper limit. For normally and lightly overconsolidated clays N_{kt} can be as low as 10 and in stiff fissured clay can be as high as 30. In very soft clays where there may be some uncertainty with the accuracy in q_t, estimate c_u from the excess pore pressure (Δu_2) measured behind the cone using $N_{\Delta u}$ from 7 to 10. For a more conservative estimate, select a value close to the upper value.
2. If previous experience is available in the same deposit, the values suggested above should be adjusted to reflect this experience.

3. For larger projects, where high- quality field data and laboratory data may be available, site specific correlations should be developed based on appropriate and reliable values of c_u.

2.5.6 Constrained modulus

Correlations have been developed to determine the constrained modulus E_s as measured in the Oedometertest (tangent modulus). Most equations have the following basic form:

$$E_S = \alpha * q_c$$

Or (being more precise) including the overburden stresses:

$$E_S = \alpha * (q_c - \sigma_{v0})$$

Basically, all relationships of α rely on empirical estimations. Lots of tables have been developed for determining the α value or setting it in certain limits. In Central European countries a very common approach for those α- values is provided in the German standard (DIN 4094-1, D6 compare Figure 37).
Influencing factors like the loading, geometry of the raft foundation or thickness or the layers where possible settlements can occur are neglected in the simple relationship described in the DIN 4094-1, D6 that is based on the soil type encountered. It should be used as a rough approximation:

leicht plastischer Ton (TL)	$q_c < 0{,}7$ MPa	$3 < \alpha < 8$
	$0{,}7$ MPa $< q_c < 2$ MPa	$3 < \alpha < 8$
	$q_c > 2$ MPa	$1 < \alpha < 2{,}5$
leicht plastischer Schluff (UL)	$q_c < 2$ MPa	$3 < \alpha < 6$
	$q_c > 2$ MPa	$1 < \alpha < 2$
ausgeprägt plastischer Ton (TA)	$q_c < 2$ MPa	$2 < \alpha < 6$
ausgeprägt zusammendrückbarer Schluff (UA)	$q_c < 2$ MPa	$1 < \alpha < 2$
stark organischer Schluff (OU)	$q_c < 1{,}2$ MPa	$2 < \alpha < 8$
Torf und stark organischer Ton (H, OT)	$q_c < 0{,}7$ MPa	
abhängig vom natürlichen Wassergehalt w_n	$50\ \% < w_n < 100\ \%$	$1{,}5 < \alpha < 4$
	$100\ \% < w_n < 200\ \%$	$1 < \alpha < 1{,}5$
	$w_n > 500\ \%$	$\alpha < 0{,}4$
schluffiger Sand	$\alpha = 2$	
Fein- und Mittelsand	$\alpha = 3{,}5$	
Grobsande und kiesige Sande	$\alpha = 5$	
sandige Kiese und Kiese	$\alpha = 6$	

Figure 37: correlations for the constrained modulus for different soils (DIN 4094-1 D-6)

Very similar to that is the table of α- values that is introduced by Mitchell and Gardner 1975 (just for clays, compare Figure 38).
Noticeable is the range of α- values for "silts of low plasticity (ML) when comparing to the German standard DIN 4094- 1 "leicht plastischer Schluff". Furthermore, information on the α- value for range

of 0,7 <q_c< 2,0MPa at “leicht plastischer Ton” is different to the values given in Mitchell and Gardner 1975. Probably, it is a mistake in the DIN 4094- 1 D-6.

$q_c < 0.7$ MPa	$3 < \alpha_m < 8$	Clay of low plasticity (CL)
$0.7 < q_c < 2.0$ MPa	$2 < \alpha_m < 5$	
$q_c > 2.0$ MPa	$1 < \alpha_m < 2.5$	
$q_c > 2$ MPa	$3 < \alpha_m < 6$	Silts of low plasticity (ML)
$q_c < 2$ MPa	$1 < \alpha_m < 3$	
$q_c < 2$ MPa	$2 < \alpha_m < 6$	Highly plastic silts and clays (MH, CH)
$q_c < 1.2$ MPa	$2 < \alpha_m < 8$	Organic silts (OL)
$q_c < 0.7$ MPa		Peat and organic clay (P_t, OH)
$50 < w < 100$	$1.5 < \alpha_m < 4$	
$100 < w < 200$	$1 < \alpha_m < 1.5$	
$w > 200$	$0.4 < \alpha_m < 1$	

Figure 38: table of α- values from Mitchell and Gardner 1975;

Table 1 summaries published α- values and ranges proposed by different authors, in accordance to the basic equation introduced (after Lunne et al. 1997):

Table 5: values and ranges of proposed α- values

Reference	α- value/ range
Mayne (2001)	8
Kulhawy and Mayne (1990)	8.25
Senneset et al. (1989)	4- 8
Meigh (1987)	2- 8

Mayne 2001 and Kulhawy and Mayne, 1990 even suggested α being the same value for all soil types.

A very different approach that is more complex is also described in the DIN 4094-1, D5. That approach was developed for a cone with a 10cm² base area, however it is widely applied also for the 15cm² cone types:

$$E_s = v * p_a * [\frac{\sigma_{ü} + 0,5 * \Delta\sigma_z}{p_a}]\hat{}w$$

With:
The vertical stresses at the foundation level $\sigma_{ü} = \gamma * (d + z)$
E_S= constrained modulus
v=stiffness factor
w=stiffness exponent (w=0,5 for sands, and w=0,6 for clays of low and medium plasticity)
p_a= atmospheric pressure (p_a=0,1MPa)
$\Delta\sigma_z$= increase of the vertical stresses at the foundation level due to loading;

The determining part in that equation is the stiffness factor that is obtained using (depending on the q_c value) the following diagrams.

For $0,6 \leq q_c \leq 3,5$:

$$v = 15{,}2 * \log(q_c) + 50$$

And for $5 \leq q_c \leq 30$:

$$if\ U \leq 3, v = 463 * \log(q_c) - 13$$

$$if\ U > 6, v = 167 * \log(q_c) + 113$$

Knowledge of the Coefficient of Uniformity (from Laboratory tests, grain size distribution) and the cone resistance is required to determine the stiffness factor (when q_c >5MPa).

The range for q_c, when determining the stress factor, is noticeable. The range of q_c between 3,5 and 5MPa is missing and no comment is given about that in the DIN 4094- 1.

Robertson (2009) developed an equation for the α- value that is related on the soil type index with the basic equation:

$$E_S = \alpha * (q_c - \sigma_{v0})$$

For Ic>2,2:

$$\alpha = Qt\ when\ Qt < 14$$
$$\alpha = 14\ when\ Qt > 14$$

For Ic<2,2:

$$\alpha = 0{,}03 * \left[10^{0{,}55*Ic+1{,}68)}\right] * (q_t - \sigma_{v0})$$

Lunne and Christophersen (1983) provided slightly different correlation for the constrained modulus for NC, unaged and uncemented silica sands (compare Lunne et al 1997):

$E_s = 4 * q_c$ for $q_c < 10MPa$

$E_s = 2 * q_c + 20\text{MPa}$ for $10MPa < q_c < 50MPa$

$E_s = 120MPa$ for $q_c > 50MPa$

And for OC sands:

$E_s = 5 * q_c$ for $q_c < 50MPa$

$E_s = 250MPa$ for $q_c > 50MPa$

Note: GEOMILCPTask- and the NOVOCPT software don´t provide any correlation to the constrained modulus Es, whereas other softwares like Datgel CPT Tool [1]do.

Comparisons of correlations for the constrained modulus are conducted in the following projects: For most projects, values/ ranges for the E_S (mostly from laboratory tests and local experience) from the geotechnical report are available.

[1] Datgel CPT Tool gINT Add-In 2.2, User guide

Table 6: Evaluation of correlations with projects

Project	*Figure(s)**	DIN 4094-1, D-5	DIN 4094-1, D-6	Robertson 2009,	Lunne and Christophersen, 1983	Mayne 2006	Results from geotechnical report
Project 3	*58, 59*	+	+	+	+	+	+
Project 4	*67, 68*	+	+	+	-	+	+
Project 1	*45*	+	+	-	+	-	-
Project 9	*101*	+	+	+	-	+	+
Project 8	*95*	+	+	+	-	+	+

* Comparions can be found in the following figures

Note: + correlation on the project available

- correlation on that project not available

Conclusion on the constrained modulus

DIN 4094- 1, D- 6 shows a good however, very conservative initial approximation. Quite often, the correlation result in a wide range of values. The application of the DIN 4094- 1, D- 6 requires (subjective) input decisions by the user, which can be seen as an uncertainty.
Robertson 2009- including I_c Soil Behavioural Type as the decisive factor- is often problematic because of the possible mistakes/ miscalculations in the Ic value. Robertson 2009 and Mayne overall show very high values compared to other correlations.
Lunne and Christophersen (1983) is (as stated) only valid for sands, which is a significant limitation for the application of this correlation. But it shows good values in sands compared to other correlations or results from the geotechnical report. Even if it is just applicable for clean sands it can be taken as a first approximation when there is a lack of local experience and no boreholes were drilled to examine the soil type.
The DIN 4094-1, D-5 (the stress depending approach) shows best estimation for the constrained modulus, when comparing with the results available. But the range of q_c from 3,5 to 5 is missing. Basically it requires differentiation between sands and clays (which can be easily done with the use of the soil type diagrams) but apart from that, it is very simple to apply for the user.
In general, close to the ground surface results for all correlations for the constrained modulus must be treated carefully.

2.5.7 Young´s modulus

According to elastic theory the Young's modulus of soil can be expressed as:

$$E = \frac{2}{1+\nu} * G_0$$

where ν = 0.2 applies for drained and ν = 0.5 for undrained conditions (Datgel User Guide, 2011)

Besides the basic equation mentionened above, Robertson 2009 introduced drained Young's modulus from CPT for young, uncemented silica sands (prerequest: Ic<2,6):

$$E = 0{,}015 * \left[10^{0{,}55*Ic+1{,}68)}\right] * (q_t - \sigma_{v0})$$

The Young´s modulus in the equation above has been defined at a mobilized strain of about 0.1%.

Another approach for sands, that is used by the NOVO CPT Software, was introduced by Bellotti et al. 1989. A chart is suggested (see Figure 39) to estimate the secant Young´s Modulus for a average axial strain of 0,1%. Both OCR and ageing are shown to have significant effects on the measured modulus that, taken into consideration, can provide a representative stiffness for most foundation problems (Schnaid 2009).

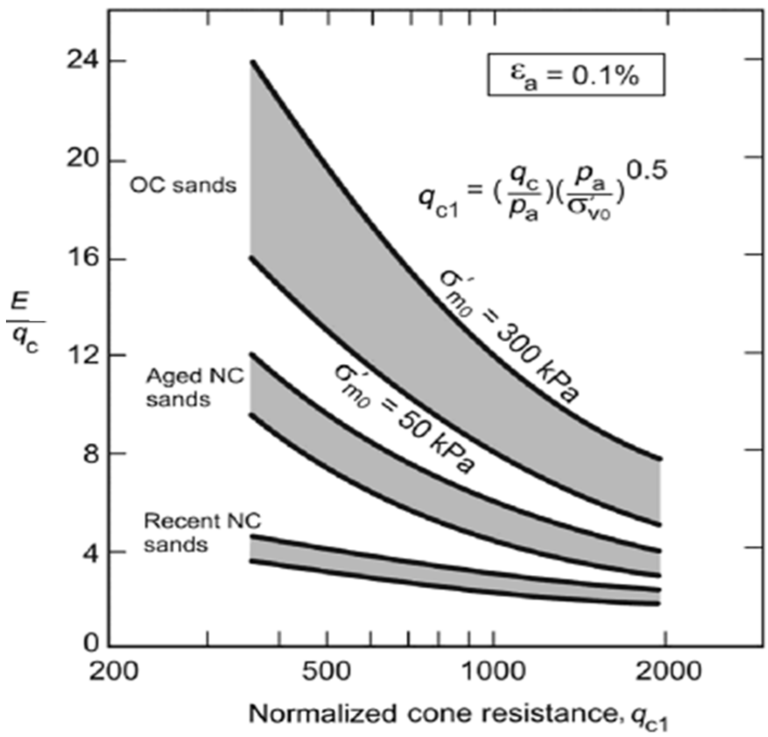


Figure 39: Young´s modulus for silica sands, (Bellotti et al., 1989)

The Young's modulus for clays is often referred to a stiffness ratio E/ c_u as a function of I_p (for example Ladd et al. 1977, compare Figure 40).

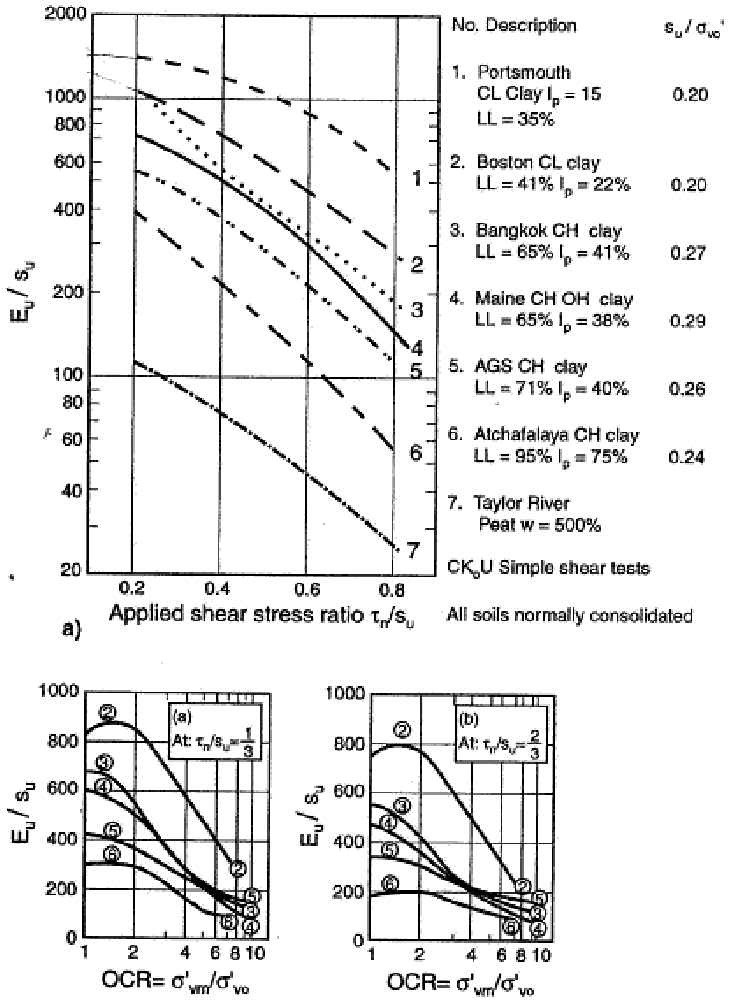


Figure 40: Stiffness ratio E/ c_u as a function of Ip after Ladd et al., 1977 Lunne et al 1997)

NOVOCPT software used the chart (see Figure 41) proposed by Duncan and Buchignami, 1976, that shows the Stiffness ratio E/ c_u as a function of OCR, for the evaluation of the Young's modulus for clays (the user should be aware that this is the undrained Young´s modulus for a mobilized strain of 0.25%!):

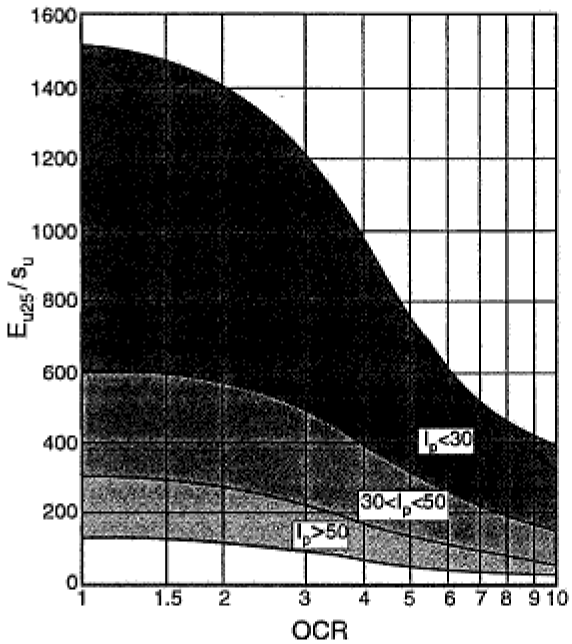


Figure 41: Stiffness ratio E/ c_u, as a function of OCR diagram developed by Duncan and Buchignami, 1976 (Lunne et al 1997)

Comparisons of correlations for the Young´s modulus are conducted in the following:

- Project 3 (Figure: 63)
- Project 4 (Figure: 73)

Conclusion on Youngs´ Modulus:

When evaluating the Young's modulus with NOVOCPT software (sand: Bellotti et al., 1989; clay: Duncan and Buchignami, 1976) results obtained seem to be high and almost impossible (especially the area of sands at Project 4).

For the Project 4 at a depth of 24 – 30m laboratory results (I_P) are available and when estimating the OCR (for instance using Powell et al, 1988) and c_u (through laboratory tests or CPT correlations) an estimation of Young's modulus can be conducted using Duncan and Buchignamis´ 1976 approach.

Generally, it is difficult to compare the different types (drained or undrained Youngs modulus). The undrained Young´s modulus is higher (because of a higher v).

Estimation of the Young´s Modulus using CPT <u>and</u> laboratory results (Project 4):

Clay (Duncan and Buchignami, 1976): At Project 4 (depth of 24- 30m, mostly clay) for example, the Young´s modulus reaches values that are 10 times higher than the values from the Geotechnical report. This may be due the ignorance of the I_P value (and perhaps also range of c_u, the OCR may be approximately calculated in NOVOCPT).

When evaluating Project 4, knowing that I_P=43% (laboratory tests) and c_u is in a range of 50 and 220KPa (compare CPT correlation with approach from Aas et al, 1986) and a range of OCR =1-2 (compare CPT correlations for OCR from Powell et al 1988 in clay), the Young's modulus shows feasible results in a range of 25- 110MPa. This procedure is also recommended by Lunne et al 1997.

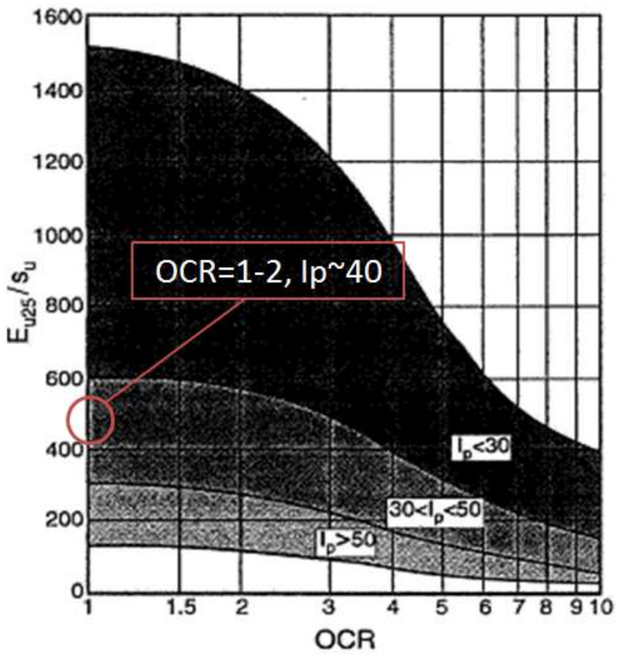


Figure 42: Evaluation of Young´s Modulus after Duncan and Buchignami 1976;

Sand (Bellotti et al., 1989, compare Figure 39): For the evaluation, the history of the soil has to be known and has a big influence. As it is problematic to implement that in a software and local experience is of high importance, is it very difficult to estimate a proper value of the Young´s modulus with that diagram.

A side note on the converting of the Young´s modulus to the constrained modulus following the Hooks law:

$$E_s = \frac{(1-\nu)}{(1+\nu)*(1-2*\nu)}*E$$

For the comparison the Young´s modulus from Robertson 2009 and a poisson´s ratio of 0,3 was used for the comparison. It is evaluated in Project 4 and doesn´t show any accordance (for E_S). The Young´s modulus derived by Robertson 2009 is already small compared to Bellotti et al. 1989 and still shows higher values than the stress dependent constrained modulus and the result from the geotechnical report (compare Figure: 74).

2.5.8 Effective friction angle φ´

Robertson and Campanella (1983) suggested a correlation to estimate the friction angle (φ´) for uncemented, unaged, moderately compressible, predominately quartz sands based on calibration chamber test results. For sands of higher compressibility (i.e. carbonate sands or sands with high silica content), the method will tend to predict low friction angles (Robertson 2010).

$$\tan \varphi´ = \frac{1}{2.68} * [\log\left(\frac{q_c}{\sigma_{v0}´}\right) + 0.29]$$

Kulhawy and Mayne (1990) suggested a relationship for clean, rounded, uncemented quartz sands, and came up with the equation using field data (Robertson 2010):

$$\varphi´ = 17{,}6 + 11 * \log(Q_{tn})$$

With Q_{tn}:

$$Q_{tn} = (\frac{q_t}{\sigma_{atm}})/(\frac{\sigma_{v0}´}{\sigma_{atm}})\hat{}0.5$$

Alternatively, an effective stress limit plasticity solution for undrained cone penetration developed at the Norwegian Institute of Technology (NTH: Senneset et al., 1989) allows the approximate evaluation of effective stress parameters (c' and φ´) from piezocone measurements. In a simplified approach for normally- to lightly overconsolidated clays and silts (c' = 0), the NTH solution can be approximated for the following ranges of parameters: 20º ≤ φ´ ≤ 45º and 0.1 ≤ B_q ≤ 1.0 (compare Mayne 2006):

$$\varphi´ = 29{,}5 * B_q^{0{,}121} * [0{,}256 + 0{,}336 * B_q + \log(Q_t)]$$

For heavily overconsolidated soils, fissured geomaterials, and highly cemented or structured clays, the above will not provide reliable results and should be verified by laboratory testing.

Also, the NOVOCPT software uses that equation for determining the effective friction angle in clays.

Another correlation just for sands is provided by the EN 1997- 2: 2010 and the DIN 4094-1, D3, (scope of application: 5 MPa< q_c < 28 MPa):

$$\varphi´ = 23 + 13{,}5 * \log(q_c)$$

Further, referring to Hutchinson 2001 another correlation in sands for the friction angle has been developed (after Witt 2008), (scope of application: 6,9 MPa< q_c < 42,5 MPa):

$$\varphi´ = 26{,}8 + 4{,}5 * \ln(q_c)$$

Comparisons of correlations for the effective friction angle φ´ are conducted for the following projects:

- Project 1 (Figure: 46)
- Project 3 (Figure: 60, 61)
- Project 4 (Figure: 71, 72)
- Project 5 (Figure: 79)
- Project 6 (Figure: 85)
- Project 9 (Figure: 103)

Conclusion on friction angle φ´:

Almost all equations for the friction angle are developed for sands. Only two correlations (Sunneset et al. 1989 and Mayne 2006, NTH solution) on clay exist that is used in the software NOVO- CPT.

Foremost, it´s eye-catching that all correlations from sands previously introduced are derived just from the cone resistance (or its intermediate parameters). Sleeve friction and the dynamic pore water pressure is never implemented in φ´- correlations except in the equation for clays developed by Mayne 2006 that is rarely suitable for practical use because of its requirement of $0.1 \leq B_q \leq 1.0$.

For clays, Sunneset et al. 1989 and Mayne 2006, show the same results. The only difference is that, when implementing Mayne 2006, it doesn´t provide any results for certain ranges (compare project: Project 4). Overall, the results of Sunneset et al. 1989 and Mayne 2006 are much lower, when comparing to the geotechnical report where values are obtained from local experience or laboratory tests that seem to be reliable. Thus, for most clays, Sunneset et al. 1989 and Mayne 2006 provide poor results and should be treated with care.

Correlations, such as in the EN 1997-2 or the one developed by Hutchinson, J. N., 2001 show lower φ´- values close to the surface compared to the equations from Robertson and Campanella (1983) and Kulhawy and Mayne (1990), and approximately the same φ´- value as Kulhawy and Mayne (1990) for greater depths (compare Project: Project 3; Project 4, Project 5, Project 1 (3-8m)).

When comparing φ´- values from correlations, to φ´- values from the geotechnical report, one can conclude that correlations of Robertson and Campanella (1983) and Kulhawy and Mayne (1990) fit best. Another advantage when using those correlations is that no requirements (range of q_c) exist. Nonetheless, care should be taken as those two correlations are just valid for sands.

A trend within these two correlations has been found: The greater the depth, the lower is the φ´- value of Robertson and Campanella (1983) compared to Kulhawy and Mayne (1990). The "transition" occurs in between a depth of 5 to 6m. Both correlations also show good results for silty sands and sandy silts.

A note on the *GEOMIL CPTask*- software: Comparing the φ´- correlations introduced in this paper and results of the *GEOMIL CPTask* software, show a big difference (up to 10°) in the φ´- value (compare Projects: Project 6 and Project 5). On request at *GEOMIL*, the following equation evaluation toll for *GEOMIL CPTask* on the φ´- value was provided[2]:

$$\varphi = ARCTAN(0{,}105 + 0{,}16 * LN(\frac{q_c}{\sigma_{v0}'})$$

The structure of that equation is similar to the correlation by Robertson and Campanella 1983. After examination it shows almost the same results (±0.5°) but with a requirement of q_c> 4MPa. It is noticeable that the φ´- graph in the Output file of the *GEOMIL CPTask* shows about 3-7° higher values than the recalculated results, which is inconsistent.

[2] copy of an entry in the online help of the software, provided by Volker Berhorst, Commercial Director of *GEOMIL*;

2.5.9 Unit weight

Lots of correlations have been developed for estimating the unit weight of soil. Most of them use the total or the effective overburden stress to correlate the unit weight, which would require an iterative process.

One correlation uses the results of the Soil Behavioural Type developed by Robertson (1986) and relates each soil type to a unit weight value.

Table 7: Approximate unit weight corresponding to a SBT Zone;

SBT Zone	Approximate unit weight (kN/m³)
1	17,7
2	12,5
3	17,5
4	18
5	18
6	18
7	18,5
8	19
9	19,5
10	20
11	20,5
12	19

Correlations obtained from q_c, f_s or u_2 will be introduced here (valid for all soil types).

Mayne et al. 2010:

$$\gamma \left(\frac{kN}{m^3}\right) = 11{,}46 + 0{,}33 * \log(z) + 3{,}10 * \log(f_s) + 0{,}7 * \log(q_t)$$

The relationship appears valid for estimates in uncemented geomaterials including a variety of clays, silts, sands, tills, and mixed soil types, however, is apparently not valid for diatomaceous clays and perhaps of limited applicability in highly calcareous soils (Mayne et al 2010).

Mayne et al. 2010 also provides correlations for the shear wave velocity (from q_c and f_s) and also correlation for estimating the unit weight using the shear wave velocity can be found, but won´t be described here.

Another correlation by Robertson 2010:

$$\gamma \left(\frac{kN}{m^3}\right) = \left(0{,}27 * \left[\log\left(R_f\right) + 0{,}36 * \left[\log\left(\frac{q_t}{p_a}\right)\right] + 1{,}236\right) * \gamma_w\right.$$

Other correlations have been developed just for sands or clays, but are not valid for all soils.

Comparisons of correlations for the soil unit weight are conducted in the following projects:

- Project 1 (Figure: 48)
- Project 3 (Figure: 62)
- Project 4 (Figure: 75)

Conclusion on the soil unit weight:

Generally, all correlations differ from each other in a larger extent and no tendencies can be visible. Results scatter in an area of ±3 KN/m³ when comparing with values from the geotechnical report. Laboratory tests on estimating the unit weight give much more reliable results than correlations of CPTs.

Note: The estimation of the correct unit weight is important because lots of correlations (for other geotechnical parameter) depend on the total or effective overburden stress σ_{V0}.

2.5.10 Overconsolidation ratio. OCR

There are big differences in the evaluation of the applicability of correlations concerning the soil type. Robertson et al. 2010 claimed correlations for sands to be "low reliable" and clay as "high applicable".

Generally, there are one correlation for sand and numerous correlations for clay.

Most correlations give the preconsolidation Stress. The OCR can be calculated using the following equation:

$$OCR = \frac{\sigma_p{}'}{\sigma_{v0}{}'}$$

Mayne et al. 2009 provides an equation that is valid for all soils but the user has to change a variable m that depends on the soil type manually. The correlation is the following (Mayne et al. 2009):

$$\sigma_p{}' = 0{,}33 * (q_t - \sigma_{v0})^m * \left(\frac{\sigma_{atm}}{100}\right)^{1-m}$$

$$OCR = \frac{\sigma_p{}'}{\sigma_{v0}{}'}$$

To distinguish between the different soil types, a table of m- values is provided by Mayne et al. 2009:

Table 8: m value according to a soil type;

Soil type	m [-]
Intact clays	1
Organic clays	0,9
Silts	0,85
Silty sands	0,8
Clean sands	0,72
Fissured clay	1,1

Correlations for clay:

Mayne 2005 (after Liao & Bell 2010):

$$\sigma_p{}' = 0{,}60 * (q_t - u_2)$$

Demers& Leroueil, 2002 (after Liao & Bell 2010):

$$\sigma_p{}' = 0{,}33 * (q_t - \sigma_{v0})$$

Note: This is the same equation as Mayne et al, 2009 for clays m=1

Chen & Mayne, 1996 (after Liao & Bell 2010):

$$\sigma_p{}' = 0{,}4 * (u_2 - u_0)$$

Powell et al 1988:

$$OCR = k * \frac{q_t - \sigma_{v0}}{\sigma_{v0}{}'}$$

With k in a range of 0,2-0,5.

Further correlations in clays require other parameters like Plasticity Index IP (Chen & Mayne, 1996), Rigidity Index IR (Mayne, 2005), small strain shear modulus (Mayne 2005), shear wave velocity (Mayne, Robertson & Lunne, 1998 or: Mayne 2006), but are not analysed here.

Correlations for sand:

Depending on the friction angle (φ´ can be accurately calculated for sands), Mayne 2005 developed the only correlation for sand (after Liao& Bell 2010):

$$OCR = \left[\frac{0{,}192 * \left(\frac{q_t}{\sigma_{atm}}\right)^{0{,}22}}{(1 - sin\varphi') * \left(\frac{\sigma_{v0}{}'}{\sigma_{atm}}\right)^{0{,}31}}\right]^{\frac{1}{sin\varphi' - 0{,}27}}$$

Comparisons of correlations for OCR are conducted in the following projects:

- Project 2 (clay) (Figure: 51)
- Project 4 (sands and clay) (Figure: 76)
- Project 1 (sands and silts) (Figure: 47)

Note: The geotechnical report of Project 2 provides an own correlation for estimating OCR that is similar to Chen & Mayne 1996, although requires the Plasticity Index as an Input parameter.

Conclusion on OCR:

In Project 4 all correlations show the same trend except Chen & Mayne, 1996, which is probably not applicable to that clay due to uncertainties of measurements of u_2. In the correlation for Project 2, all correlations show best accordance, even with the own correlation and the laboratory results (with one outlier at the depth of 4m).

Moreover, in sands, consistent results have been achieved, which would question the mentioned evaluation of the applicability: Robertson et al, 2010 evaluated correlations for sands to be "low reliable".

2.5.11 Small strain stiffness modulus

The small strain stiffness modulus represents the elastic stiffness of the soils at shear strains less than 10-4%.

One correlation by Tanaka & Tanaka (1999) modified by Mayne 2006, provides results for all soil types:

$$G_0 \approx 50 * \sigma_{atm} * \left[\frac{q_t - \sigma_{v0}}{\sigma_{atm}}\right]^{m*}$$

Soil type	m*
clean quartz sands	0,6
for silts	0,8
clays of low to medium sensitivity	1,0

The small strain stiffness modulus can also be derived from the Youngs modulus. Elastic theory further states that the small strain Young's modulus, E is linked to G_0, as follows:

$$G_0 = E/(2 * (1 + \vartheta))$$

However, most other correlation distuingish between clay and sand:

Correlations for sands:

Rix and Stokoe, 1992:

$$G_0 = 1634 * \left(q_c * \frac{10^3}{\sqrt{\sigma_{v0}{}'}}\right)^{-0,75} * q_t$$

Schnaid, 2005:

- Uncemented:

$$G_0 = 110 * \sqrt{}(q_c * \sigma_{v0}{}' * p_a)$$

- Cemented:

$$G_0 = 280 * \sqrt{}(q_c * \sigma_{v0}{}' * p_a)$$

Correlations for clays:

Robertson 2009 published a correlation depending on the I_C value from Robertson & Wride, 1998. It is less reliable in the region for fine-grained soils (i.e. when I_C > 2.60) since the sleeve friction f_s and hence F_r, are strongly influenced by soil sensitivity:

$$G_0 = 0,0188 * [10^{(0,55*I_c+1,68]} * (q_t - \sigma_{v0})$$

Other correlations have been developed that require laboratory test result and are worth mentioned:

Mayne & Rix, 1993 (after Schnaid 2006):

$$G_0 = 406 * q_c^{0,695} * e_0^{-1,130}$$

Shybuya et al 1997, (after Schnaid 2006):

$$G_0 = 24000 + (1 + e_0)^{-2,4} * \left(\sigma_v^{\prime}\right)^{0,5}$$

Both are valid for clays and require the void ratio e_0 as additional parameter.

If seismic measurements (down-hole or cross-hole method) are performed, G_0 can be calculated using the following equation (shows best results):

$$G_0 = {v_s}^2 * \rho$$

Comparisons of correlations for G_0 are conducted in the following projects:

- Project 7 (clay) (Figure: 92)
- Project 4 (sands and clay) (Figure: 77)
- Project 2 (sands) (Figure: 50)

Conclusion for G_0

Comparing the correlations, G_0 shows significant differences, when using correlations just derived from CPT input parameters. Hence, correlations additionally using laboratory test results (void ratio) are recommended and give more reliable results. Another possible approach (when no laboratory tests are conducted) is to correlate the void ratio from CPT parameters. NOVOCPT showed a very reliable correlation for e_0, but still, laboratory tests are preferred:

$$e = 1,152 - 0,233 * log(q_{c1}) + 0,043 * log(OCR)$$

However, the best method is to measure the shear have velocity (using SCPT).

Converting the Young´s modulus to G_0, result in low values for G_0 because correlations like Robertson 2009 (for sands) applies for 0,1% stain.

2.6 Project 1

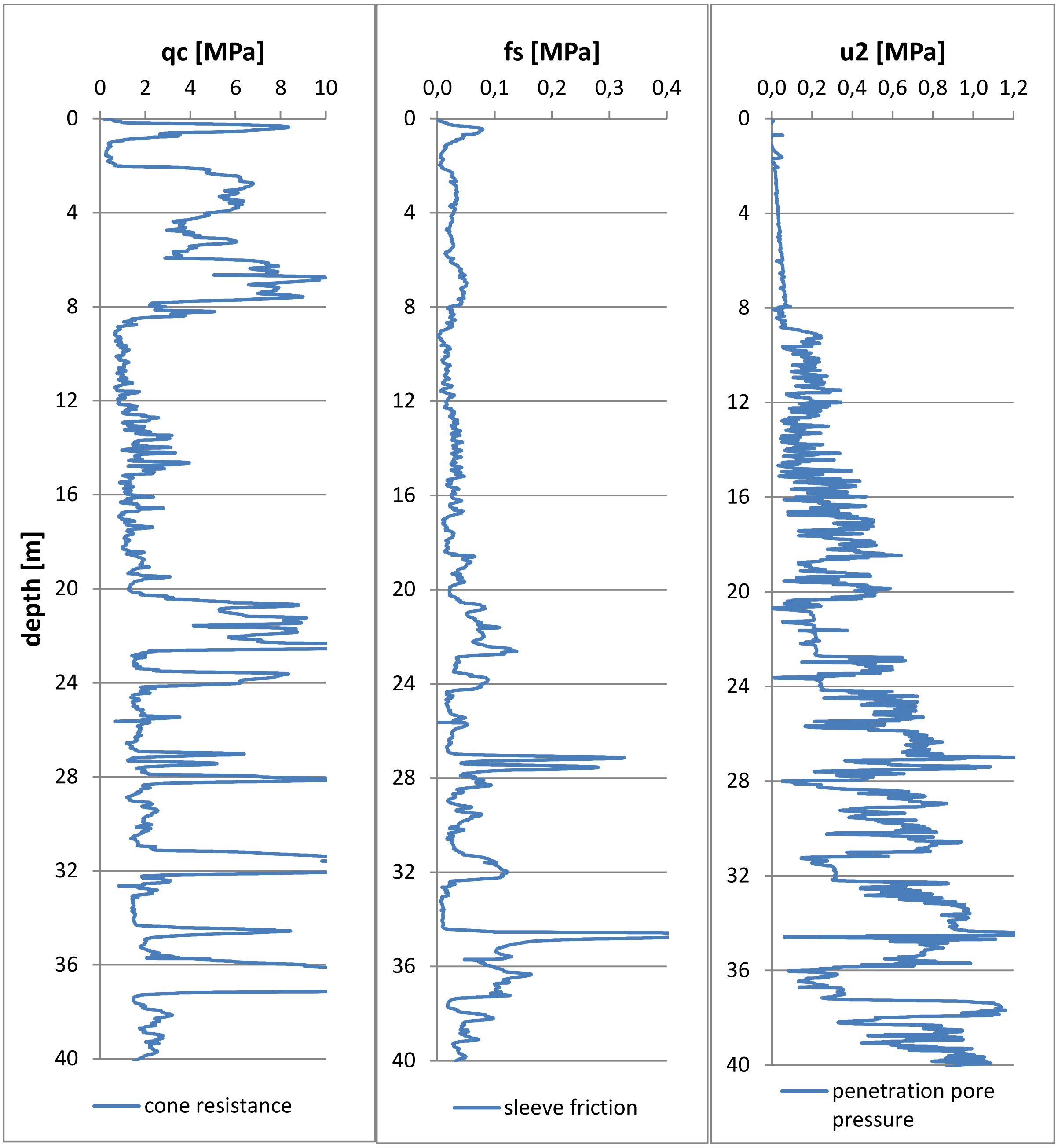


Figure 43: Input parameter of CPTu Project 1;

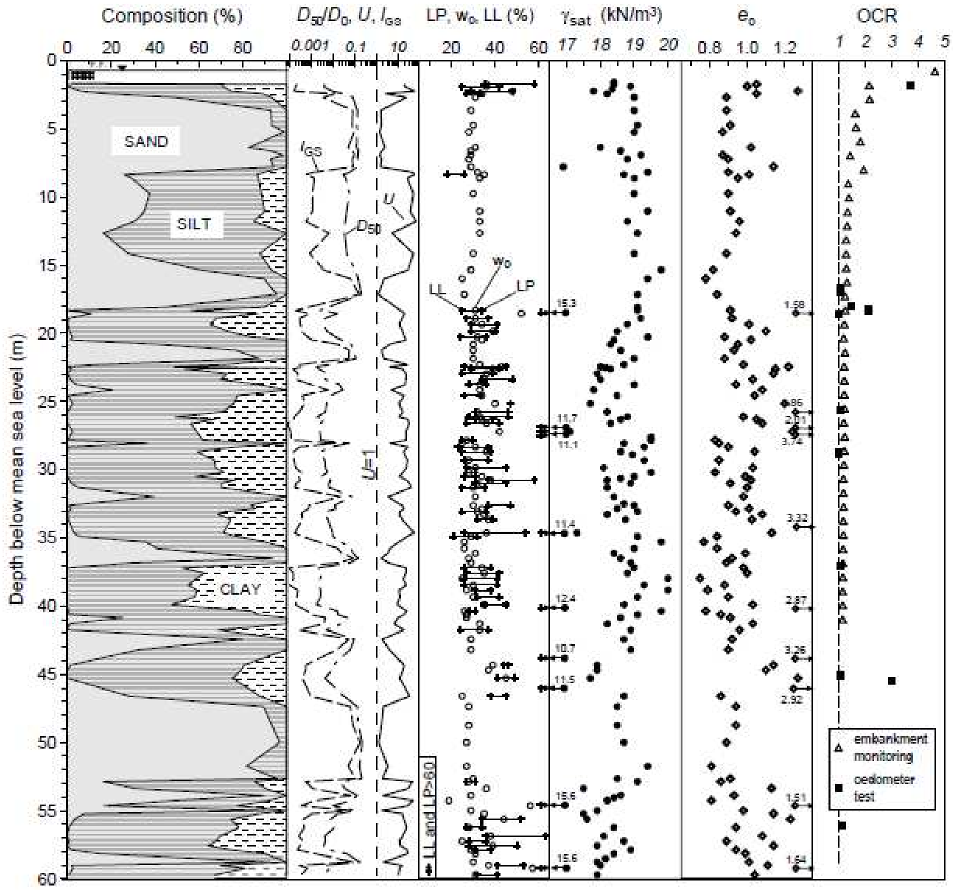


Figure 44: Soil profile, basic properties and stress history at the TT (P. Simonini et al 2006);

Table 9: Results of the geotechnical report (Simonini et al 2006)

Layer	depth	characterisation
A	1-3	soft silty clay
B	**3-8**	**medium-fine silty sand**
C	8-20	clayey silt, with sand lamination between (15 to 18 m)
D	20-22	medium-fine silty sand
E	22-45	alternate layers of clayey and sandy silt
F	45-55	medium-fine silty sand

Frequent laminations of peat are present below 25 m, in layers E and F.

Tests conducted for the evaluation of parameters in the geotechnical report:

- SCPTu
- DMT with dissipation tests,
- laboratory tests
- boreholes

Unfortunately results for the constrained modulus and the effective friction angle have not been provided in that project.

<u>Note on Undrained shear strength:</u> In the Venetian silty clays the process of penetration is partially drained and therefore the interpretation of cone resistance in terms of undrained strength is rather questionable (P. Simonini et al 2006).

2.6.1 Estimation of E_S

For this project no values for the constrained modulus have been published in the geotechnical report. However all correlations are applied and show the same trend. The German Standard (DIN 4094- 1, D6) shows conservative values (α in a range of 2- 3,5 for fine sands and silty sands) whereas the stress- dependant constrained modulus and both correlations from Lunne and Christophersen (1983) show higher but similar results. The user should be aware of Lunne and Christophersen (1983) being just applicable for sands.

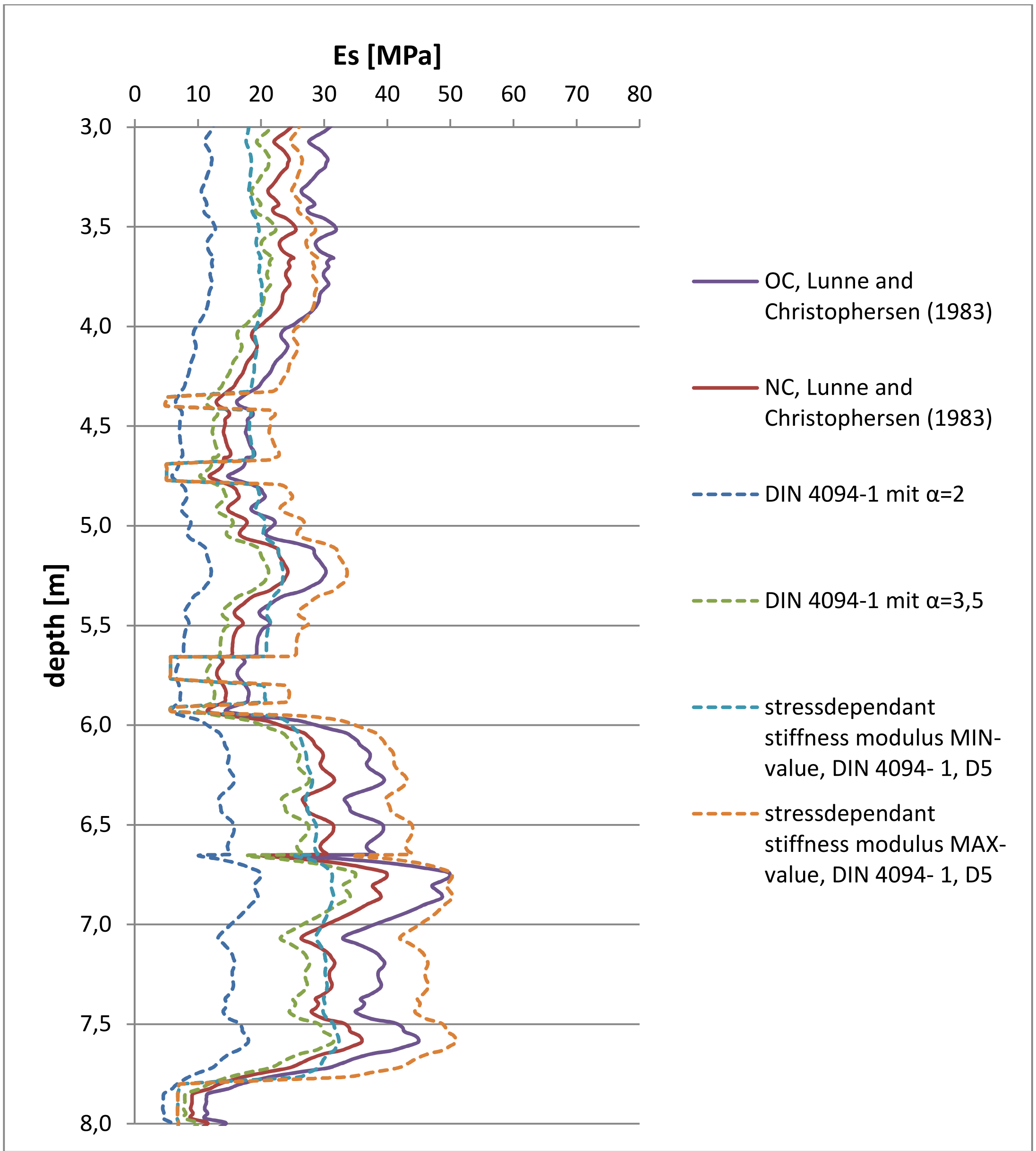


Figure 45: Correlation of Es, project 1;

2.6.2 Estimation of φ´

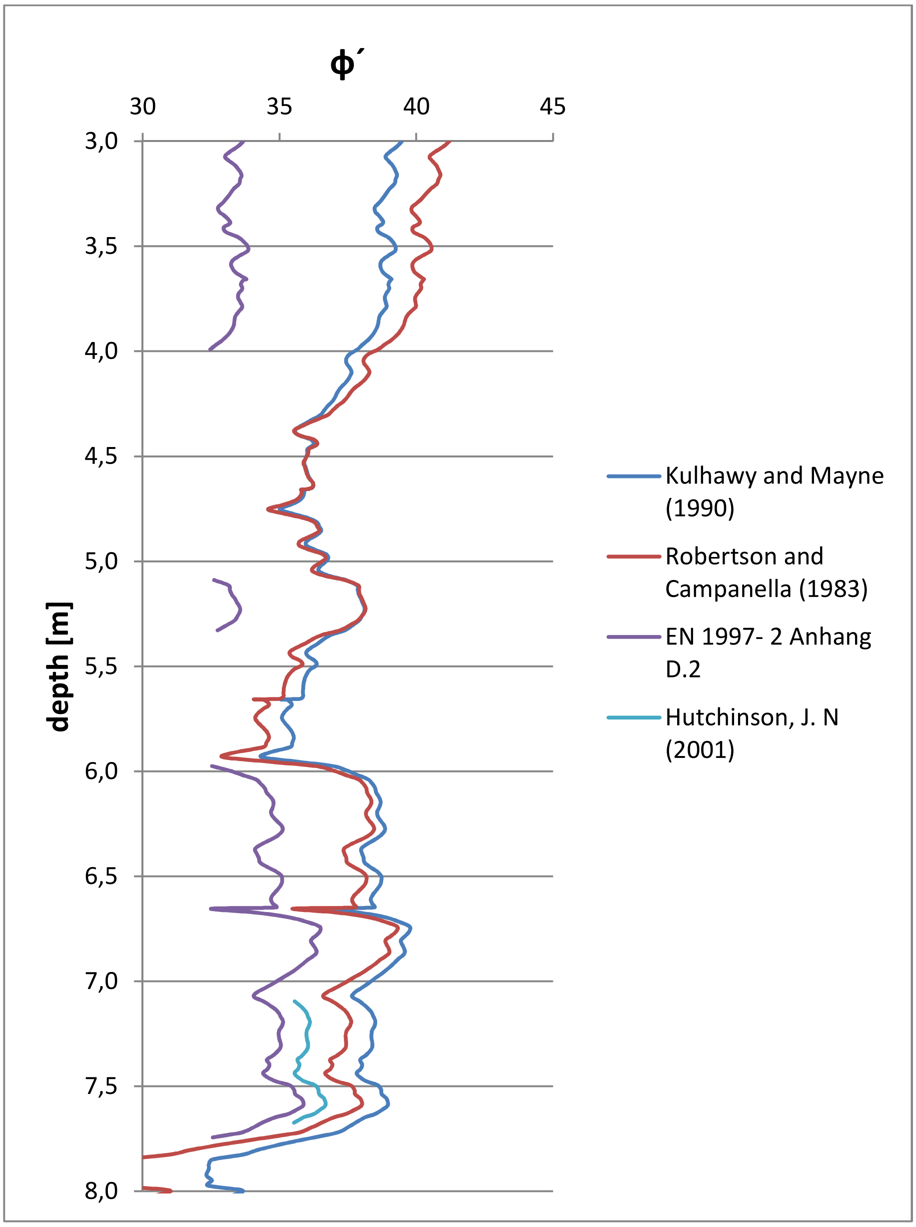


Figure 46: Correlation of φ´;

Note: EN 1997- 2 Anhang D.2 and Hutchinson 2001 require a minimum q_c value.

2.6.3 Estimation of OCR

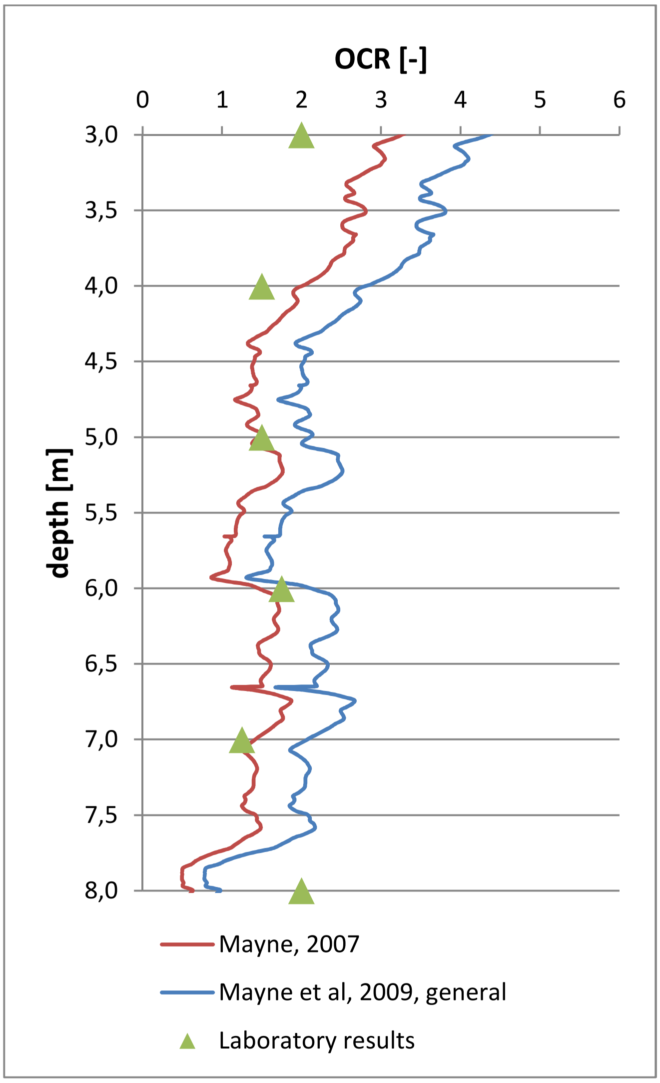


Figure 47: correlations of OCR for sands including laboratory results, (depth: 3-8m);

2.6.4 Estimation of soil unit weight

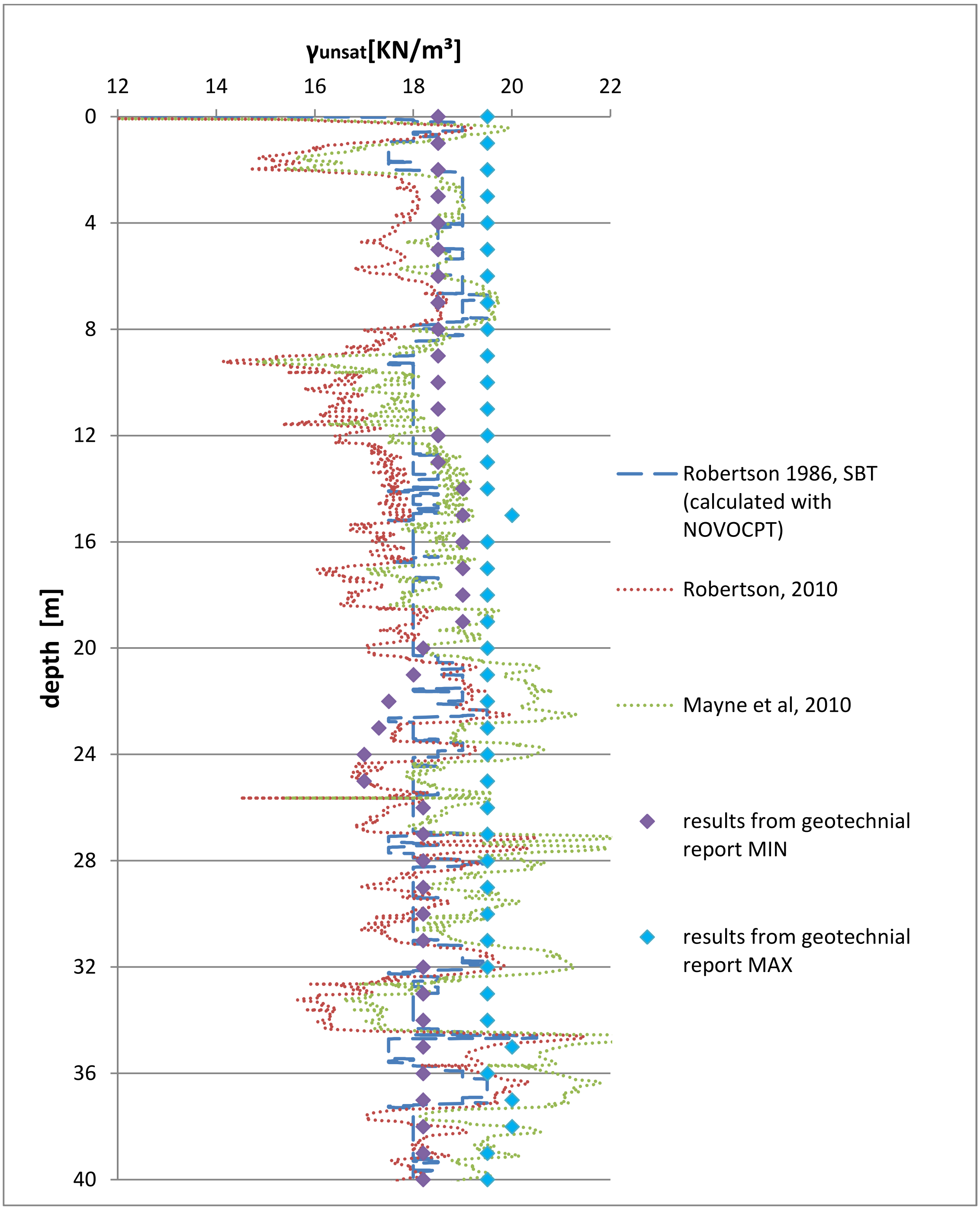


Figure 48: estimation of soil unit weight compared to laboratory tests, project 1;

2.7 Project 2

Note: The only information on CPT parameters available to the author was the cone resistance value (which is sufficient to evaluate the OCR value).

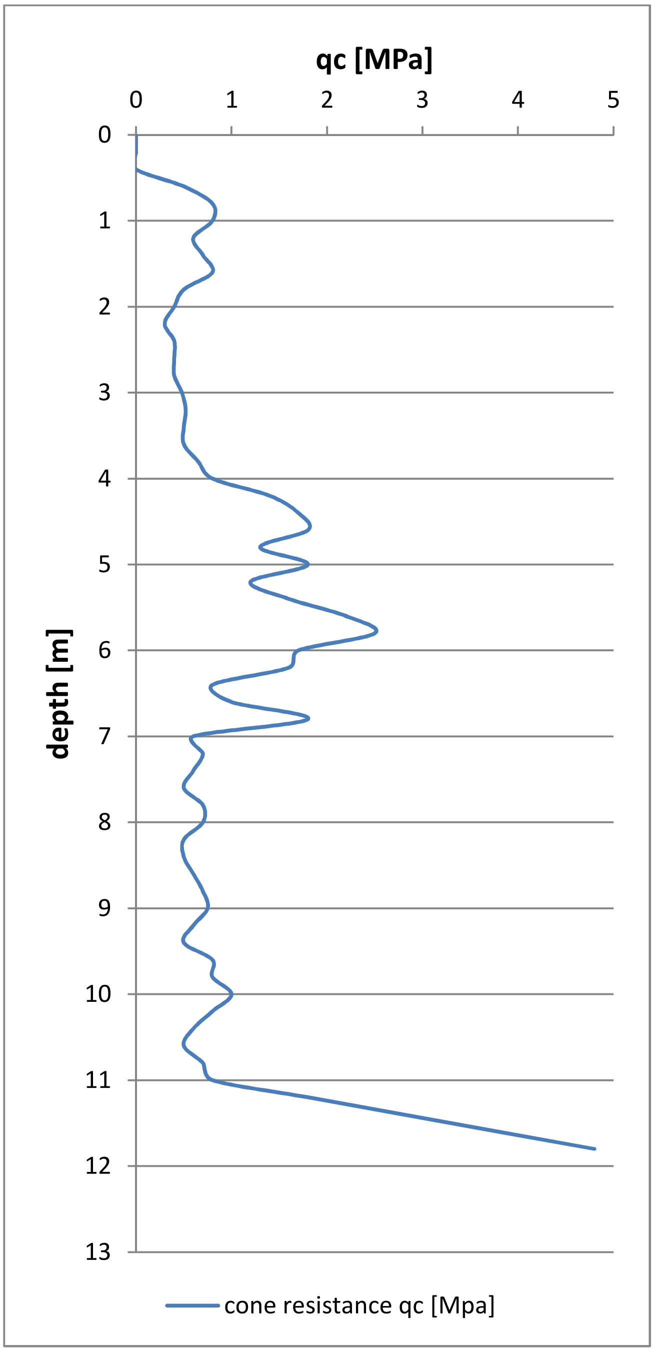


Figure 49: Input parameter for Project 2;

2.7.1 Estimation of G_0

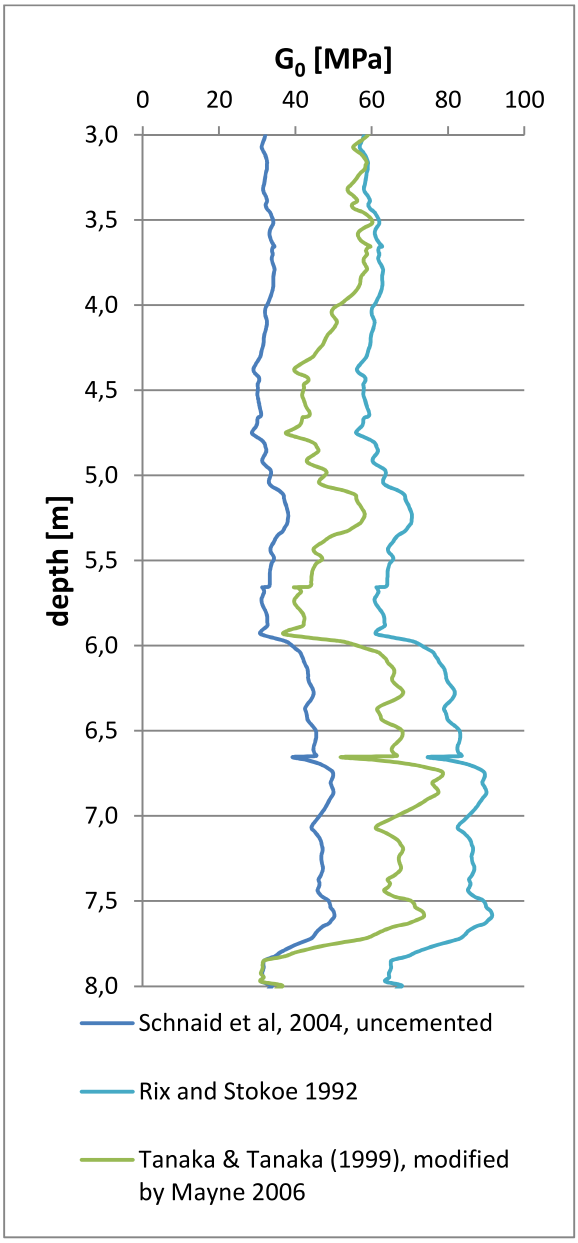


Figure 50: Correlation for G0 in clays, project 2;

2.7.2 Estimation of OCR

Own correlation for OCR for Project 2:

$$\mathrm{OCR} = 0{,}646 * \left[\frac{q_t - \sigma_{v0}}{\sigma_{v0}}\right]^{1{,}01} * \mathrm{IP}^{-0{,}21}$$

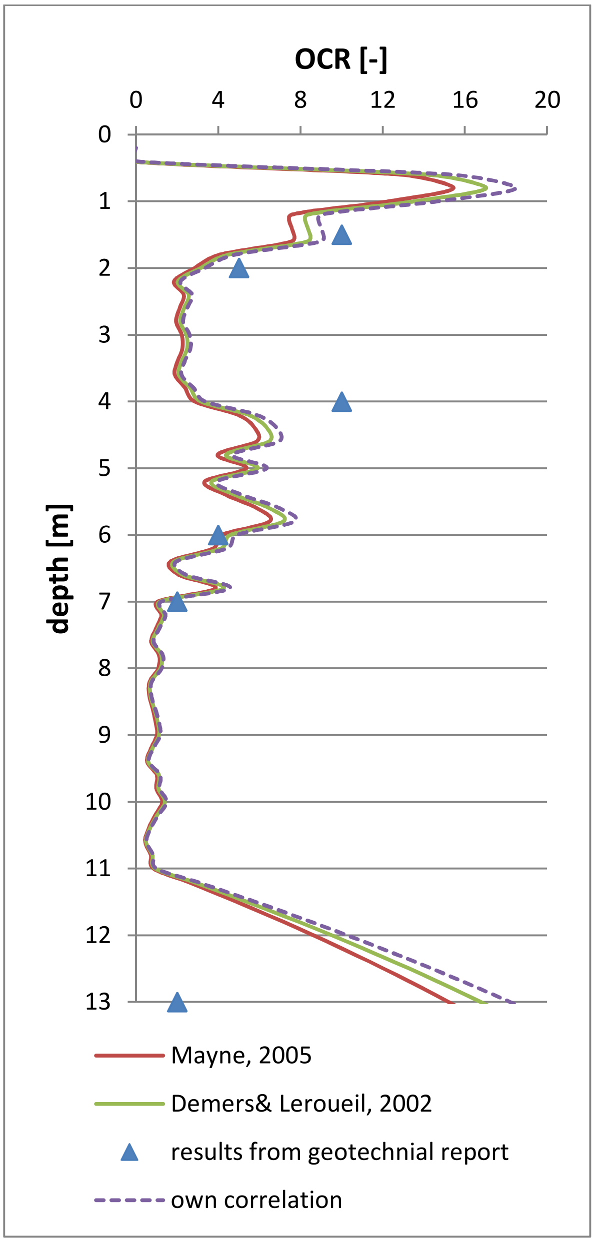


Figure 51: Correlation for OCR in clays;

Note: The results from the geotechnical report probably refer the results from the “own” CPT correlation and laboratory tests.

2.8 Project 3

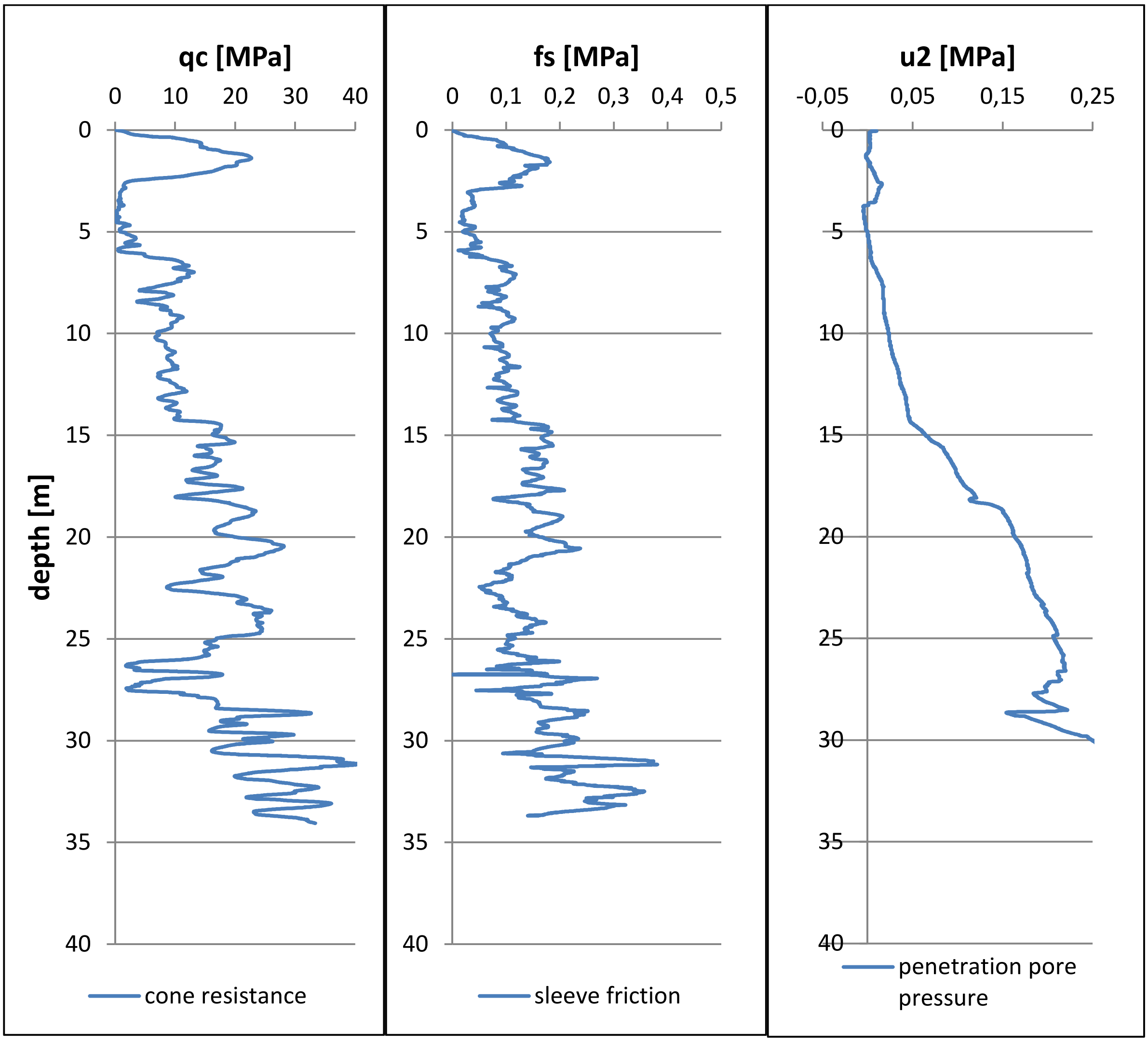
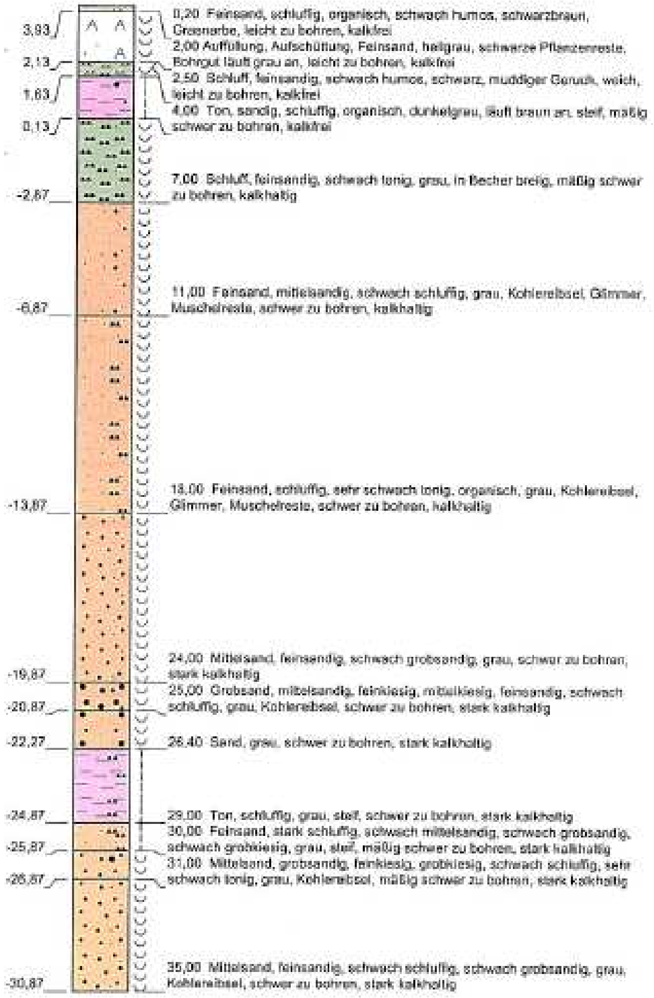


Figure 52: Input parameter for Project 3;



Bodenart	charakteristische Bodenkennwerte					
	Wichte feucht γ_k kN/m³	Wichte Auftrieb γ'_k kN/m³	Reibungswinkel φ_k °	Kohäsion c'_k kN/m²	Kohäsion $c_{u,k}$ kN/m²	Steifemodul (Erstbelastung) E_{0k} MN/m²
Auffüllung sandig	19	11	37,5	--	--	50 - 75
Klei (Schluff, Sand, Ton, organisch)	15-19	3-8	20	10-15	30-55*	1,5 – 3,5 √z
Torf	11-12	1-2	15-20	2-5	15	0,5 - 1
Sand, locker gelagert	17	9-10	30	--	--	7,5 – 12,5 √z
Sand, mittel-dicht gelagert	18	10,5	32,5	--	--	15 – 25 √z
Sand, dicht gelagert	19	11	37,5	--	--	20 – 30 √z

z = Tiefe in m unter Gelände

* Einzelproben für die Kohäsion c_u wurden auch mit ca. 200 kN/m² gemessen.

Figure 53: Left: borehole BK right: Table of geotechnical parameters for further calculations (compare geotechnical report)

For the CPT, which is located next to the borehole BK the following stratification is valid:

Note on the determination of the parameters in the geotechnical report: Basically all parameters have been obtained using laboratory tests.

Table 10: stratification and geotechnical parameter

Test conducted		Triaxial test	Ödometric test	Triaxial test	
Layer	**Depth [m]**	**φ´ [°]**	**Es [MPa]***	**c_u [KPA]**	**Unit weight [kN/m³]**
Backfill/ sandy	0-2,5	37,5	50÷75	0	19
Silty Clay	2,7- 7	10-15	1,5÷3,5*√z	30-55**	15-19
Fine sand	7- 11	30	7,5÷12,5*√z	---	17
Medium sand	11- 26	32,5	15÷25*√z	---	18
clay	26- 28	10- 15	1,5÷3,5*√z	30-55**	15- 19
Dense sand	28- 35	37,5	20÷30*√z	---	19

*The constrained modulus is given for initial loading (50 to 100KPa). The modulus for unloading-reloading can be estimated as the threefold value of the given (initial loading) value.

**Some specimen showed c_u - values higher than 200KPa.

Laboratory results: The grain size distribution shows coefficients of uniformity in a range of 2,3 to 4,3 (depth from 11- 33m). I_P that has also been tested from the borehole aside and was determined to be ~20 (compare: geotechnical report).
Results for the undrained shear strength and the effective friction angle have been obtained using triaxial test (according to the DIN 18137).

Note on undrained shear strength: When analysing the laboratory results, the undrained shear strength value was never below 50 KN/m² and half of the specimen showed values of about 200KN/m².

2.8.1 Estimation of c_u (depth from 2,5 to ~6m, clayey- silty)

C_u - values obtained from laboratory test vary from 50- 200KPa, whereas in the table of parameters in the geotechnical report that is provided for further calculations, c_u is described to be 30-55KPa, which may be a very conservative approximation. That is also proved when determining I_R (with diagram proposed by Keaveny and Mitchell 1986, compare Figure 54) from I_P (from laboratory tests) and the OCR (compare CPT correlations for OCR), and using I_R for further calculations of N_{kt} referring on theoretical solutions, for example: studies by Baligh or Vesic (compare Figure 56).

When back-calculating the N_{kt} value from a c_u value of 30KPa, very unrealistic results appear for N_{kt} (values in a range of 20 up to 120, compare Figure 54)

Estimation using the total cone resistance:

When investigating empirical solutions like Aas et al 1986 with the range from 8-16 for the N_{kt}- value, one can see that solutions are very similar to theoretical studies performed by Baligh (1975) and Vesic (1975) and fit well to laboratory solutions. Two different values for I_R (180 and 80, to account for the varying OCR, ~1-6) have been considered for calculation of the N_{kt} and then c_u. Only small differences have been encountered due to the varying I_R value (within one correlation).

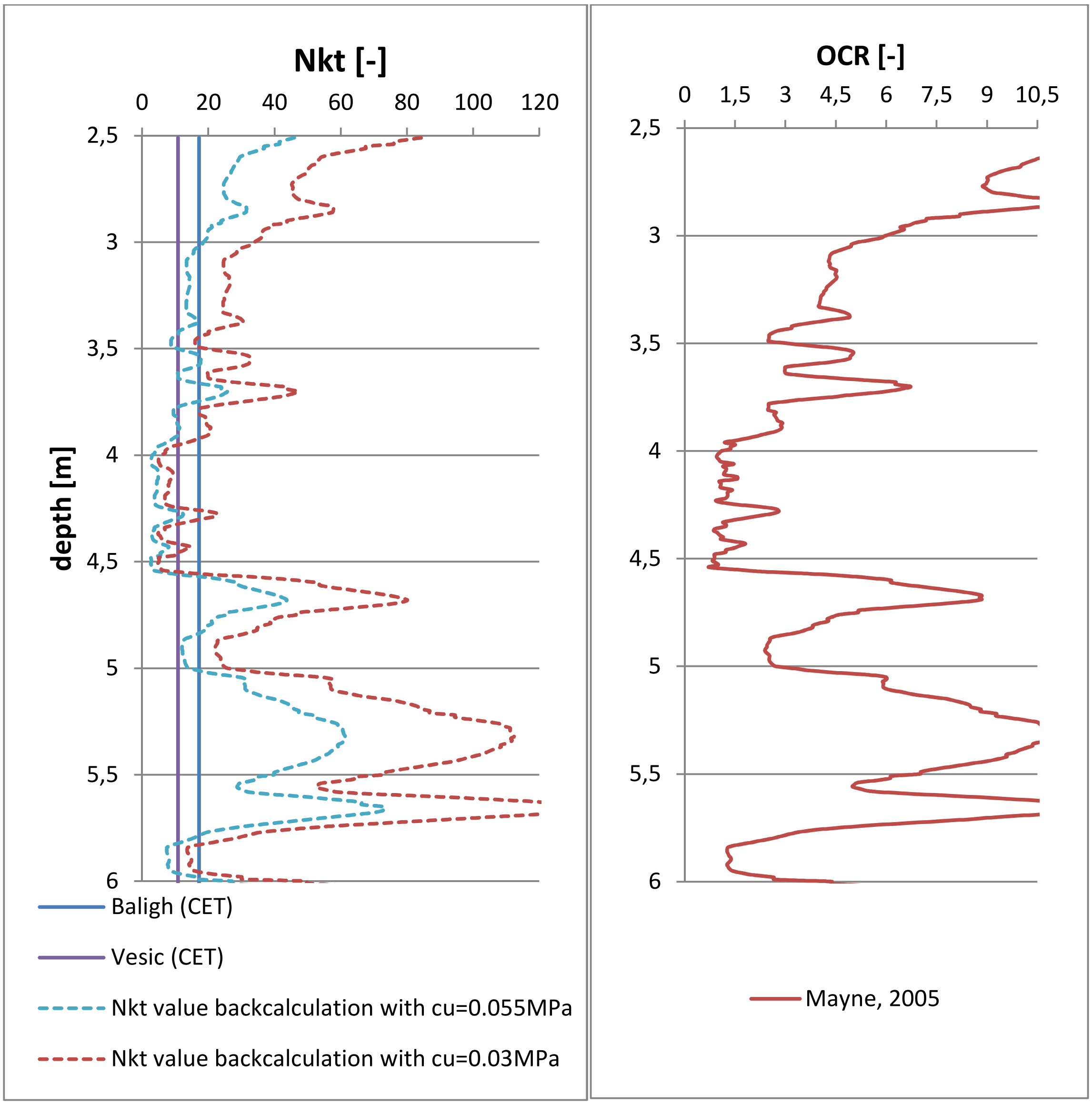


Figure 54: left: Nkt values from Baligh and Vesic compared to back-calculated c_u results; right: correlation from Mayne 2005 for OCR in clays;

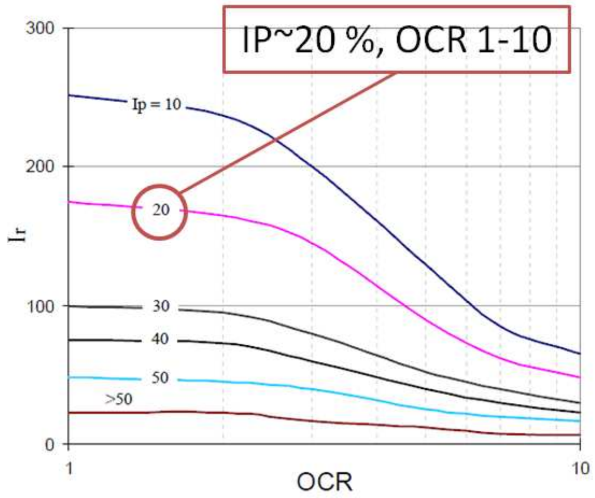


Figure 55: Estimation of IR, with OCR from 1-10 and IP~20 after Keaveny and Mitchell 1986; range of IR= 80- 180

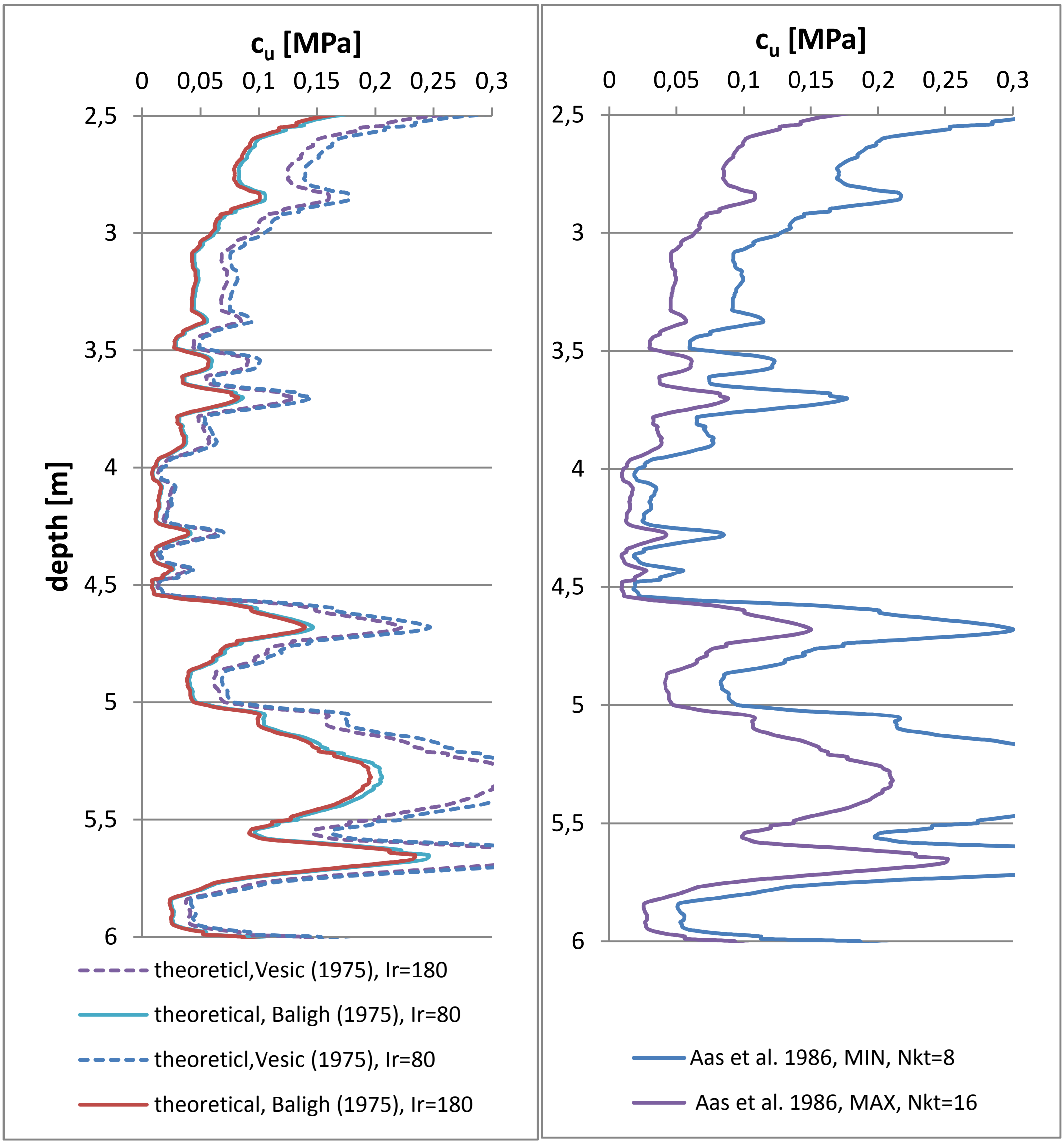


Figure 56: Results for c_u values;

Estimation using the effective cone resistance:

Results using suggested N_{ke} values from Senneset et al. 1982 show the same tendency, but higher than empirical and theoretical solutions (from the total cone resistance).

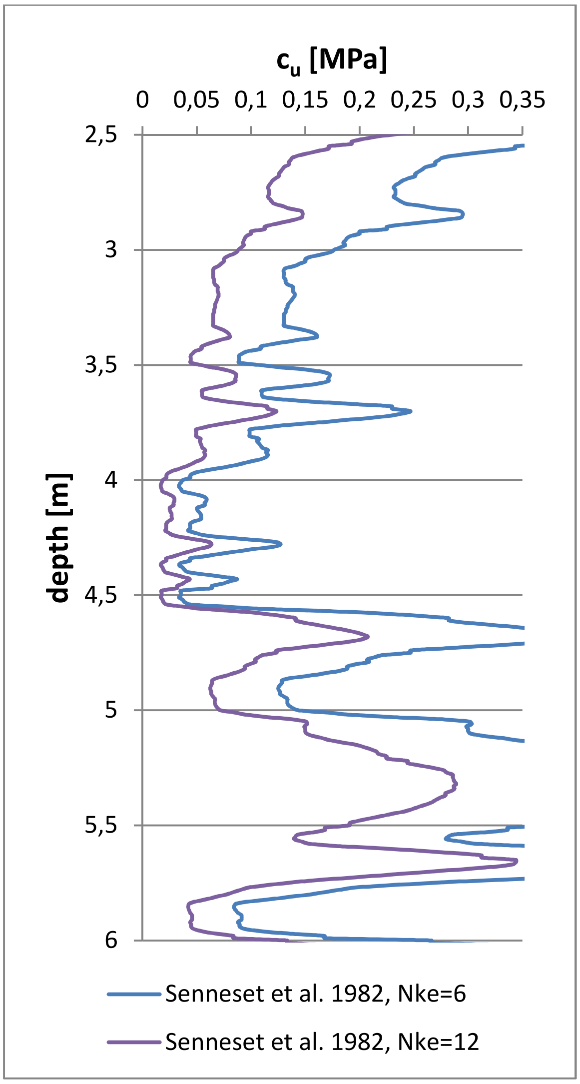


Figure 57: Range of c_u, using the effective cone resistance;

2.8.2 Estimation of Es:

According to the geotechnical report, one dimensional compression tests have been conducted to evaluate to constrained modulus. When comparing laboratory results with other correlations from CPT/u results, one can see that the estimation of the stress- dependant constrained modulus fits best to the value mentioned in the geotechnical report. The mean constrained values that are introduced in the DIN 4094- 1, D6 and the correlation provided by Lunne and Christophersen (1983) show conservative estimations (compare Figure 58).

Table 11: table of α- values used in DIN 4094-1, D-6;

Layer	α_{MIN}	α_{MAX}
Backfill	5	6
kley	2	3.5
Fine sand	2	3.5
Medium sand	3.5	5
Dense sand	5	

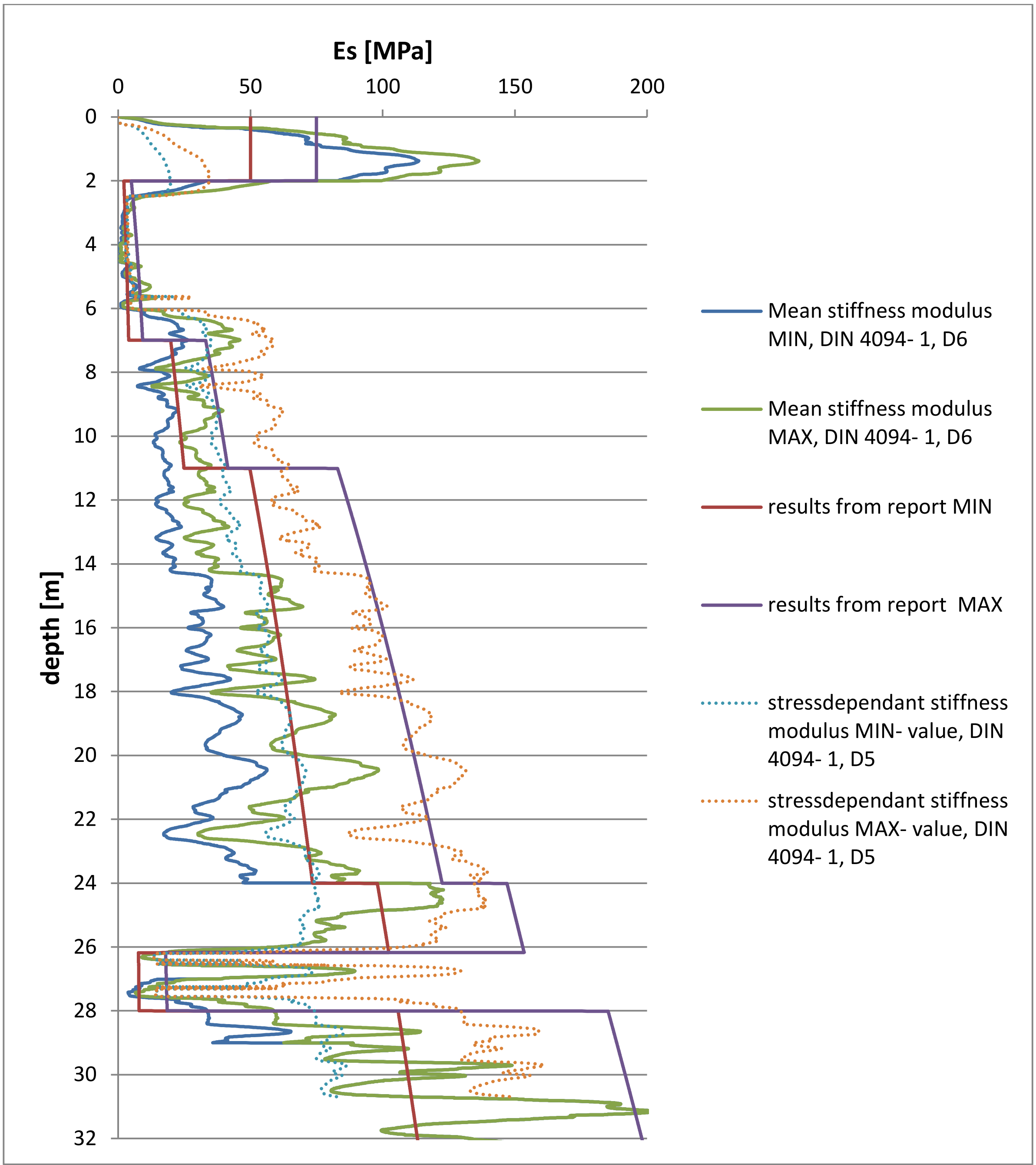


Figure 58: Comparison of correlations for Es;

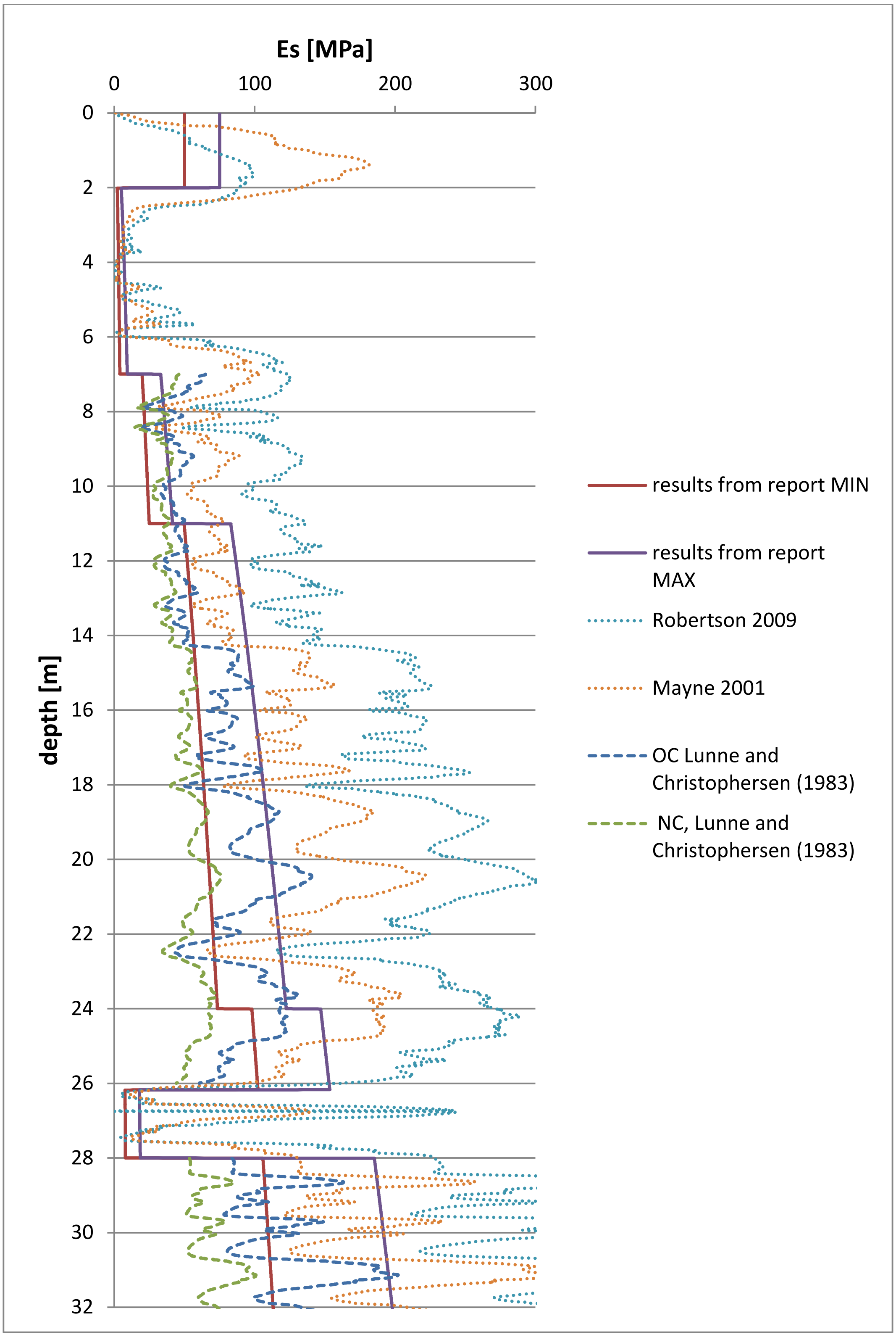


Figure 59: Comparison of correlations for Es, with Lunne and Christoperson not applied in kley or clayey soils;

2.8.3 Estimation of φ´

The first diagram shows the two most used correlations for sands combined with the one for clay (sand: Robertson and Campanella (1983) and Kulhawy and Mayne (1990), clay: Sunneset et al. 1989), and comparison with the results from geotechnical report are presented. The correlations for sands result in higher values than shown in the geotechnical report, which may be due to a conservative estimation of the effective friction angle in the geotechnical report (those values are provided for further calculations. Correlations for the clayey part (depth of 3- 6m and 26 to 28m) show lower values.

Because of its requirement, correlations like Hutchinson, J. N (2001) and the one recommended by EN 1997- 2, D.2 can just be applied in certain areas (here e. g.: depth 8- 26m). They show similar results compared to the correlation from Kulhawy and Mayne (1990).

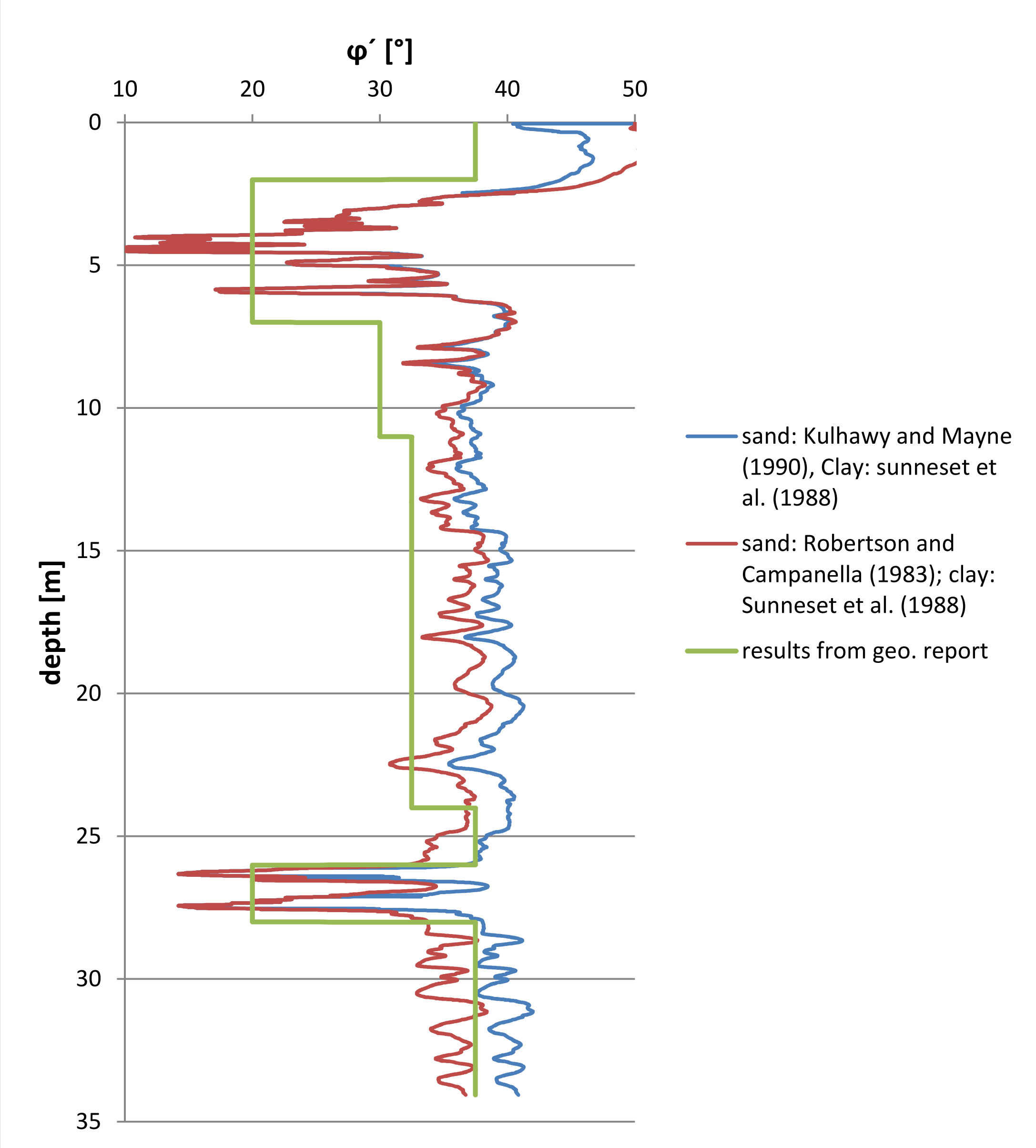


Figure 60: comparison of correlation for φ´;

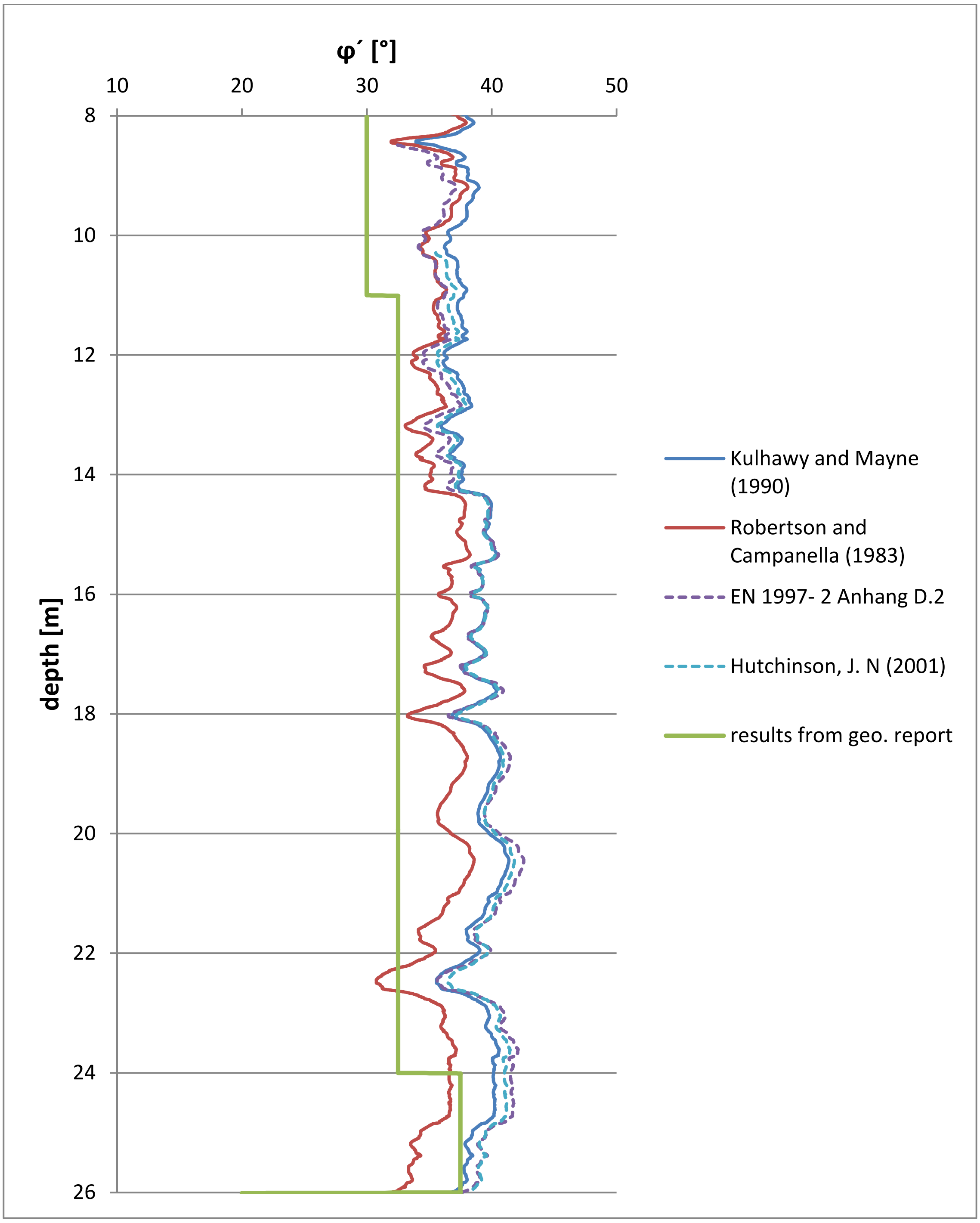


Figure 61: further correlations for φ;

2.8.4 Estimation of the soil unit weight

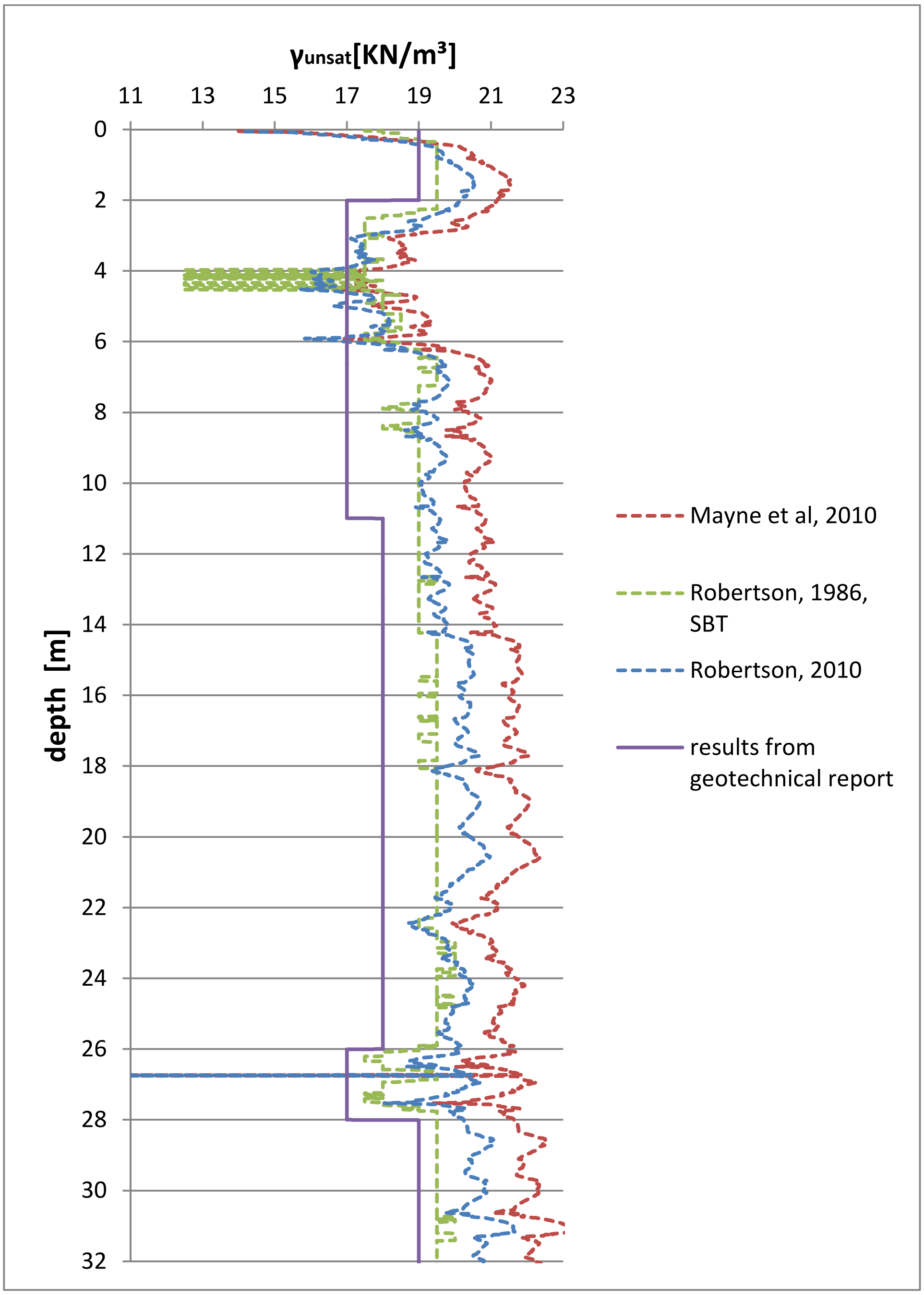


Figure 62: correlations for soil unit weight;

2.8.5 Estimation of the Young´s Modulus

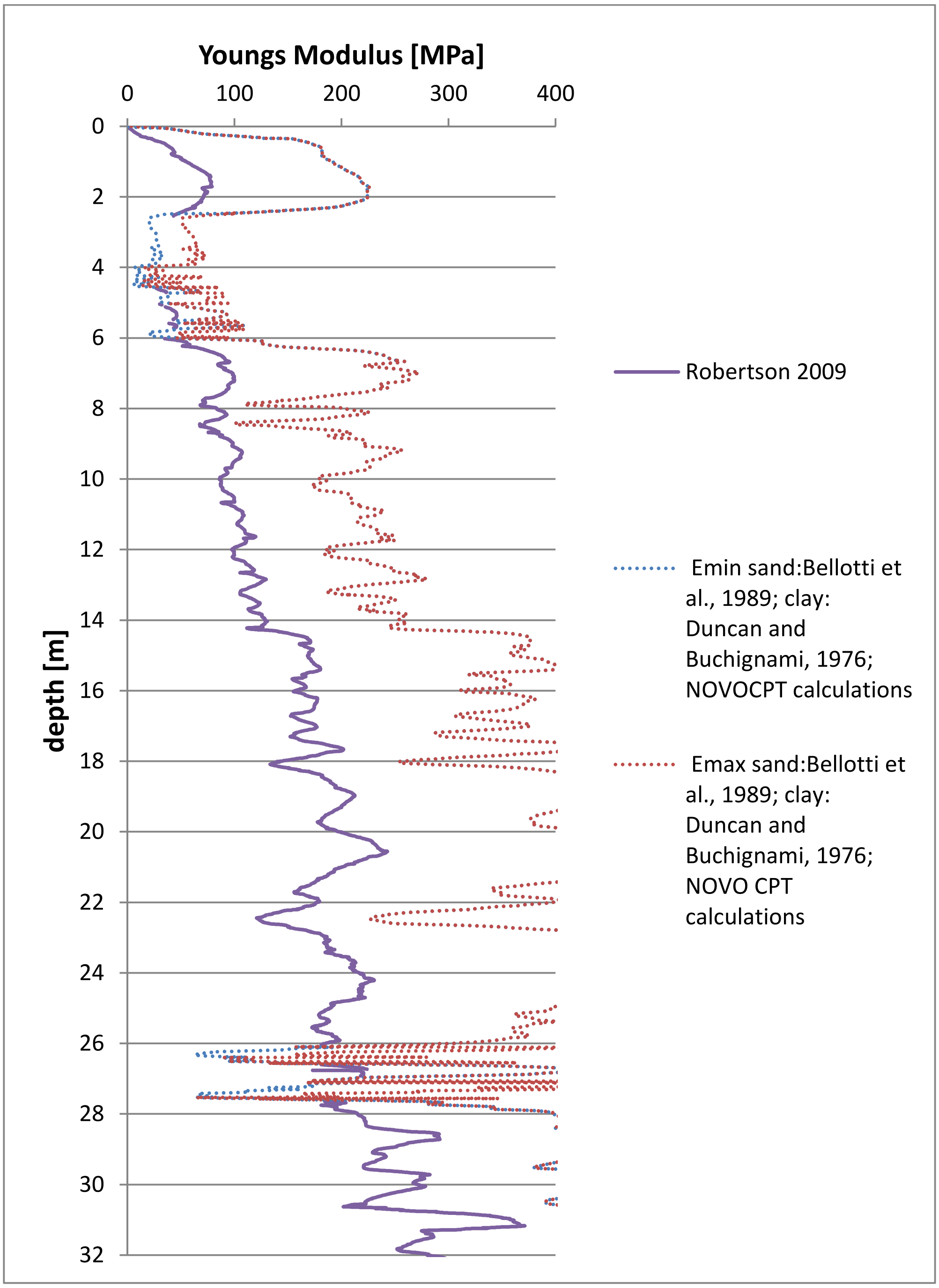


Figure 63: correlations for Young´s modulus;

2.8.6 Estimation of soil type

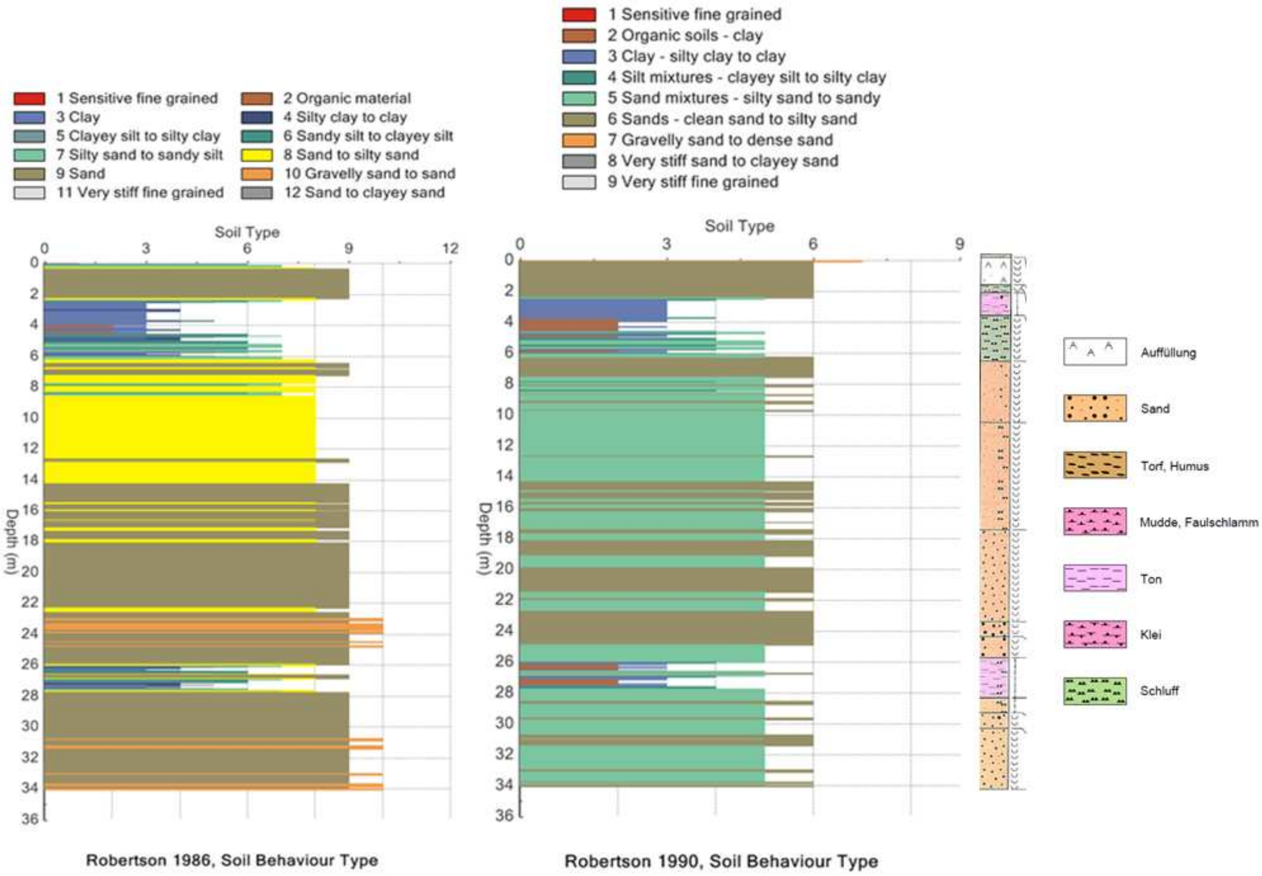


Figure 64: Soil type evaluation comparison of Robertson 1986 and 1990 and borehole (right);

2.9 PROJEKT 4

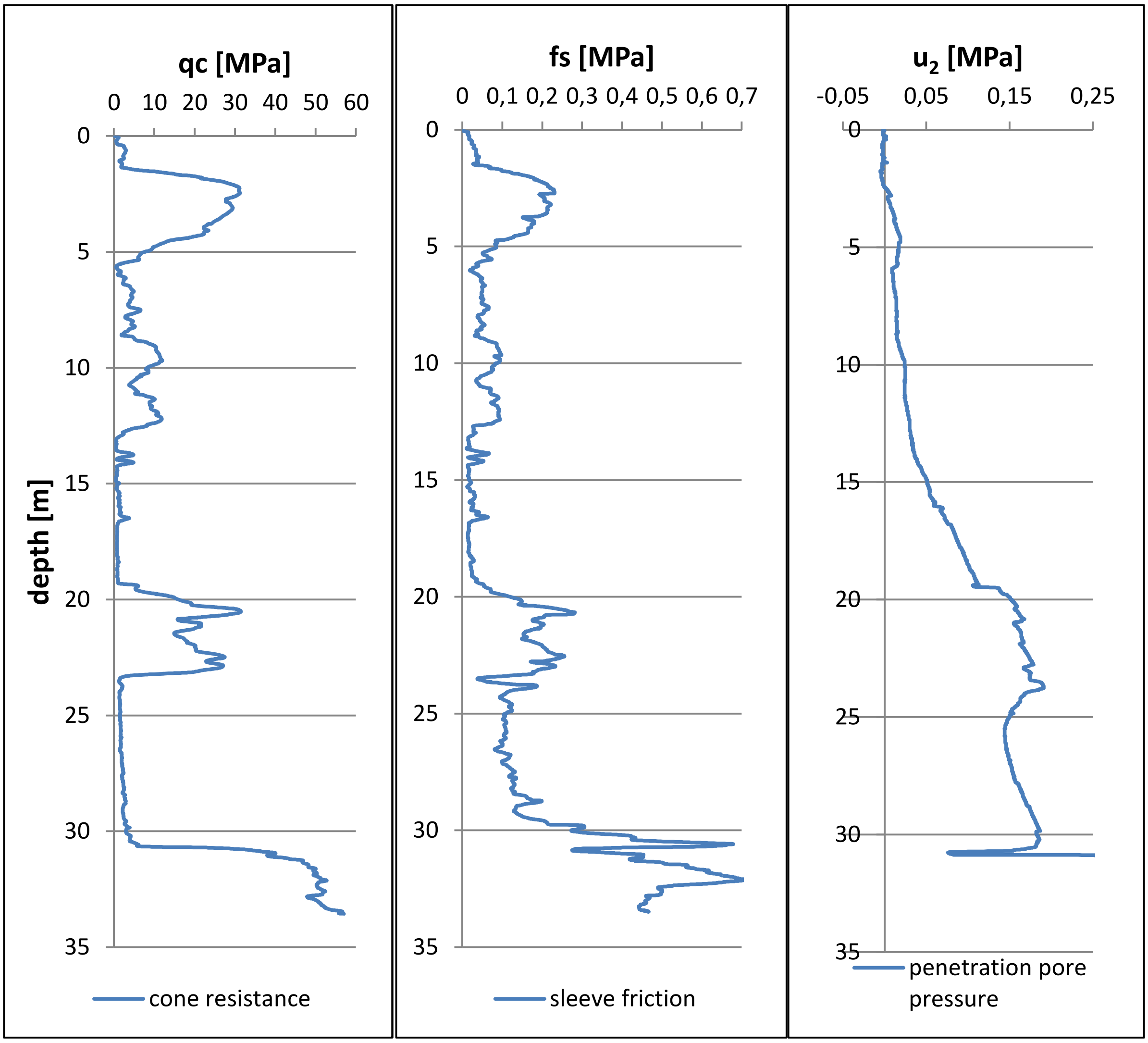


Figure 65: CPTu Input parameter from Project 4;

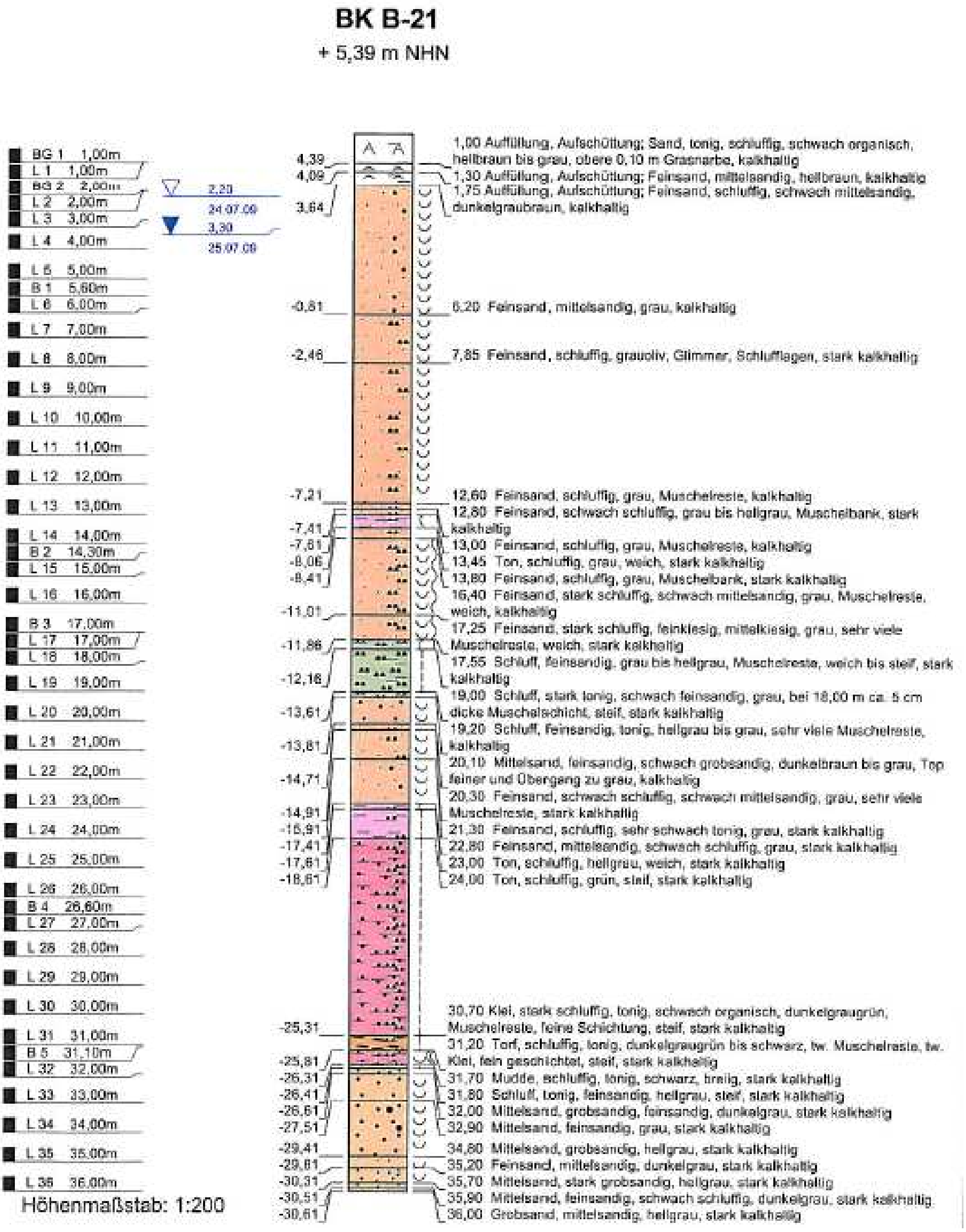


Figure 66: borehole that is located next to the CPTu ;

Table 12: layers, description and geotechnical parameters (compare: geotechnical report)

Test conducted			Triaxial test	Ödometric test	Triaxial test
Layer	**Depth [m]**	**Unit weight [KN/m³]**	**φ´ [°]**	**Es [MPa]**	**c_u [KPa]**
Backfill	0-2	19	37,5	50÷75	0
Medium Sand	2- 5,5	18	32,5	15÷25*√z	---
Fine sand	5,5- 13	17	30	7,5÷12,5*√z	---
Clay	13- 14	15-19	20	1,5÷3,5*√z	30- 55**
Fine sand/ Silt	14- 16,7	17	25	1,5÷3,5*√z	30- 55**
silt	16,7- 19,3	15-19	20	1,5÷3,5*√z	30- 55**
Fine to medium Sand	19- 23,5	18	32,5	7,5÷12,5*√z	---
clay	23,5- 24	15-19	20	1,5÷3,5*√z	30- 55**
kley	24- 32	15-19	20	1,5÷3,5*√z	30- 55**
Medium Sand	32- 36	18	32,5	15÷25*√z	---

*The constrained modulus is given for initial loading. The modulus for unloading- reloading can be estimated as the threefold value of the given (initial loading) value.

**Some specimen showed c_u - values higher than 200KPa.

Basically, all parameters obtains are results of laboratory testing.

Further Laboratory test results:

The grain size distribution show values for the coefficient of uniformity in a range of 1,7 and 5,2. The plasticity index IP gives a value of 43,9% (at a depth of 26,6m) (compare: Geotechnical report).

2.9.1 Estimation of Es:

Except in the first 2m, the correlation estimating the stress- dependant constrained modulus (DIN 4094-1, D-5), again fits best to the values from the geotechnical report. As in Project 3, the mean constrained modulus show conservative values (except in the depth of 1,5 to 5m).

Table 13: table of α- values used for estimation of the mean constrained modulus (DIN 4094-1, D-6);

Layer	α_{MIN}	α_{MAX}
Backfill	5	6
kley	2	3.5
Fine sand	2	3.5
Medium sand	3.5	5
Dense sand	5	5
Clay	2	6

silt	1	6

Other correlations like Robertson (2009), Mayne (2001) again show higher values, that may occur because of a high Ic value.

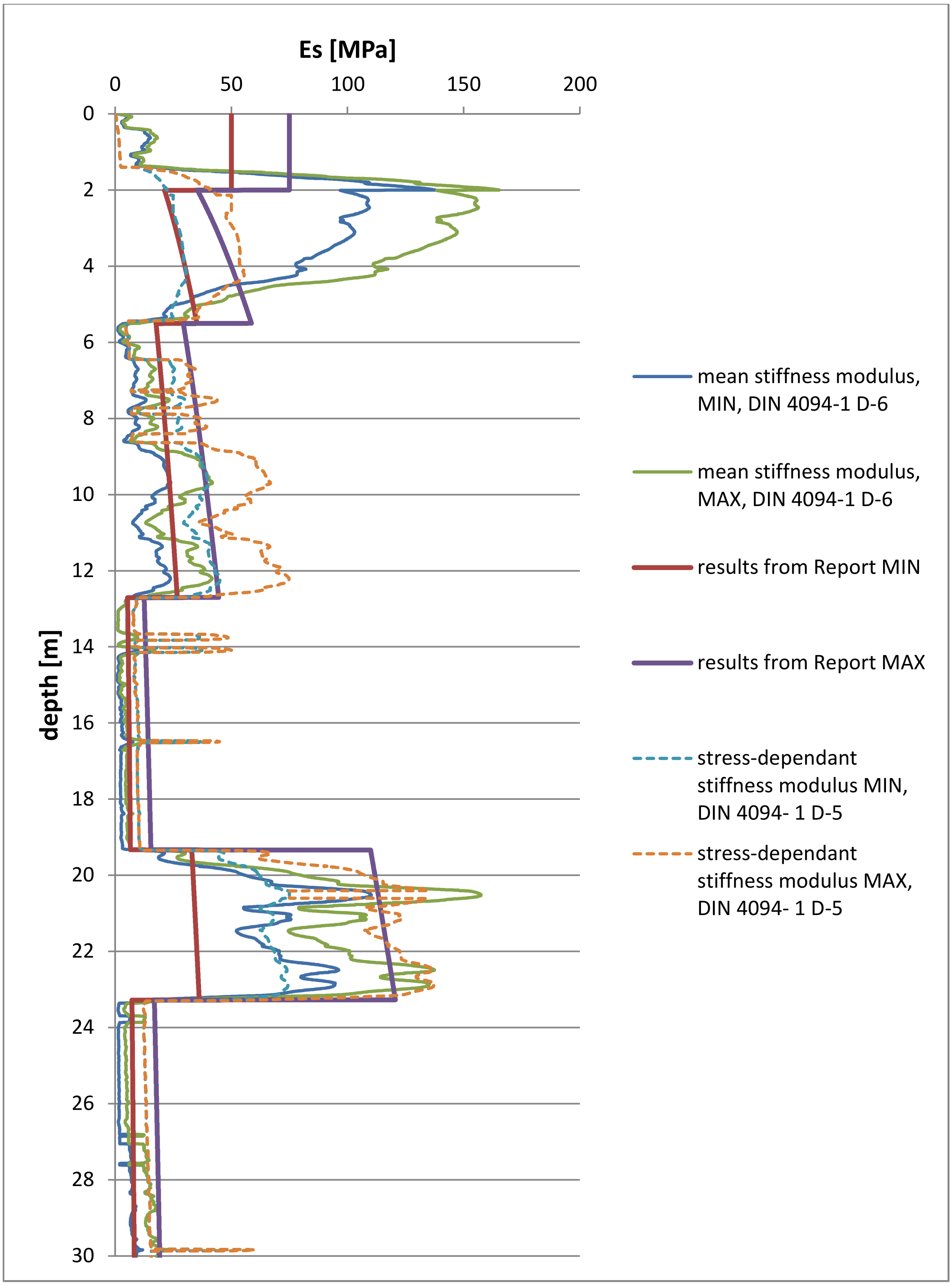


Figure 67: comparisons of correlations for Es, project 4;

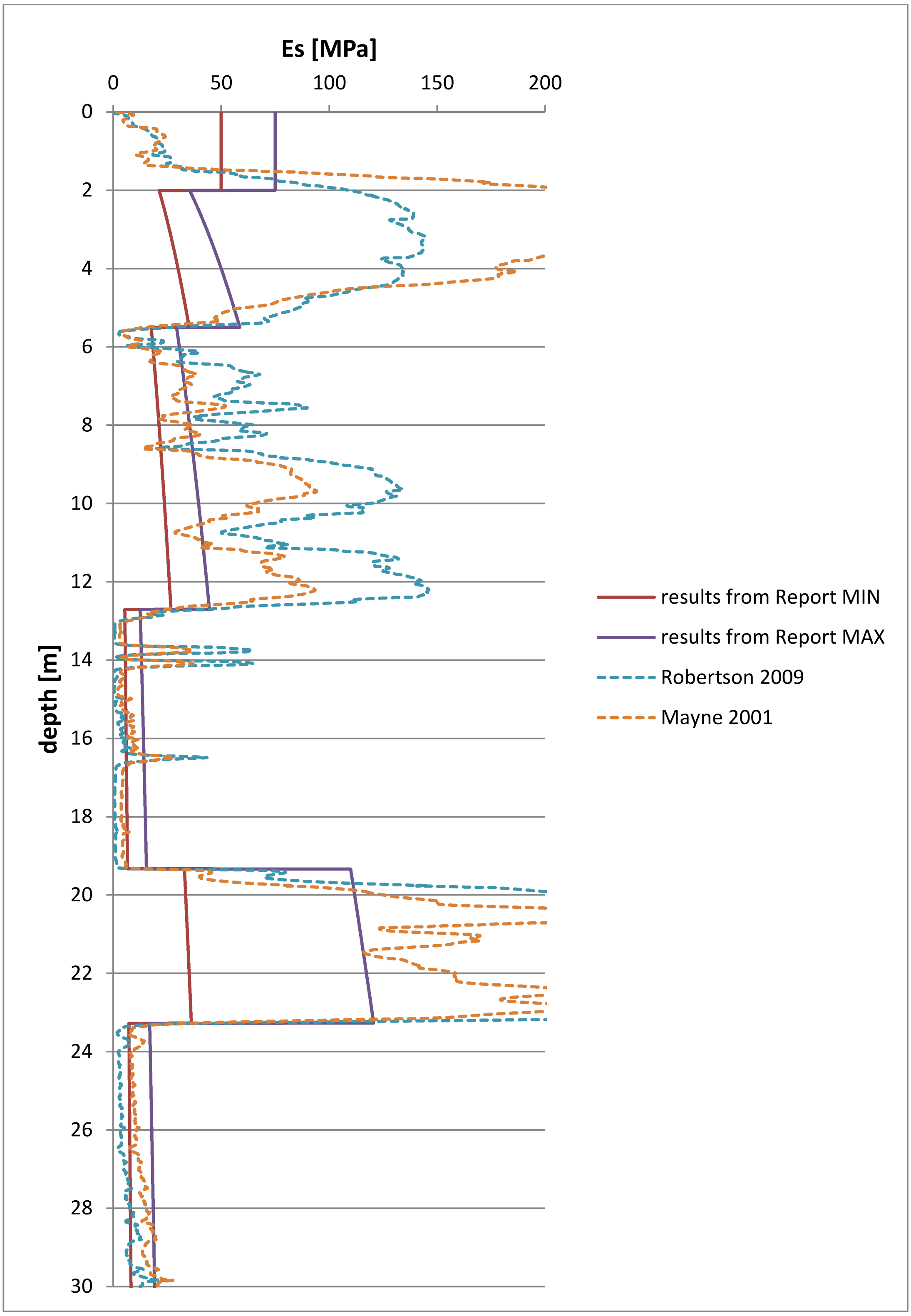


Figure 68: more correlations on Es;

2.9.2 Estimation of c_u in a depth between 23 and 31m

C_u was evaluated from laboratory test and results have been obtained between 50KPA and 220KPa for the kley layer (mainly consisting of clay),.

For the **total cone resistance**, the theoretical approach of Vesic 1975 and Baligh 1975 was applied for estimating N_{kt} and the approach of Aas et al 1985 (N_{kt} in a range of 8 and 16) represents the numerous empirical solutions available.

When estimating I_R from the diagram developed from Keaveny and Mitchell 1996 by using IP and the OCR one can determine N_{kt} by using the relationship found by Vesic 1975 or Baligh 1975.

In this project, I_P in a depth of 26m has a value of 43 [%], and OCR is a range of 1-2 (compare Powell et al 1988 Figure 69). So after applying Keaveny and Mitchells´ (1996) approach, I_R was determined to be ~80.

The table below shows the results of the N_{kt} value from Vesic 1975, Baligh 1975 (I_R= 80) and also includes the empirical approach from Aas et al 1985.

Table 14: comparisons of Nkt values

Author	**equation**	**N_{kt} [-]**
Vesic (1975)	$Nkt = 3{,}9 + 1{,}33 * \ln(I_R)$	9,728
Baligh (1975)	$Nkt = 12 + \ln(I_R)$	16,38
Aas et al (1985)	*empirical*	8- 16

The results obtained by the theoretical and empirical approach show good accordance with the laboratory tests for c_u in that layer (between 50 and 220KPa).

The range N_{ke} values suggested by Senneset et al 1982 will be applied for the **effective cone resistance**.

The results of the effective cone resistance are significantly higher compared to the total cone resistance.

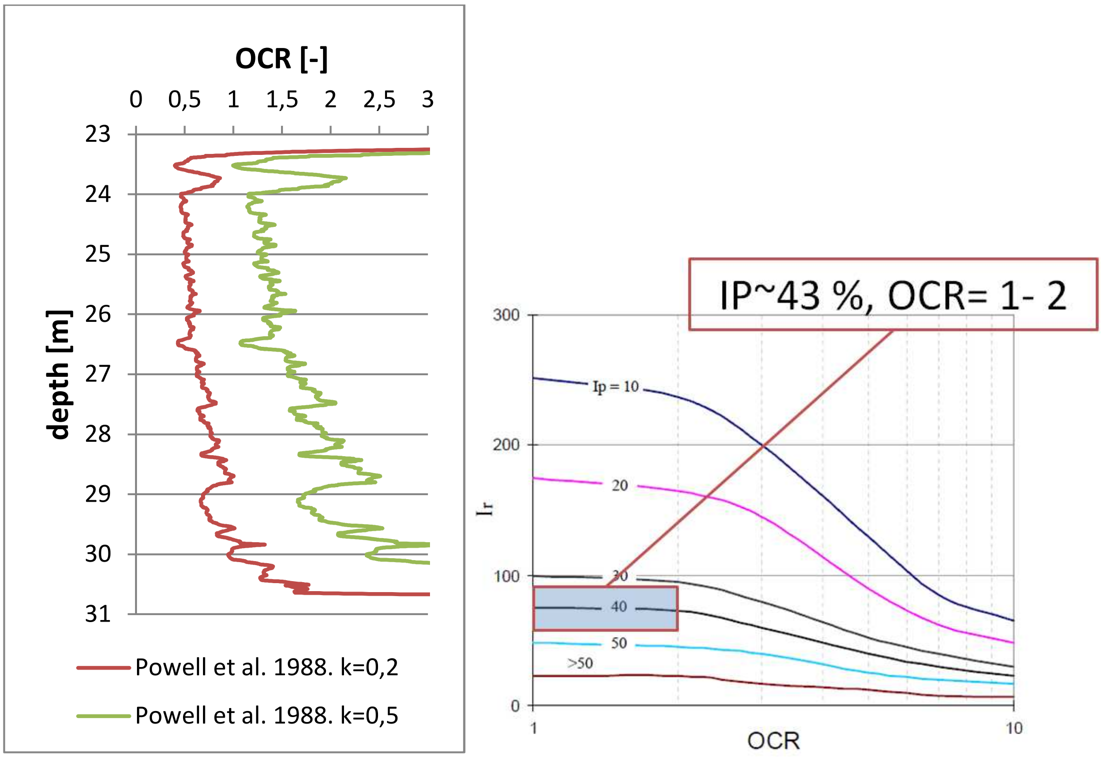


Figure 69: left: approximation of OCR value that is used for the determination of Ir after Keaveny and Mitchells (1996); data: Project 4;

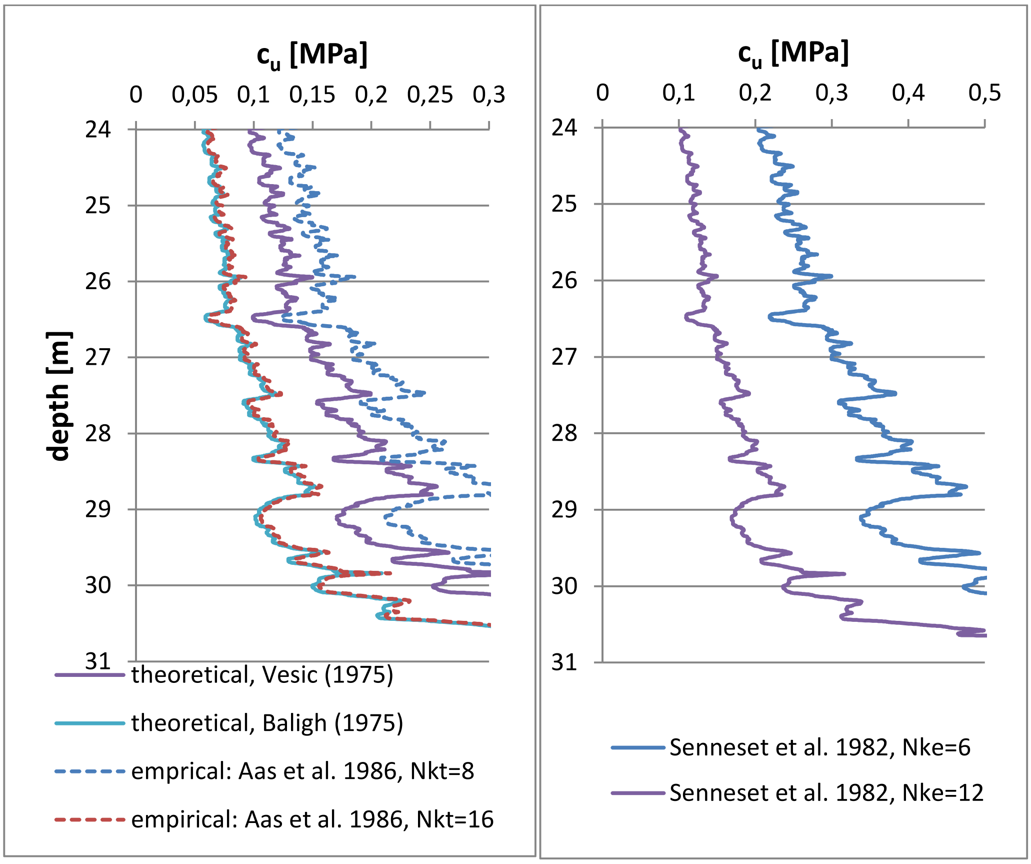


Figure 70: left: comparisons of c_u values calculated using the total cone resistance; right: calculations using the effective cone resistance;

2.9.3 Estimation of φ´

Correlations like Robertson and Campanella (1983) and Kulhawy and Mayne (1990) show good results in sands. The values are slightly higher than recommended in the geotechnical report.

Close to the ground surface Robertson and Campanella (1983) show higher φ´- values than Kulhawy and Mayne (1990), but with greater depths, φ´- values from Robertson and Campanella (1983) get less.

In that project another correlation for clay (Mayne, 2006- NTH solution, that is a simplification of Sunneset et al. 1988) is applied that doesn´t provide values in a certain range (basically the whole clay area), which is the main difference within this two correlations for clay.

In Figure 72, other correlations that require a minimum value of q_c are examined in sands. In the first few meters correlations by Hutchinson, J. N (2001) and EN 1997- 2, D.2 show lower values than Robertson and Campanella (1983) and Kulhawy and Mayne (1990) but reach a higher value with greater depths.

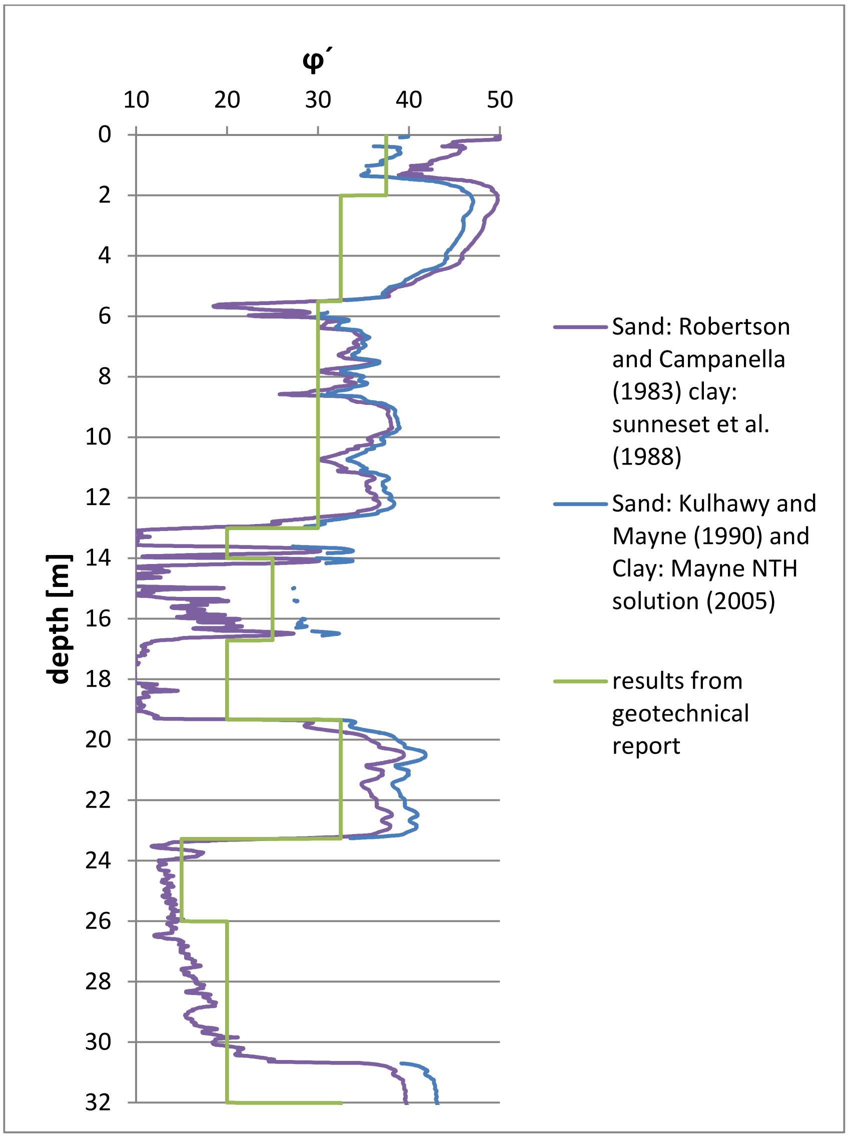


Figure 71: comparisons of φ´ correlations;

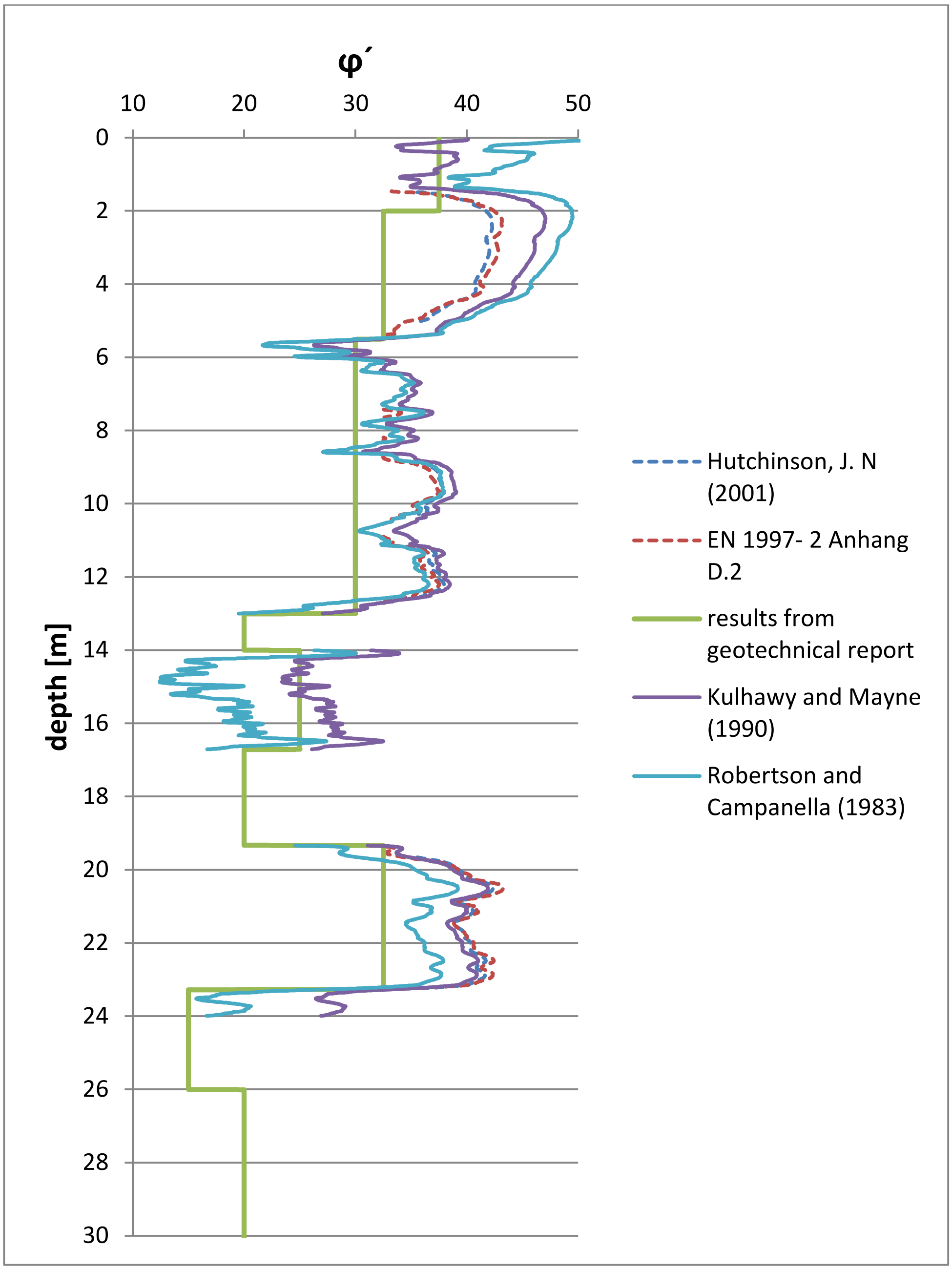


Figure 72: more correlations on φ´;

2.9.4 Estimation of the Young´s Modulus

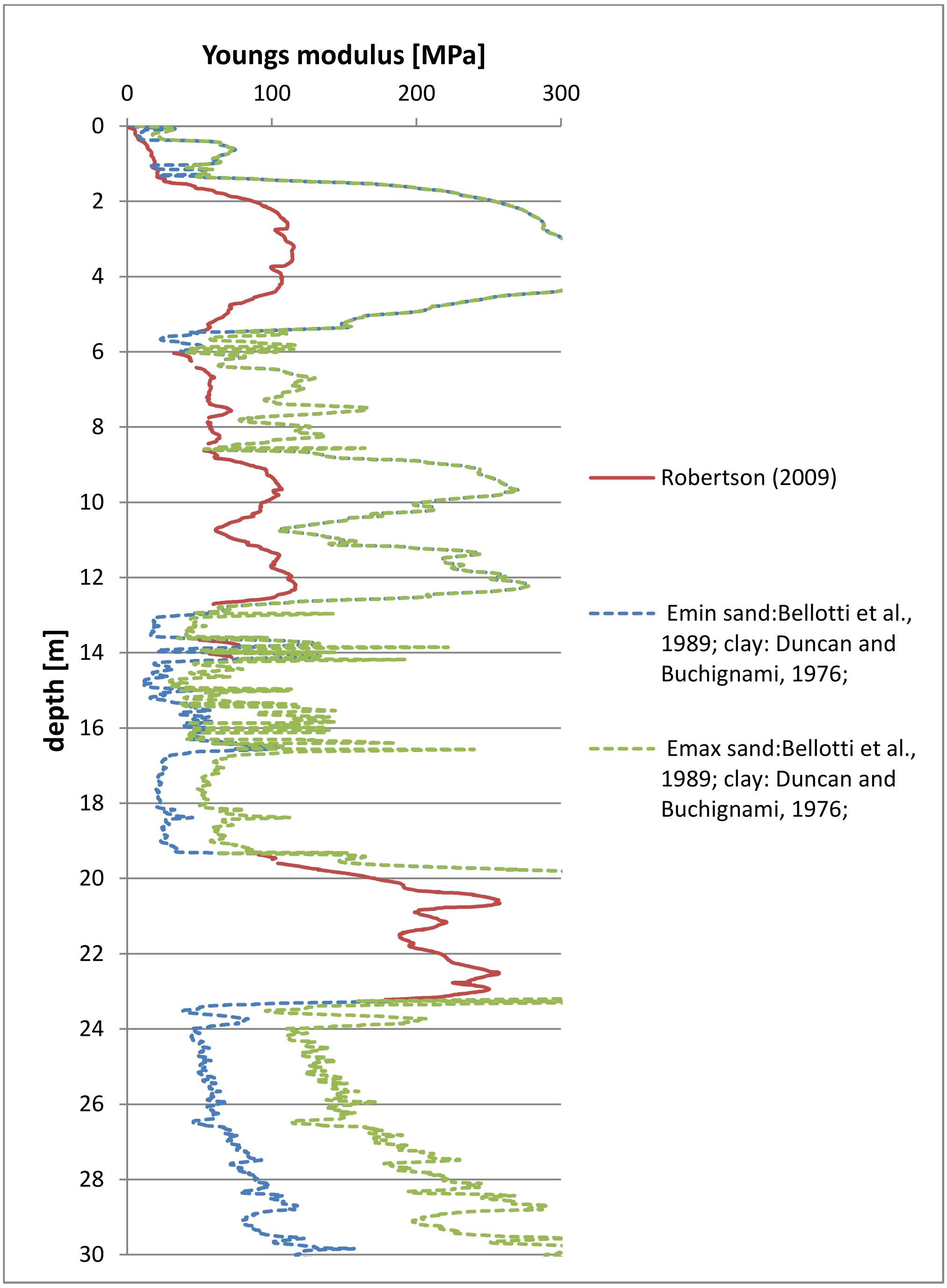


Figure 73: correlations for Youngs modulus at project 4;

2.9.5 Constrained modulus derived from Young´s modulus

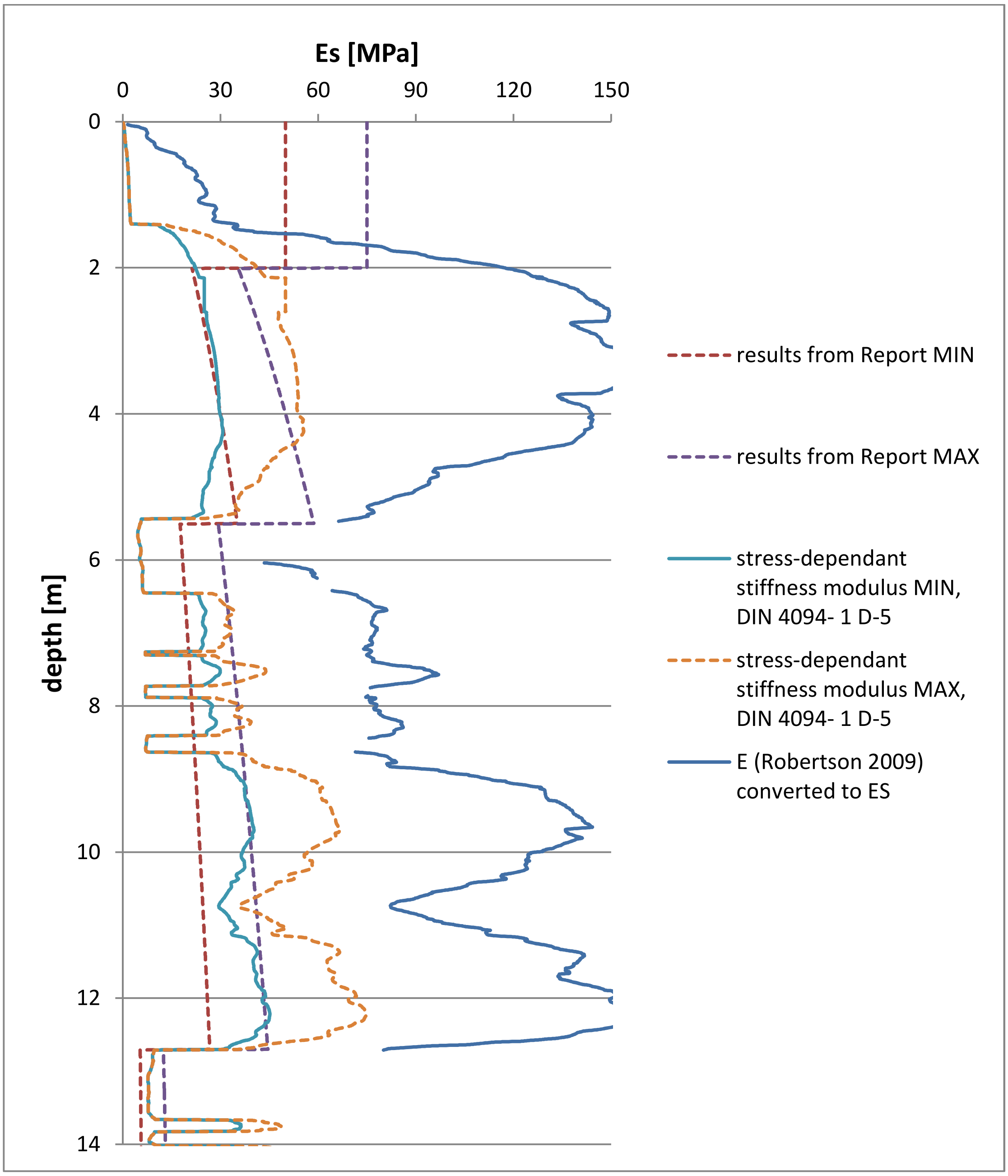


Figure 74: comparions of constrained modulus (converted Young´s modulus, and correlation of CPT result from Din 4094,D5 and results from geotechnical report) at project 4;

2.9.6 Estimation of the soil unit weight

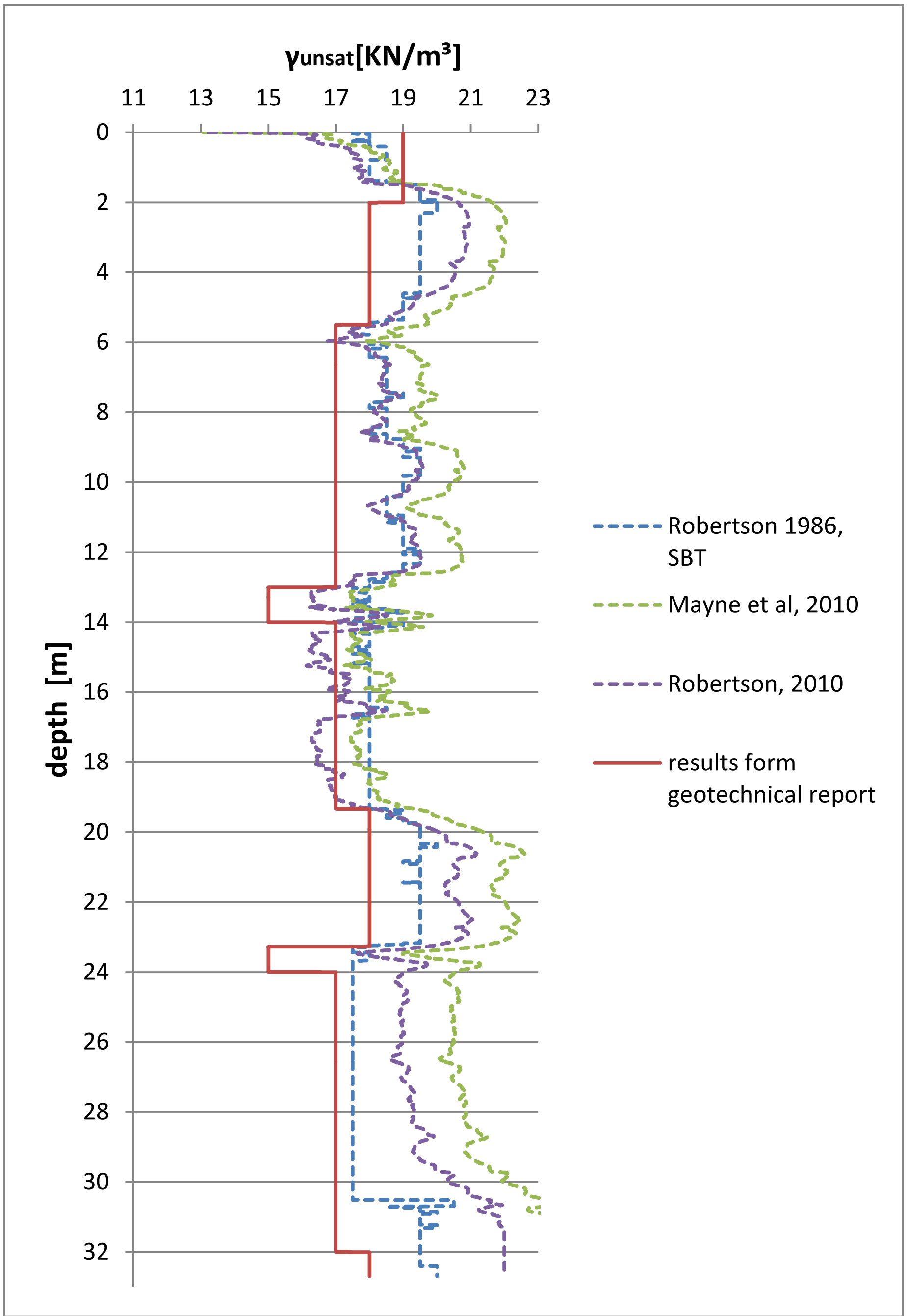


Figure 75: comparison of soil unit weight, data project 4;

2.9.7 Estimation of OCR

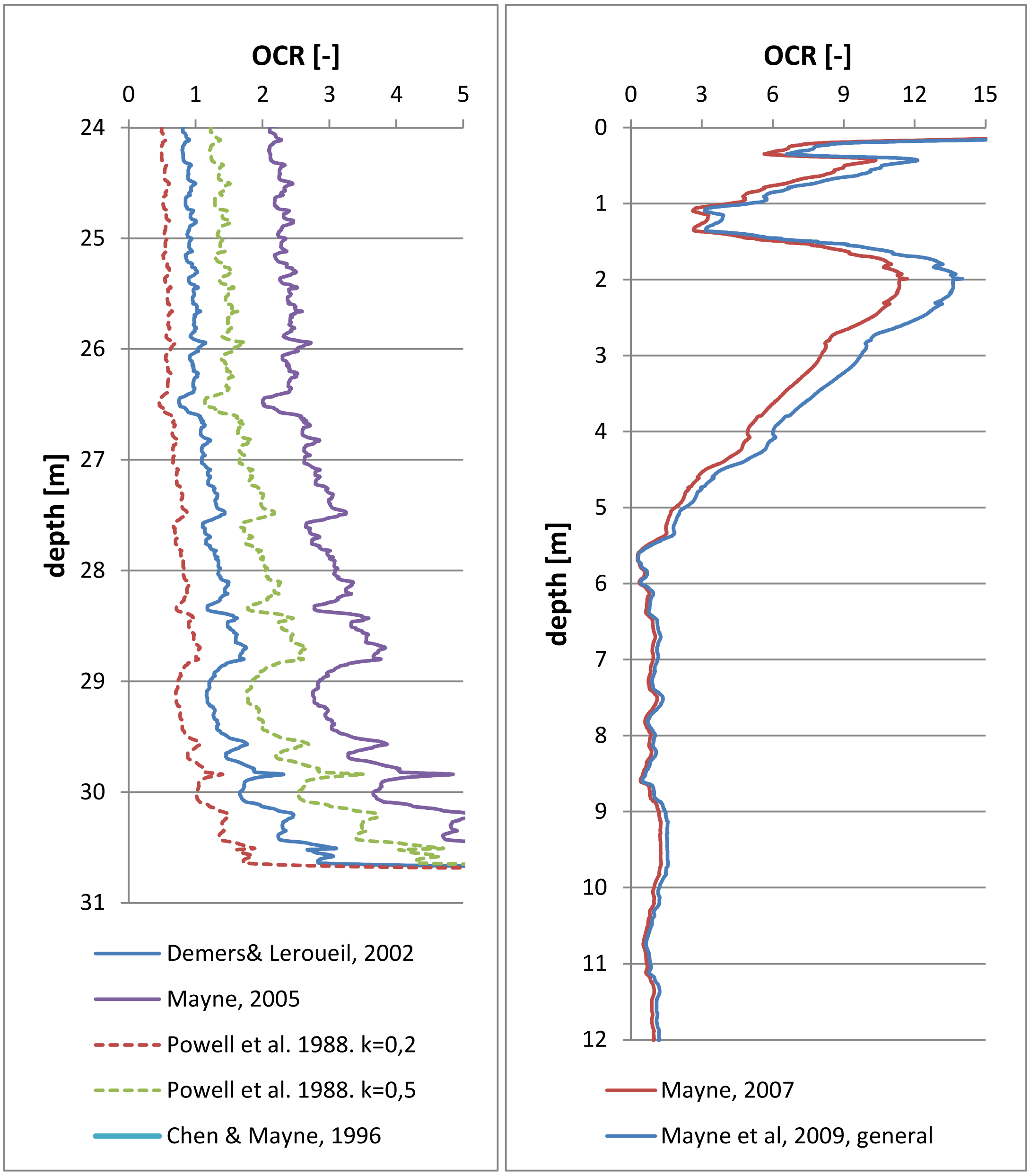


Figure 76: left: correlations for OCR in clays (depth 24- 31m); right: correlations for OCR in sands (depth 0- 12m);

2.9.8 Estimation of G_0

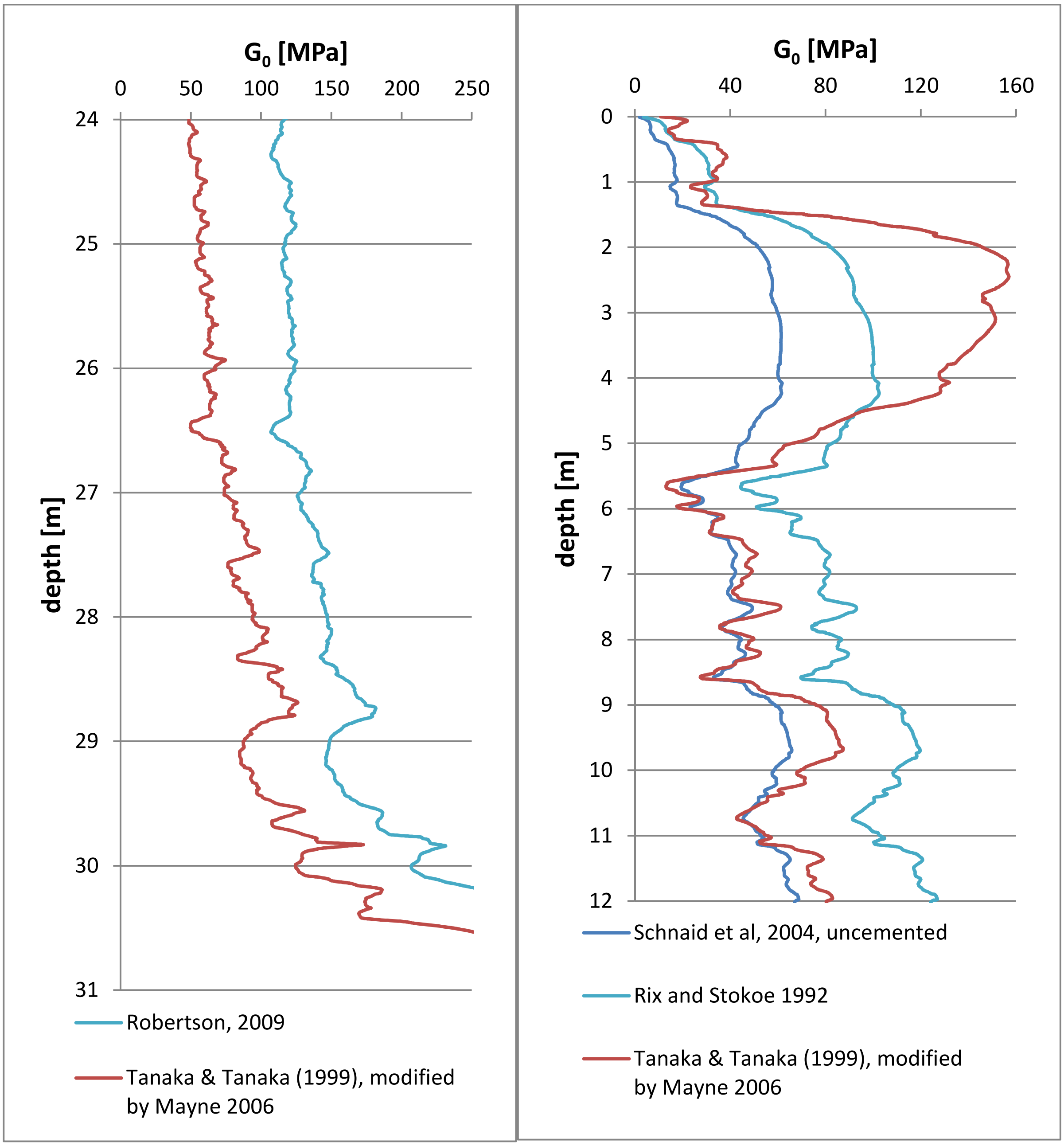


Figure 77: left: correlations for G0 in clays data (depth 24- 31m); right: correlations for G0 in sands (depth 0- 12m);

2.10 Project 5

For this project information was provided based on the output of GEOMIL software. A geotechnical report was not available.

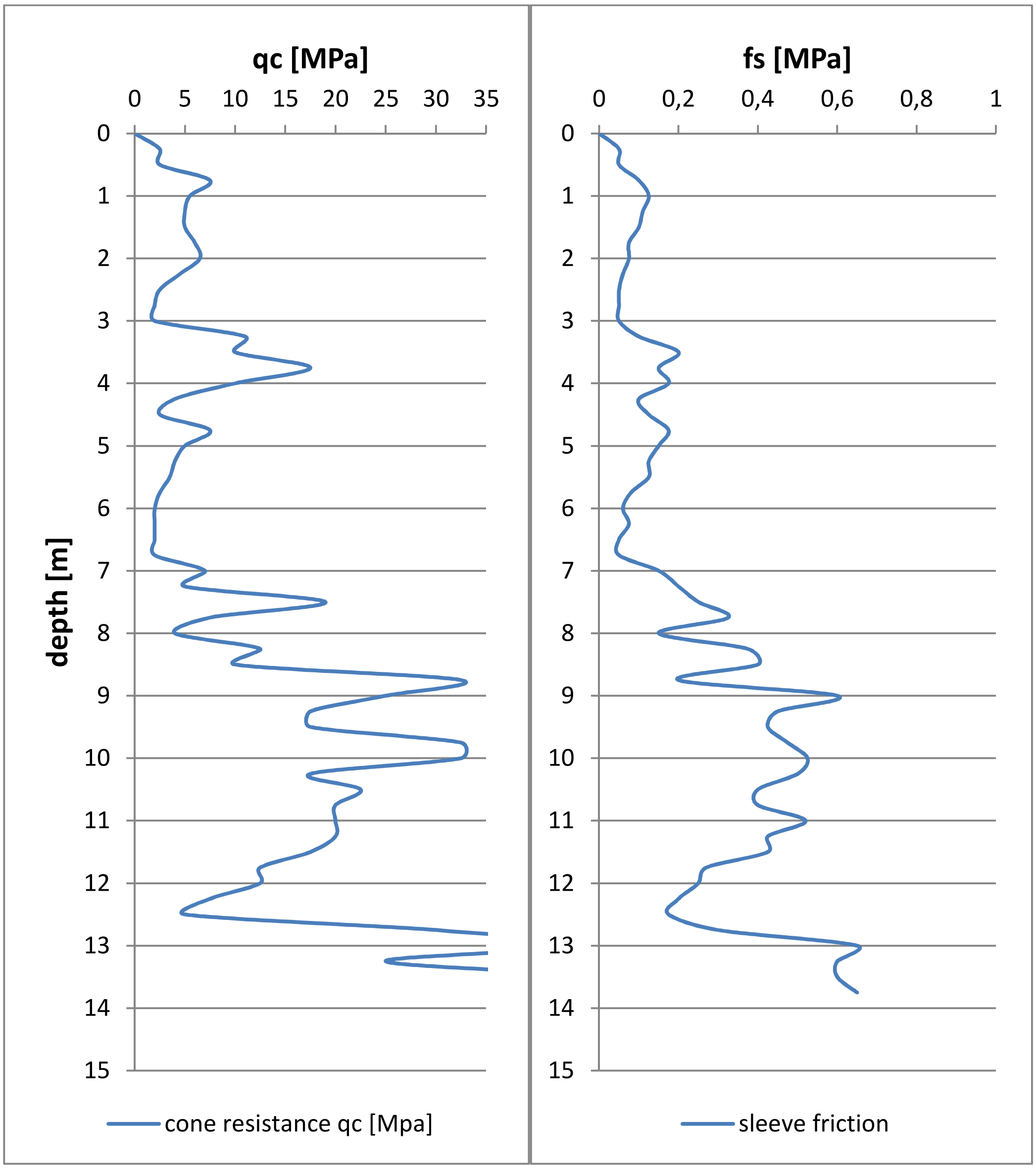


Figure 78: input parameters for project 5;

Because information about the undrained shear strength, the constrained modulus or other parameters is not provided, only the friction angle (because the soil mainly consists of sand) and the soil type are evaluated.

2.10.1 Estimation of φ´

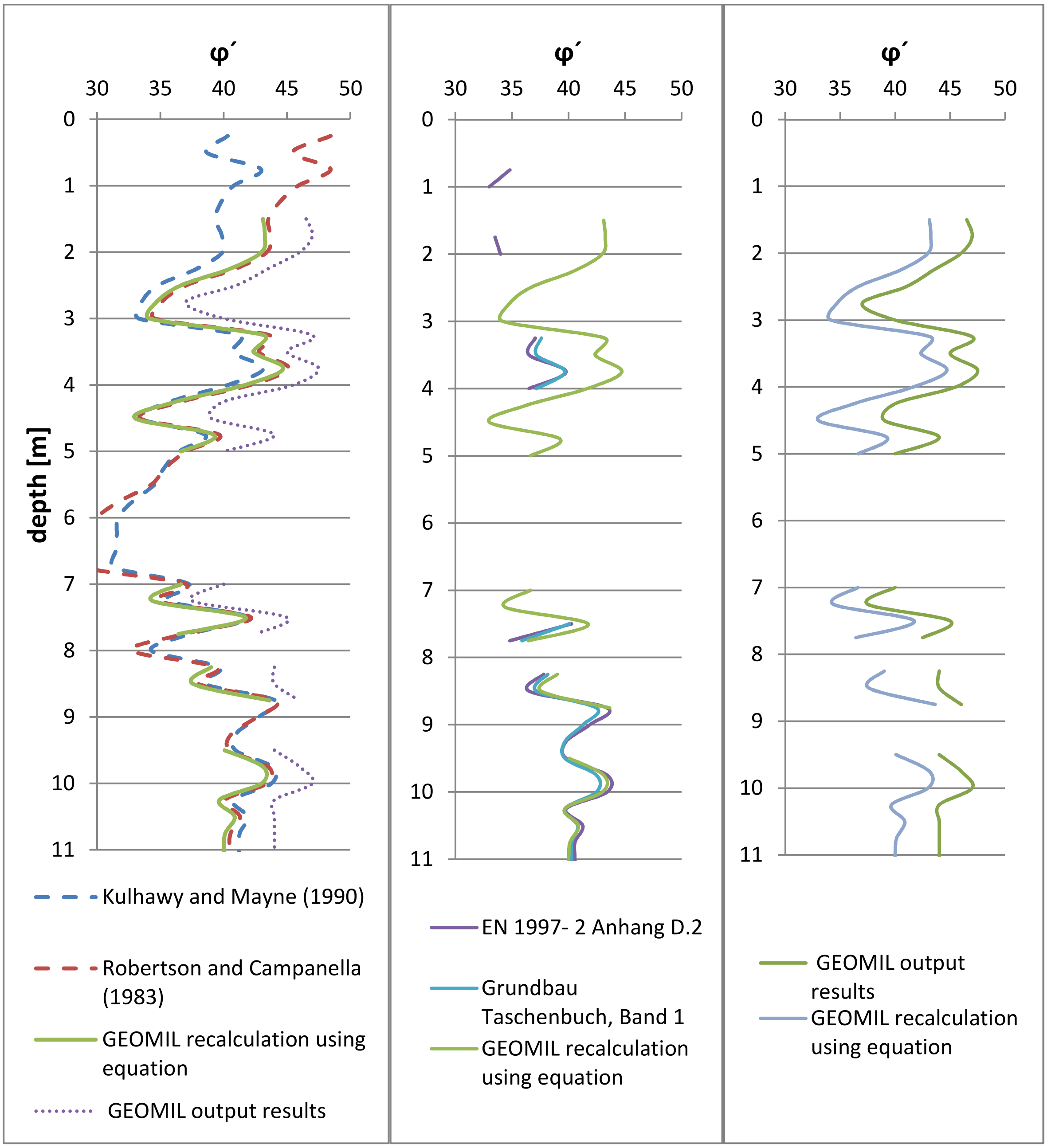


Figure 79: left: comparisons of φ´ correlations, middle: further correlations, right: comparison of GEOMIL output value and value obtained using equation provided by GEOMIL, data: project 5;

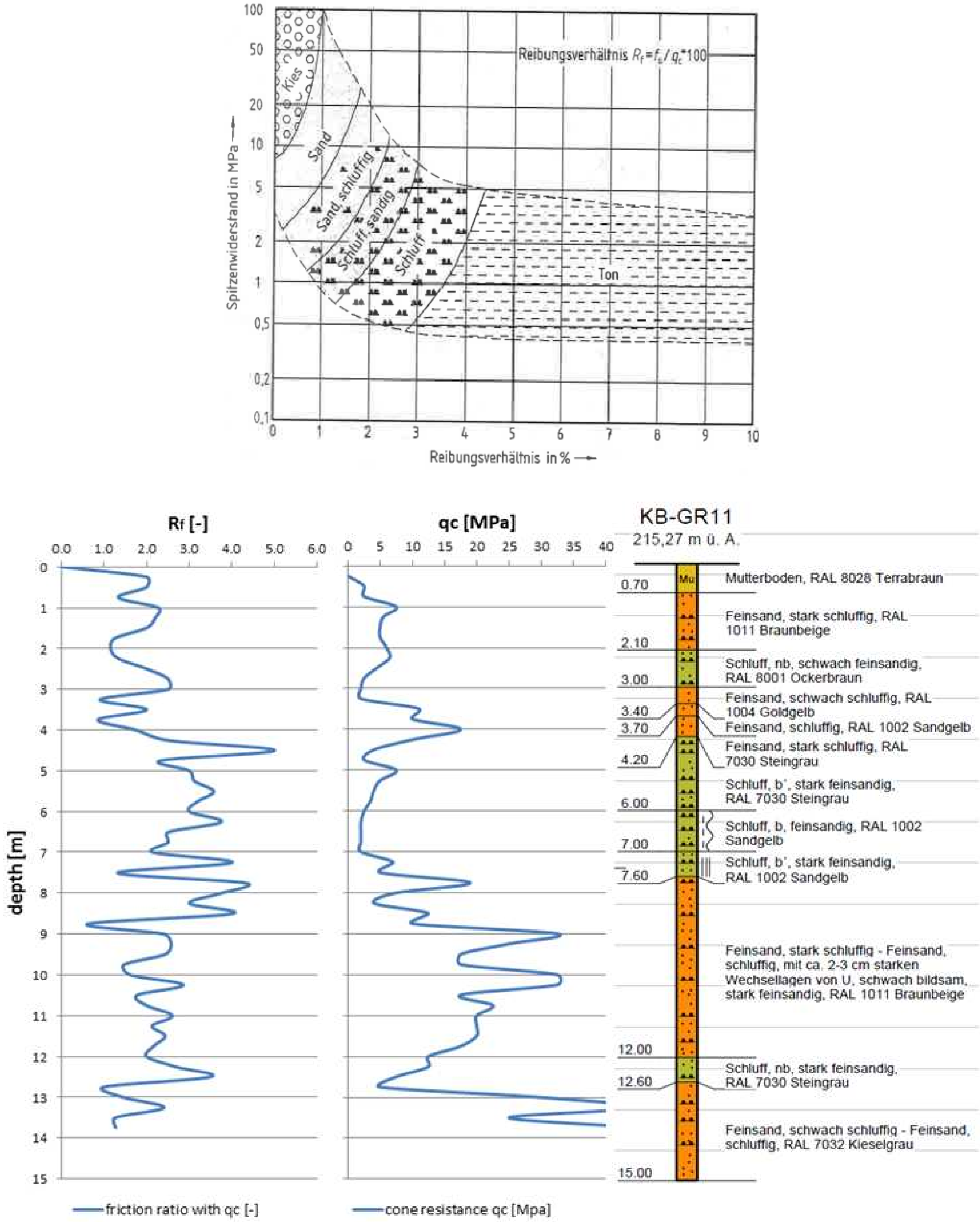


Figure 80: Soil type classification using figure from DIN 4094- 1, C1, project 5;

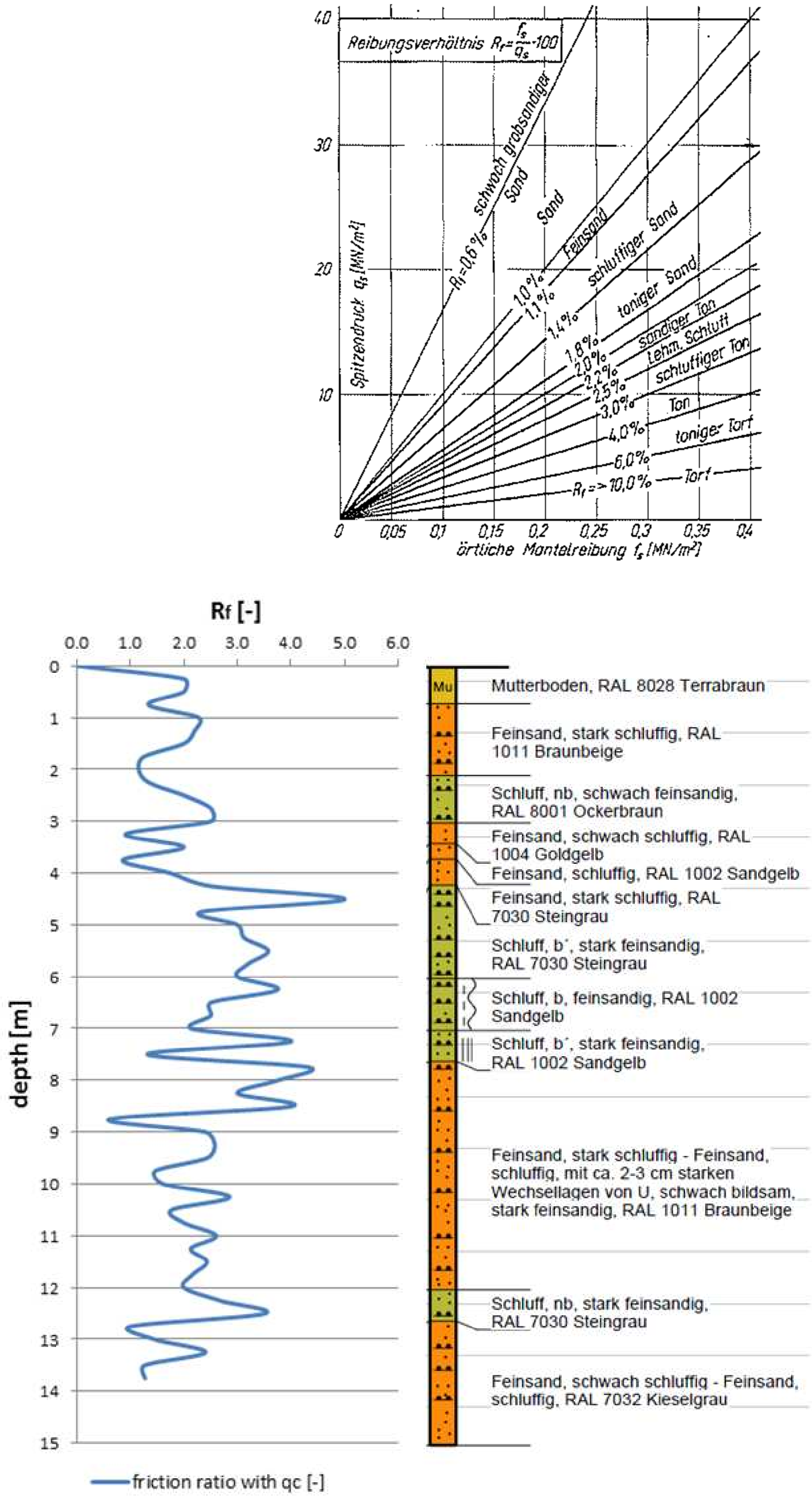


Figure 81: Soil type classification using the Fugro chart, project 5;

Table 4.1 Soil behavioral type by CPTu material index, I_c

Soil Classification	SBT Zone	Range CPTu Index, I_c	Range CPT Index I_{cRW}
Sands with gravels	7	$I_c < 1.25$	$I_{cRW} < 1.31$
Sands: clean to silty	6	$1.25 < I_c < 1.80$	$1.31 < I_{cRW} < 2.05$
Sandy mixtures	5	$1.80 < I_c < 2.40$	$2.05 < I_{cRW} < 2.60$
Silty mixtures	4	$2.40 < I_c < 2.76$	$2.60 < I_{cRW} < 2.95$
Clays	3	$2.76 < I_c < 3.22$	$2.95 < I_{cRW} < 3.60$
Organic soils	2	$I_c > 3.22$	$I_{cRW} > 3.60$
Sensitive soils	1	NA	NA

Notes: 1. Index I_c after Jefferies & Been (2006).
2. Index I_{cRW} after Robertson & Wride (1998).

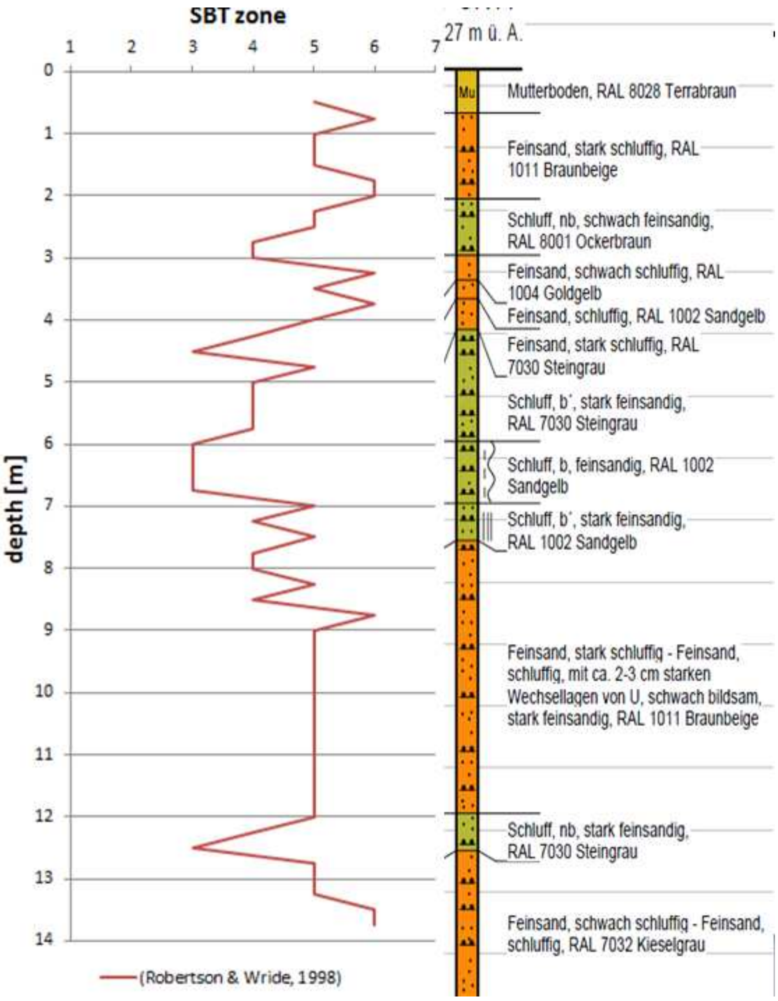


Figure 82: Soil type classification using approach of Robertson& Wride 1998, project 5;

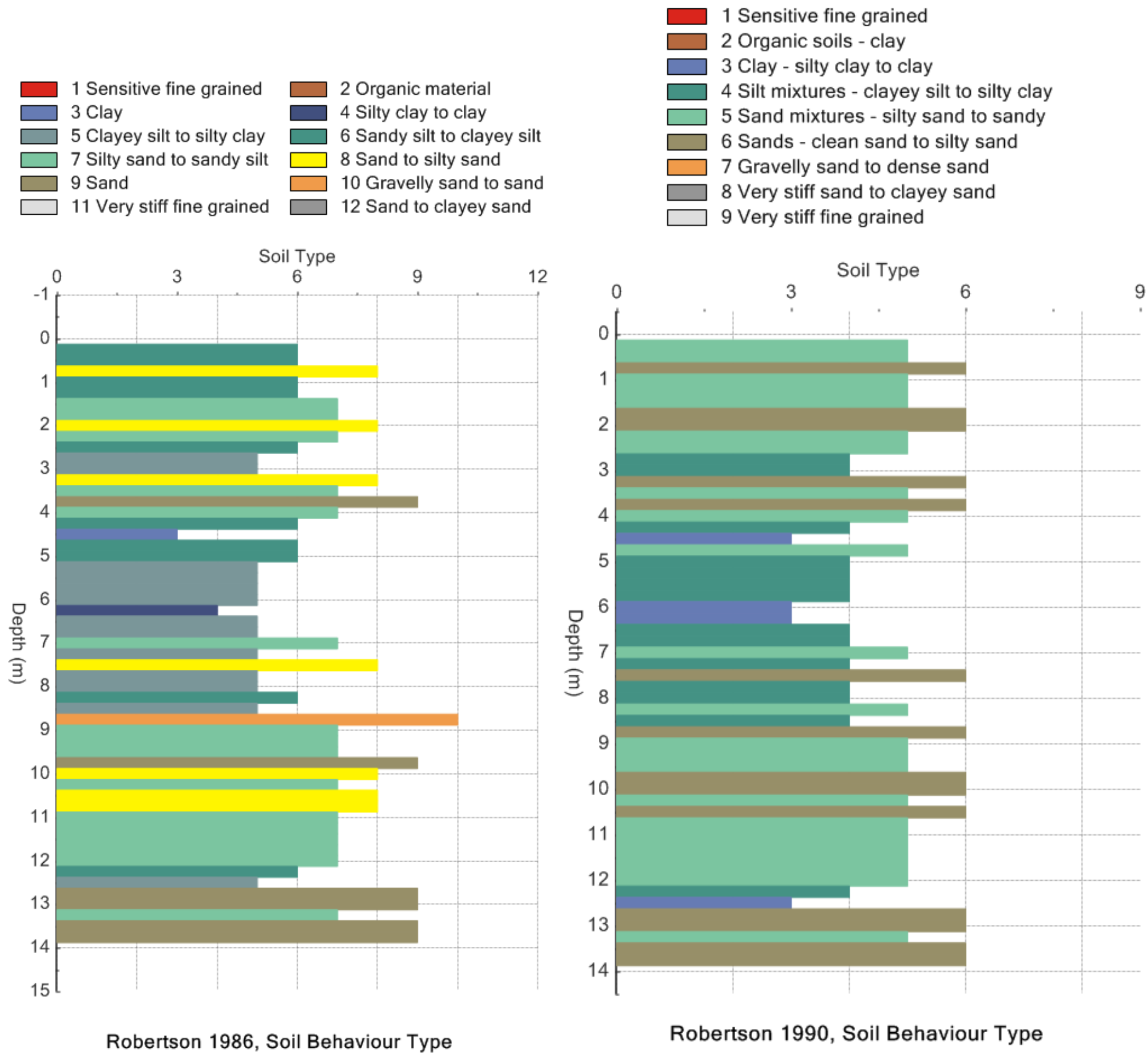


Figure 83: soil type evaluation comparison of Robertson 1986 and 1990, project 5;

2.11 Project 6

As for project 5, information of a borehole and the GEOMIL evaluation was available.

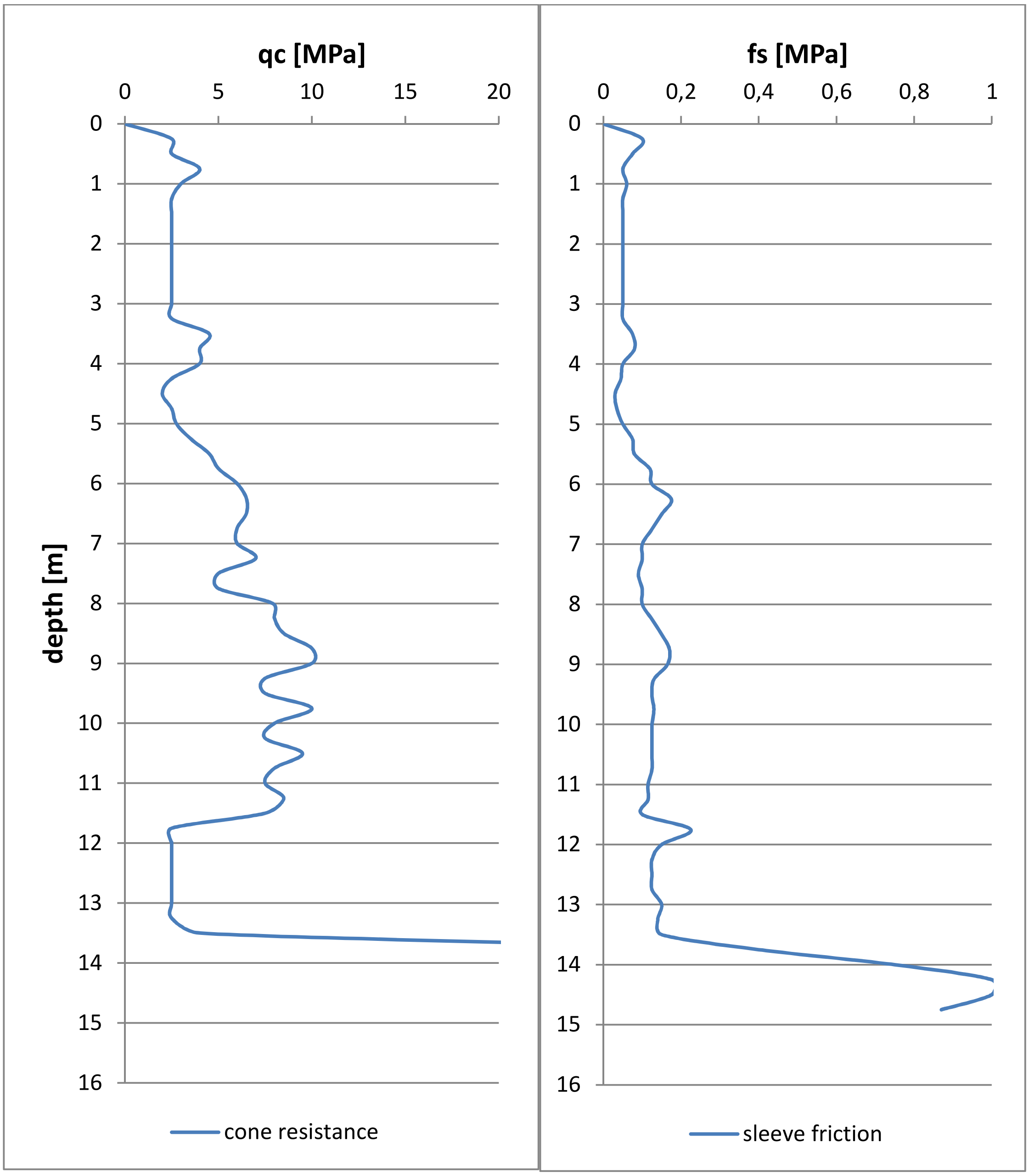
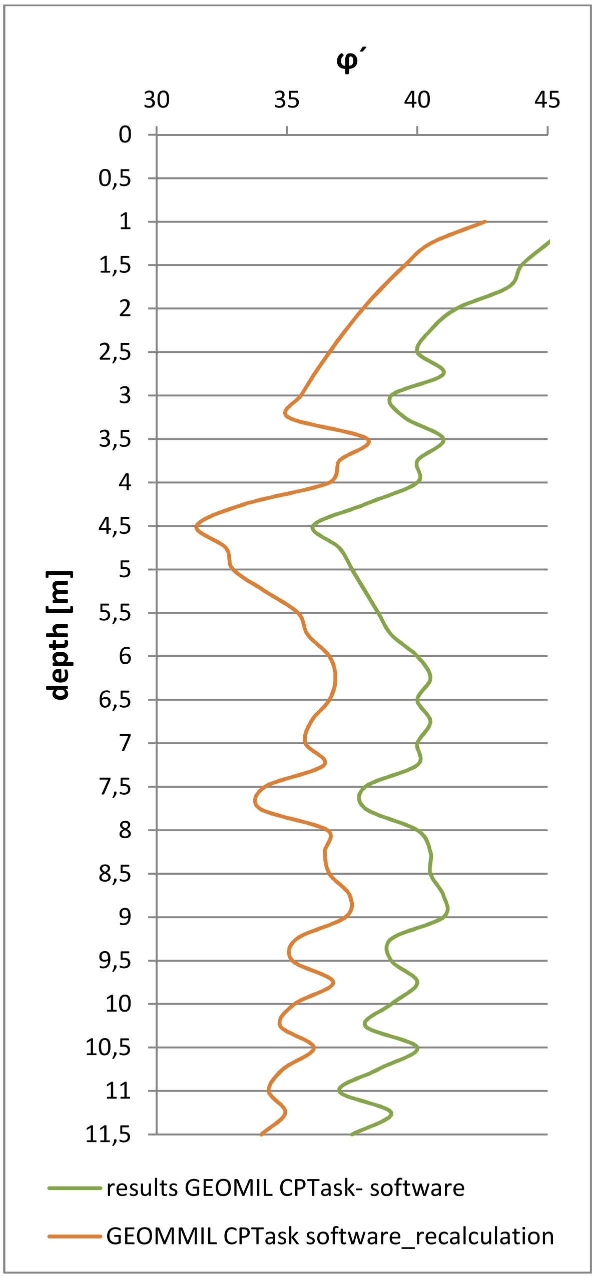


Figure 84: Input parameter for project 6;

2.11.1 Estimation of φ´



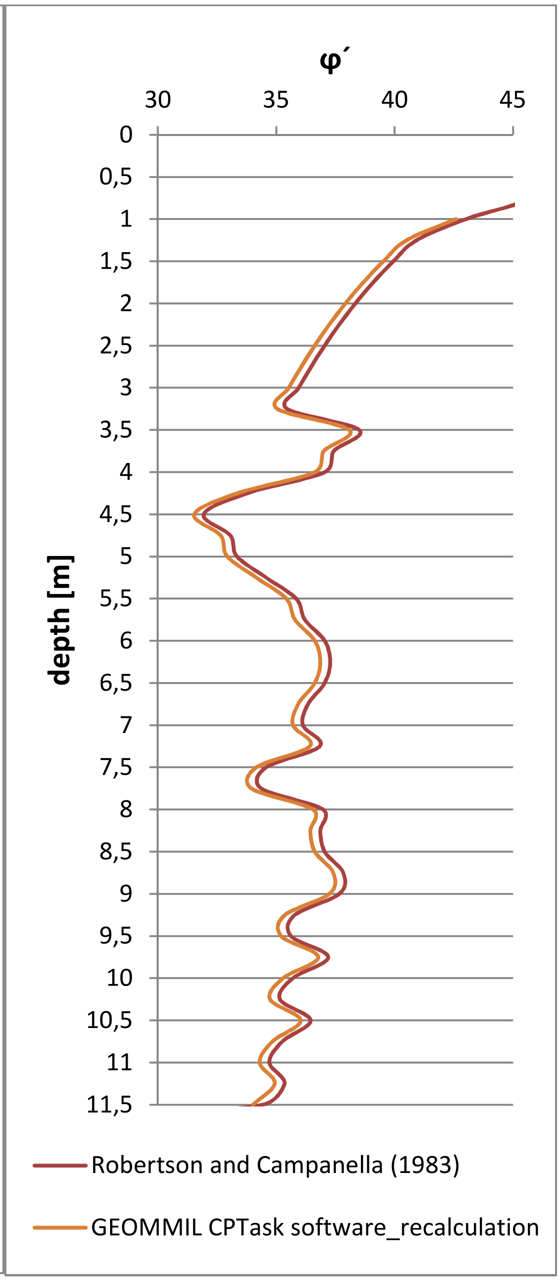


Figure 85: Comparison of φ´ (output value of GEOMIL, equation provided by GEOMIL and Robertson and Campanella (1983)

2.11.2 Estimation of soil type

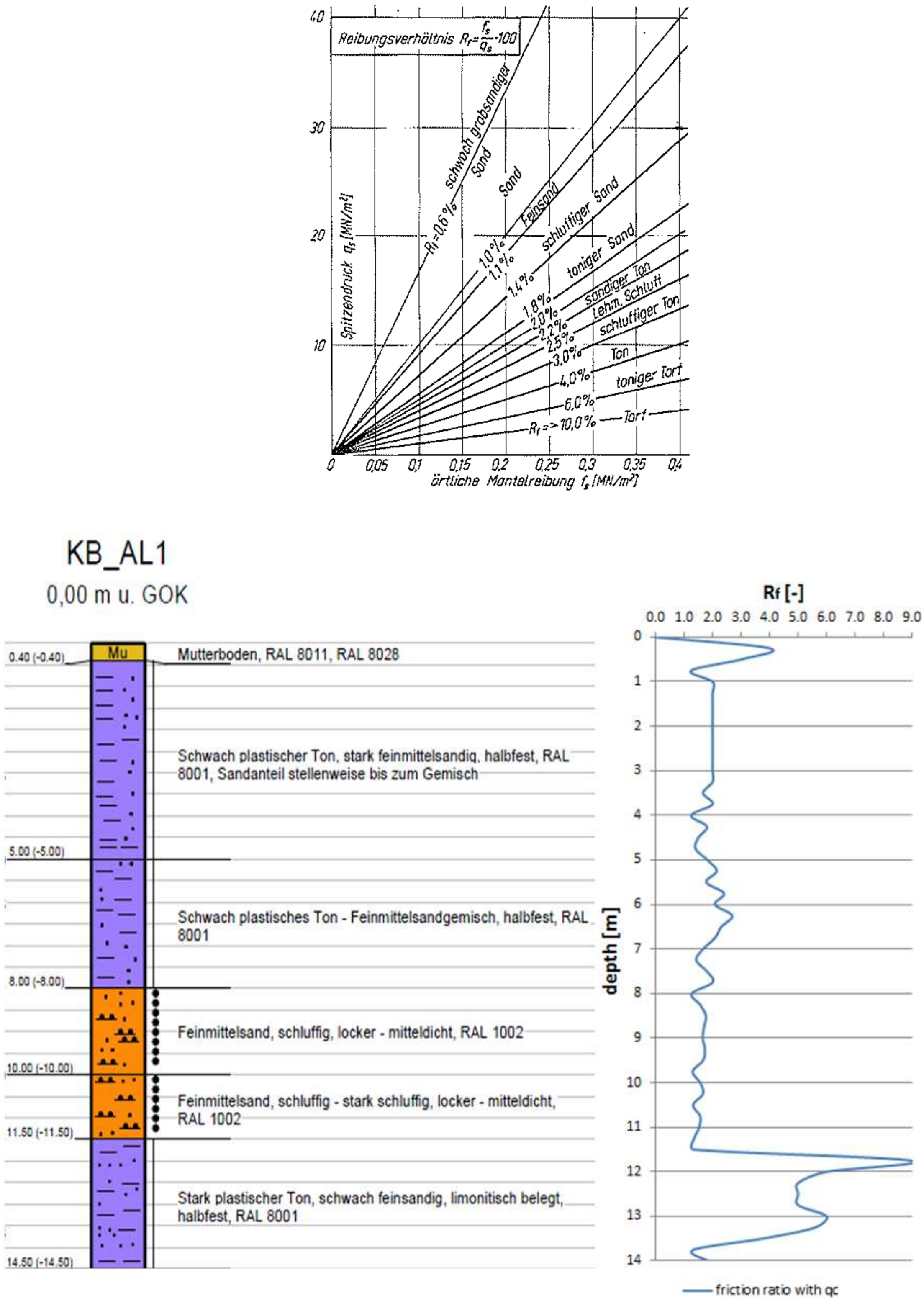


Figure 86: Comparison of borehole results and Fugro- correlation for soil type classification

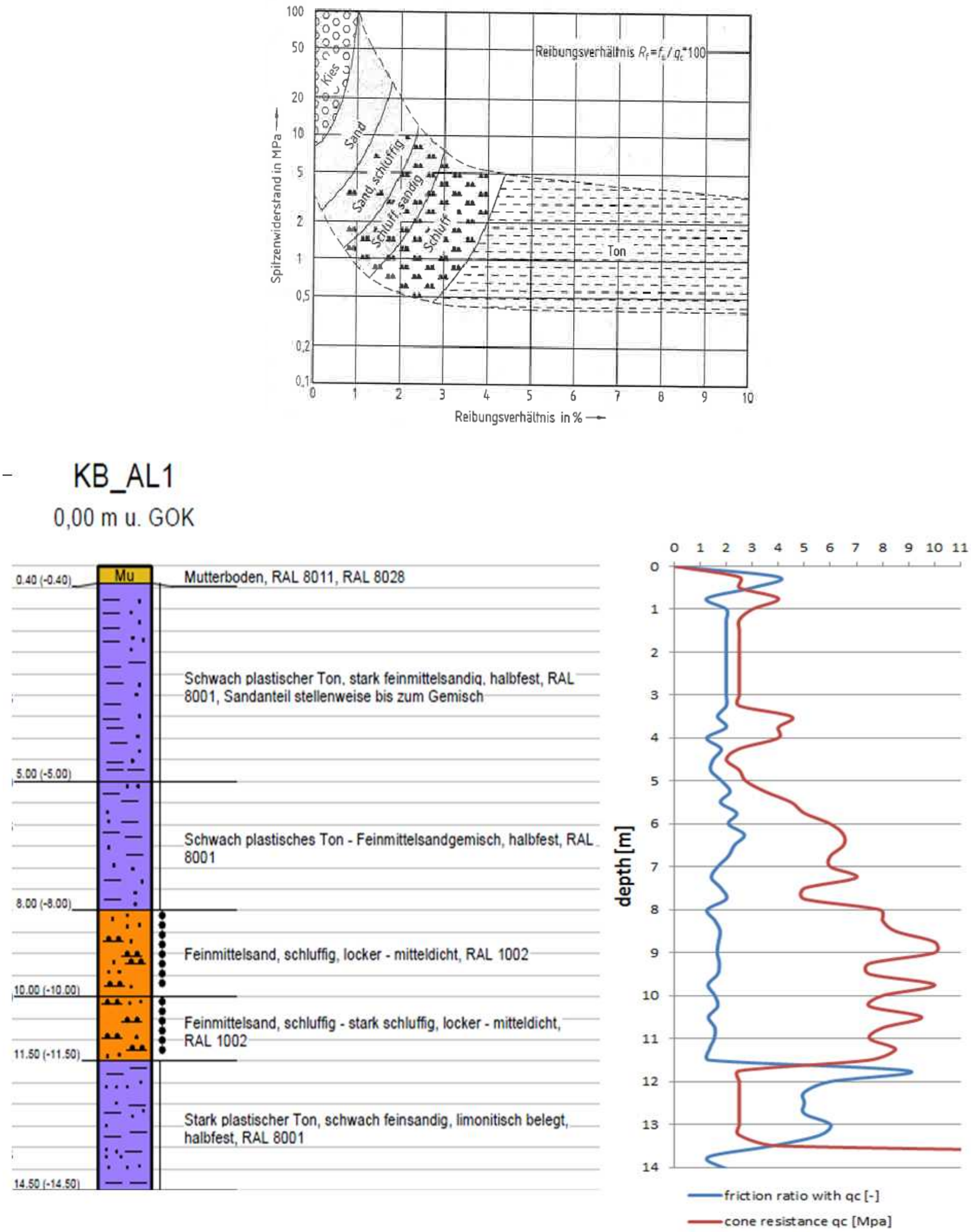


Figure 87: Soil type classification using figure from DIN 4094- 1, C1 (project 6);

Table 4.1 Soil behavioral type by CPTu material index, I_c

Soil Classification	SBT Zone	Range CPTu Index, I_c	Range CPT Index I_{cRW}
Sands with gravels	7	$I_c < 1.25$	$I_{cRW} < 1.31$
Sands: clean to silty	6	$1.25 < I_c < 1.80$	$1.31 < I_{cRW} < 2.05$
Sandy mixtures	5	$1.80 < I_c < 2.40$	$2.05 < I_{cRW} < 2.60$
Silty mixtures	4	$2.40 < I_c < 2.76$	$2.60 < I_{cRW} < 2.95$
Clays	3	$2.76 < I_c < 3.22$	$2.95 < I_{cRW} < 3.60$
Organic soils	2	$I_c > 3.22$	$I_{cRW} > 3.60$
Sensitive soils	1	NA	NA

Notes: 1. Index I_c after Jefferies & Been (2006).
2. Index I_{cRW} after Robertson & Wride (1998).

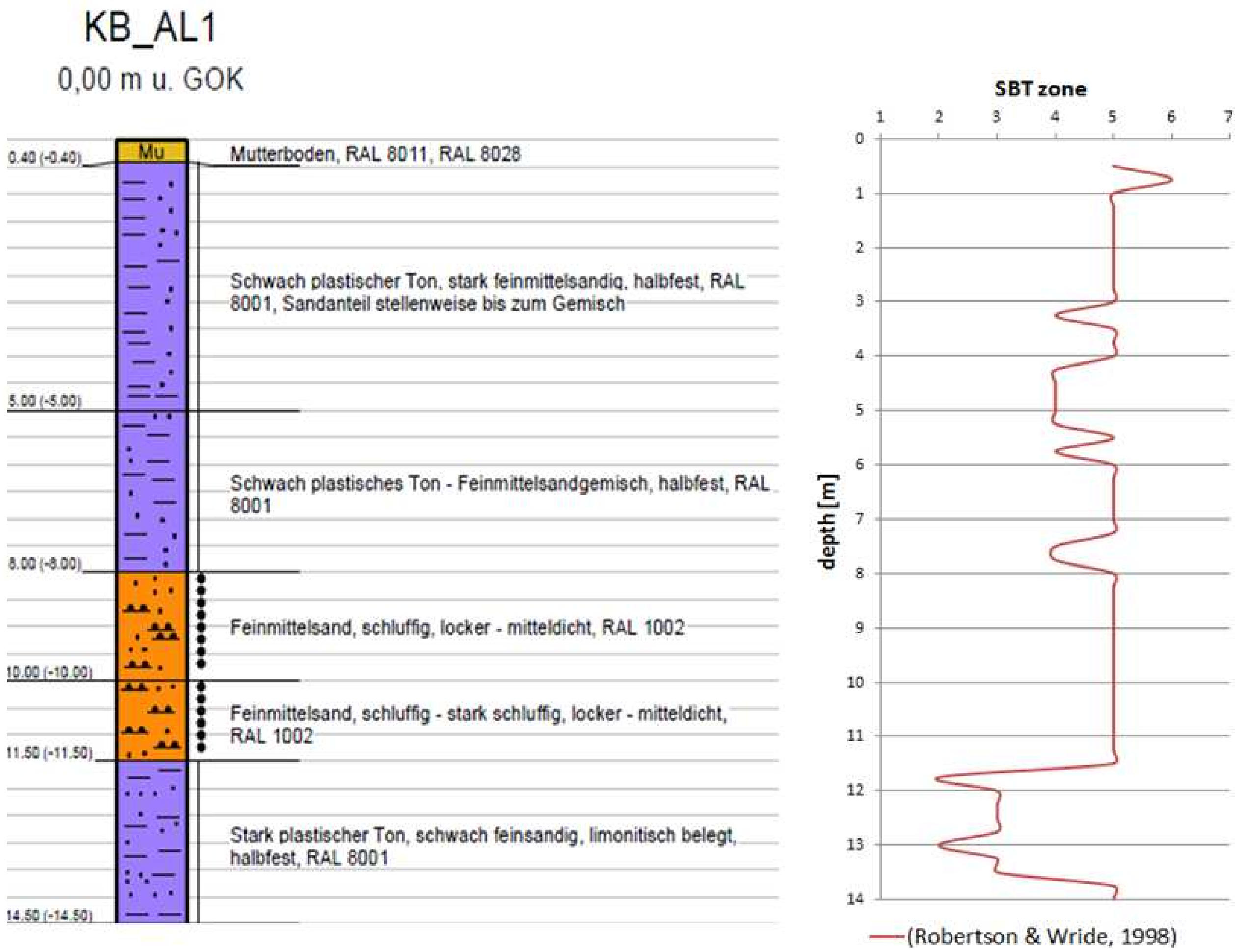


Figure 88: Soil type classification using SBT zones by Robertson& Wride 1998 (project 6);

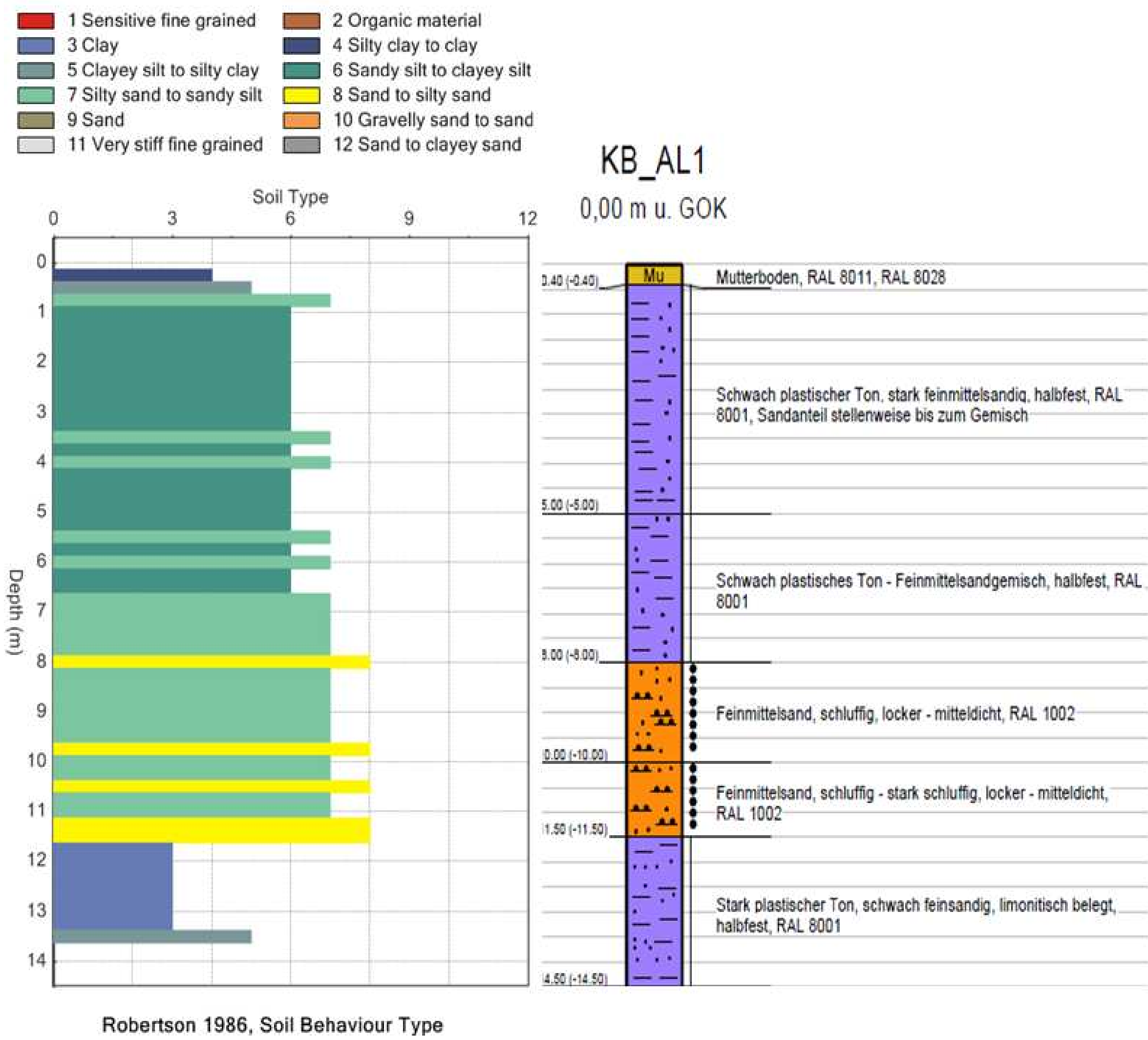


Figure 89: Soil type classification using Robersom 1986, SBT (project 6);

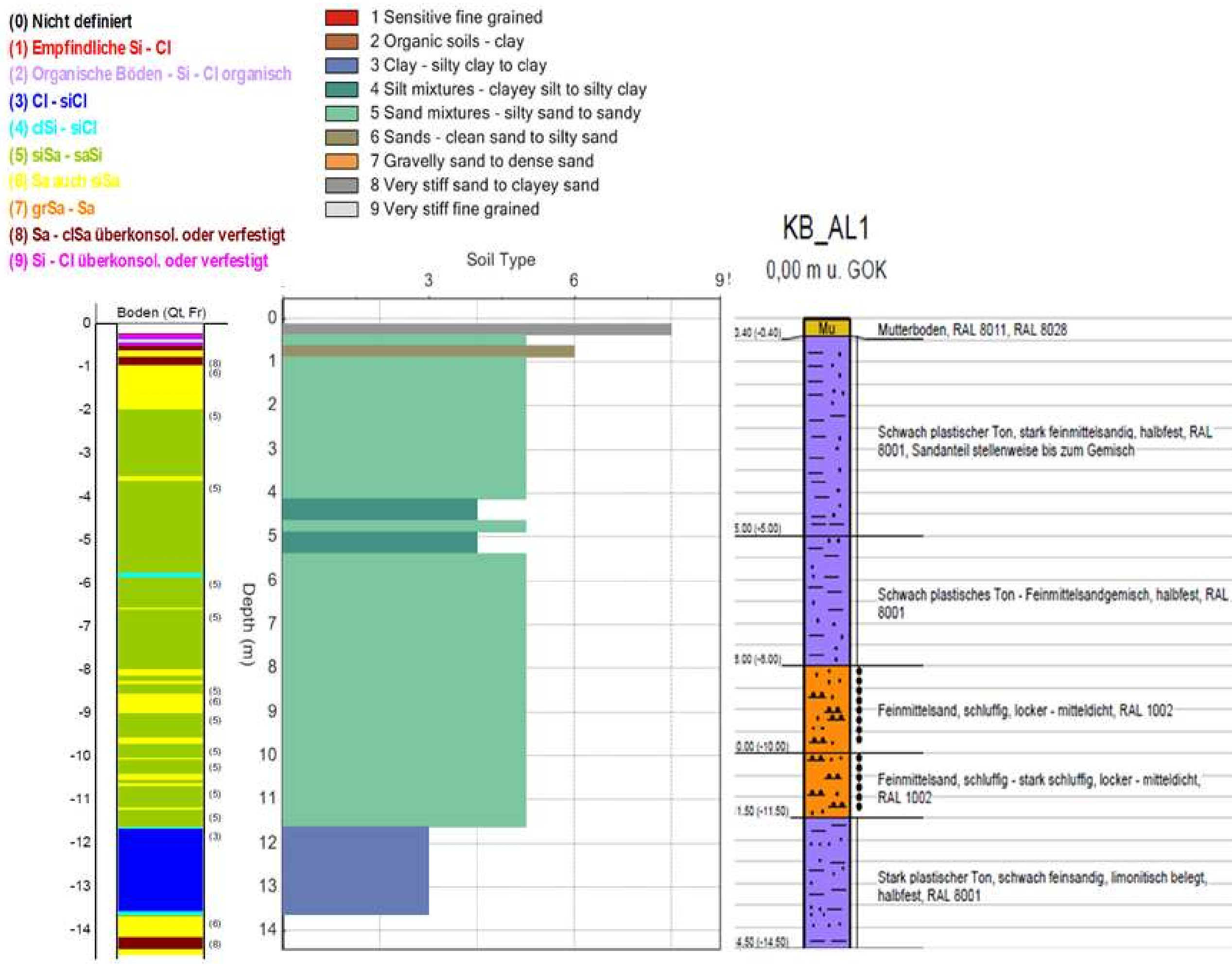


Figure 90: comparison of output results from GEOMIL, NOVOCPT, both using Robertson 1990 and borehole (project 6);

2.12 Project 7

Information of the GEOMIL evaluation and a borehole was provided.

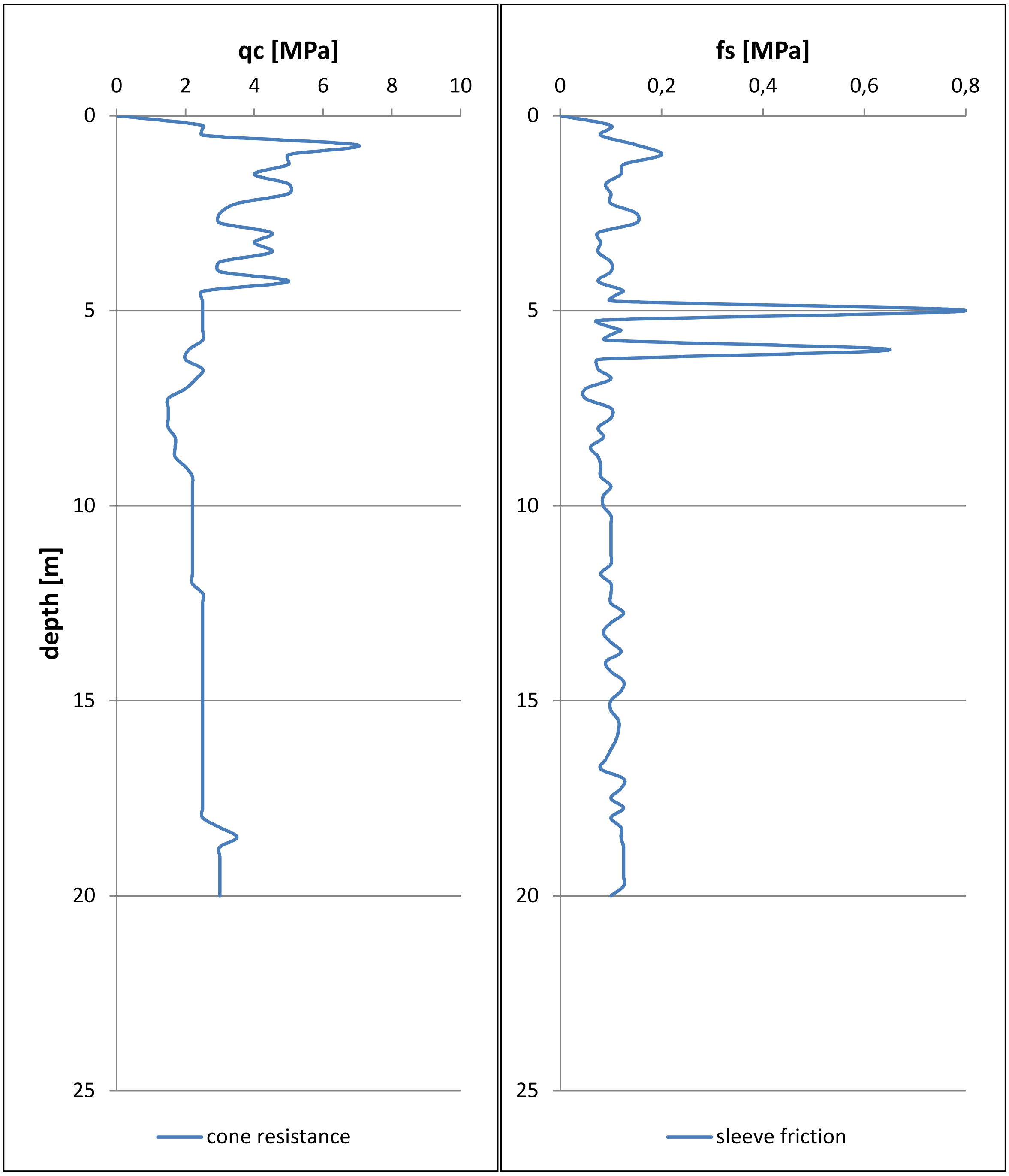


Figure 91: Input parameter, project 7;

2.12.1 Estimation of G_0

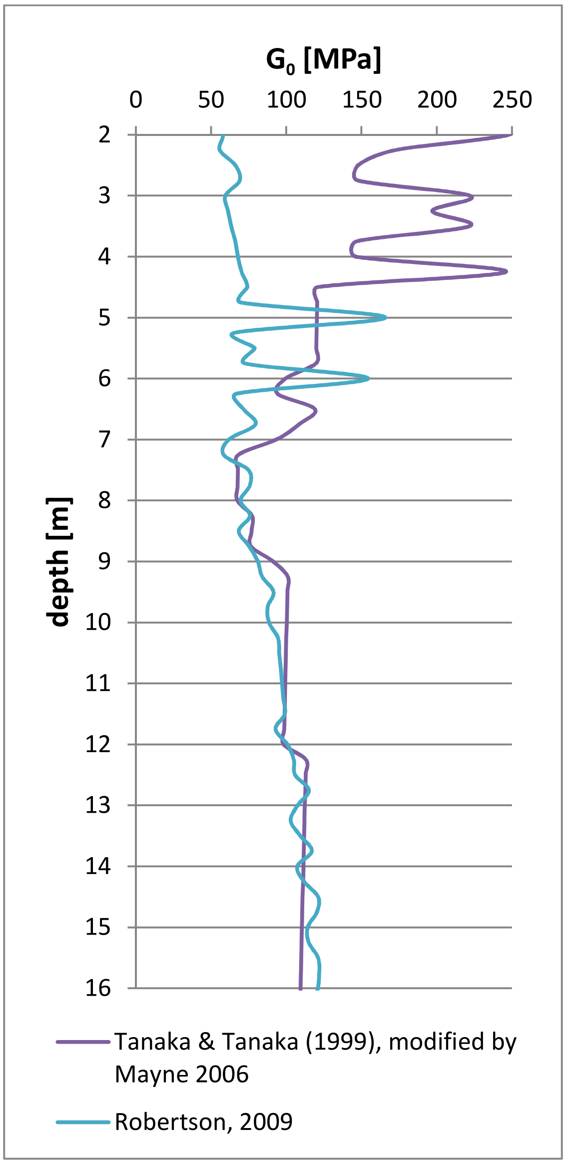


Figure 92: correlations for G0 in clays with data project 7;

2.13 Project 8

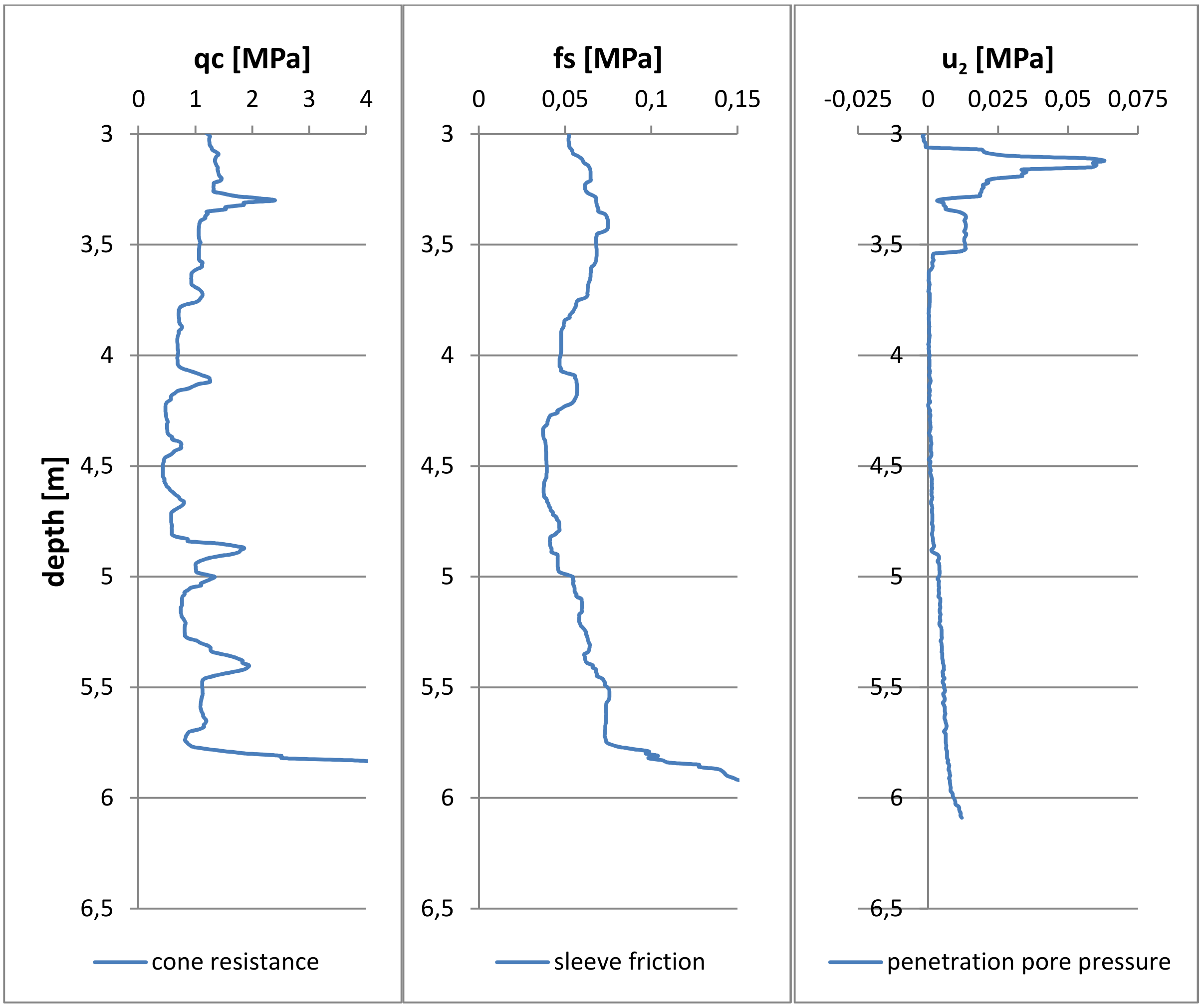


Figure 93: Input parameter from project 8;

More information on the project: The Test was conducted according to NEN 5140 standard as an electric CPT with filter elements for measuring the dynamic pore water pressure located behind the cone (u_2). The cone that is used has a base area of 15cm², which is not the standard cone. Results and correlations are presented according to the German Standard (DIN 4094-1). Predrilling was required in the first few meters (with varying depth, approximately 3m) (compare geotechnical report)

The Geotechnical report further states that predrilling has been conducted at the CPT in the backfill area (~3m) and should be considered in the evaluation of CPT results. Neither layer water nor a groundwater horizon was encountered in the CPT.

Table 15: Stratification and classification of the subsoil (compare geotechnical report)

Schichtkomplex	Tiefe von [m unter GOK]	bis [m unter GOK]	Bezeichnung
A	0	2,5/5	Anschüttung: sandiger Kies
B	2,5/5	7,5/11,0	Torf oder organischer Schluff
C	7,5/11,0	17,5/>20,0	Sandiger Kies bzw. Kiessand

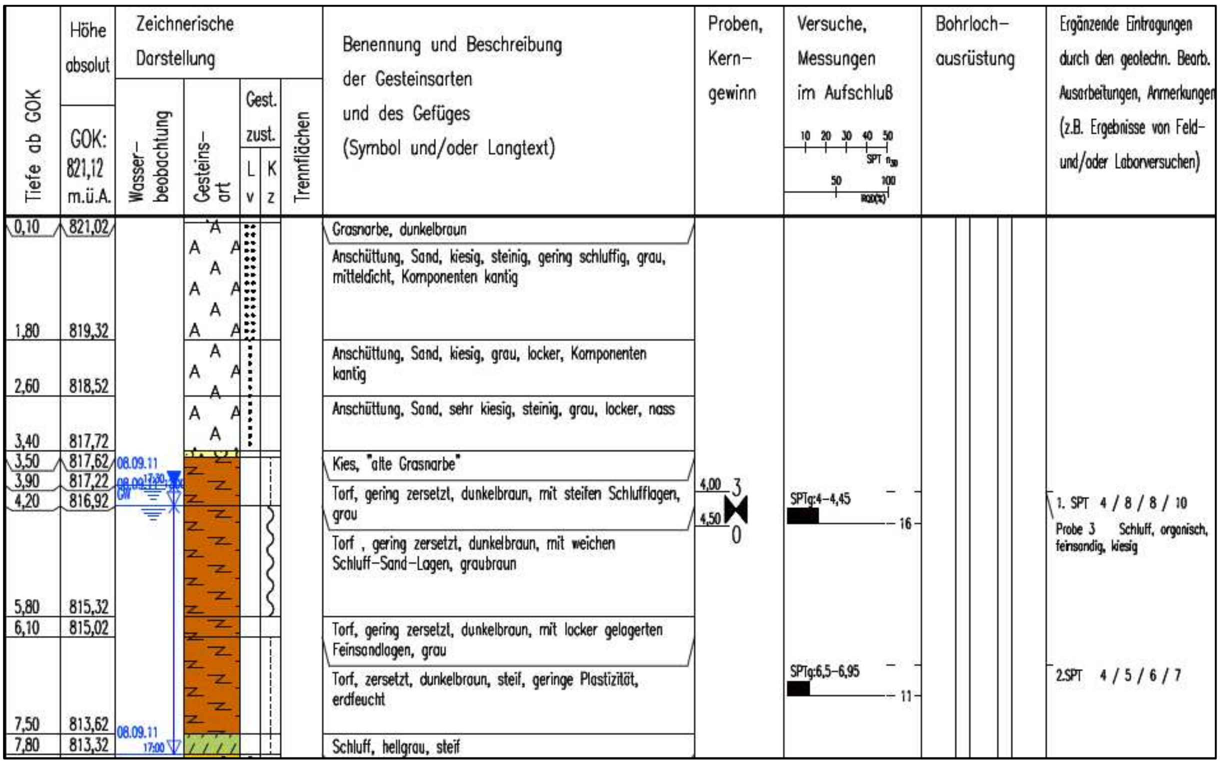


Figure 94: Borehole, located close to the CPT (compare geotechnical report)

The parameters E_s and c_u in the geotechnical report have been evaluated using CPT data according to Din 4094-1.

The results and correlations of the undrained shear strength (c_u) that vary between 40- 140 KN/m² and the constrained modulus (E_s) in a range of 1.500- 9.500 KN/m² in the layer B will be investigated. Remarkable is the broad range of both values (E_s and c_u) stated in the geotechnical report.

Furthermore, verifying the peat and organic layer with the use of CPT correlations on the soil type will prove the correctness of existing soil type charts and equations.

2.13.1 Estimation of Es

The conservative estimations after DIN 4094-1, D-6 show good consistency with results from the geotechnical report.

The α - value is assumed to be in a range of 1, 5 (min value) and 8 (max value) (compare: DIN 4094- 1, D-6).

The stress- dependant correlation is right in between and doesn´t show any outliers.

Results from Mayne (2001) and Robertson (2009) give again somewhat higher values than recommended in the geotechnical report.

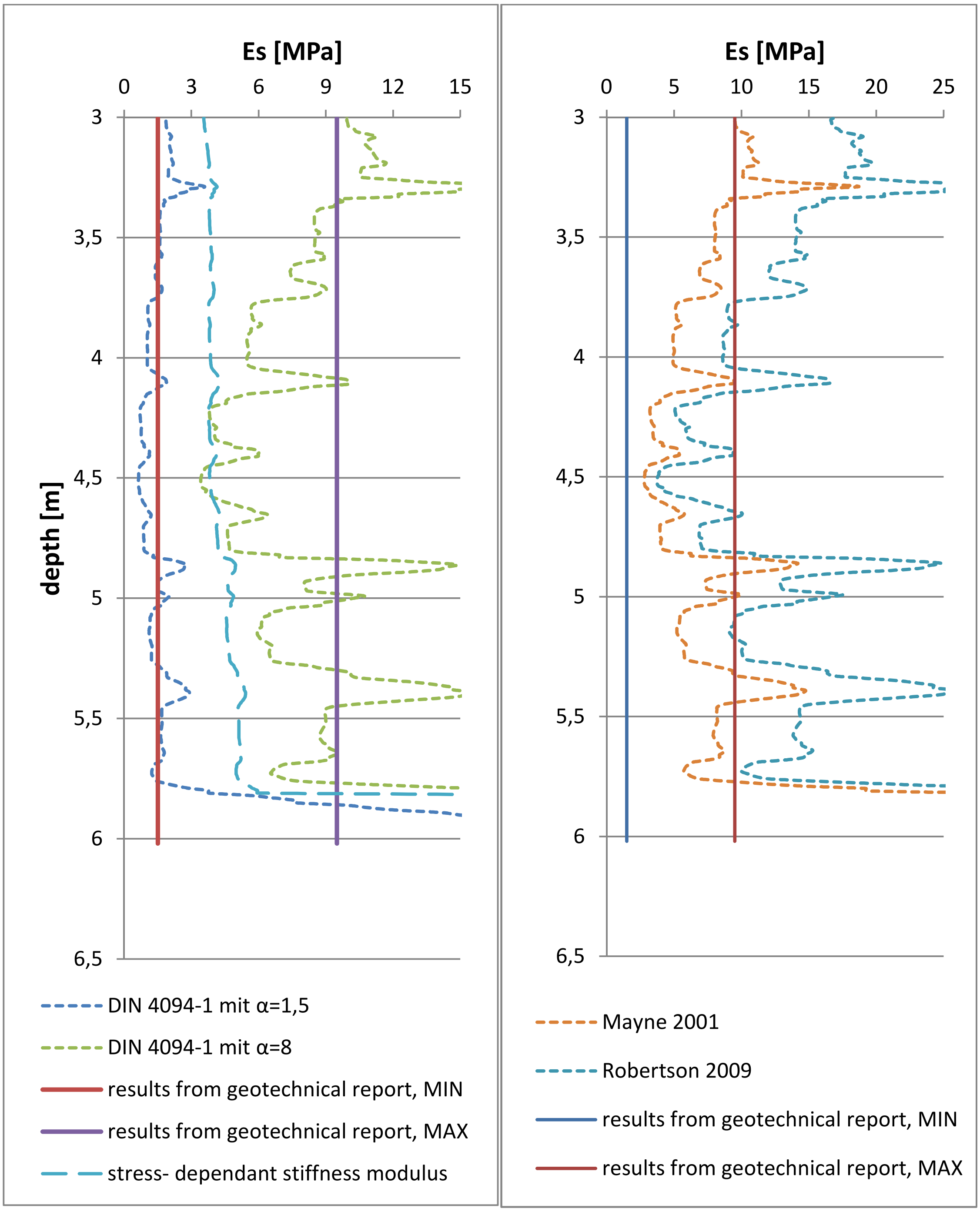


Figure 95: Correlations of Es, data: project 8;

2.13.2 Estimation of c_u

Because laboratory tests don´t provide any information on the plasticity index I_P (which may not be possible to obtain in such soils), the estimation of c_u is just conducted by applying empirical approaches from Aas et al (1986) using the total cone resistance. The values fit quite well (except some outliers) with those shown in the geotechnical report (40 to 140KPa, compare Figure 96).

Estimations using the effective cone resistance (with the approach from Senneset et al. 1982, 6- 12 for the range of N_{ke} value) show higher values than those, introduced in the Geotechnical Report and from the total cone resistance (compare Figure 96).

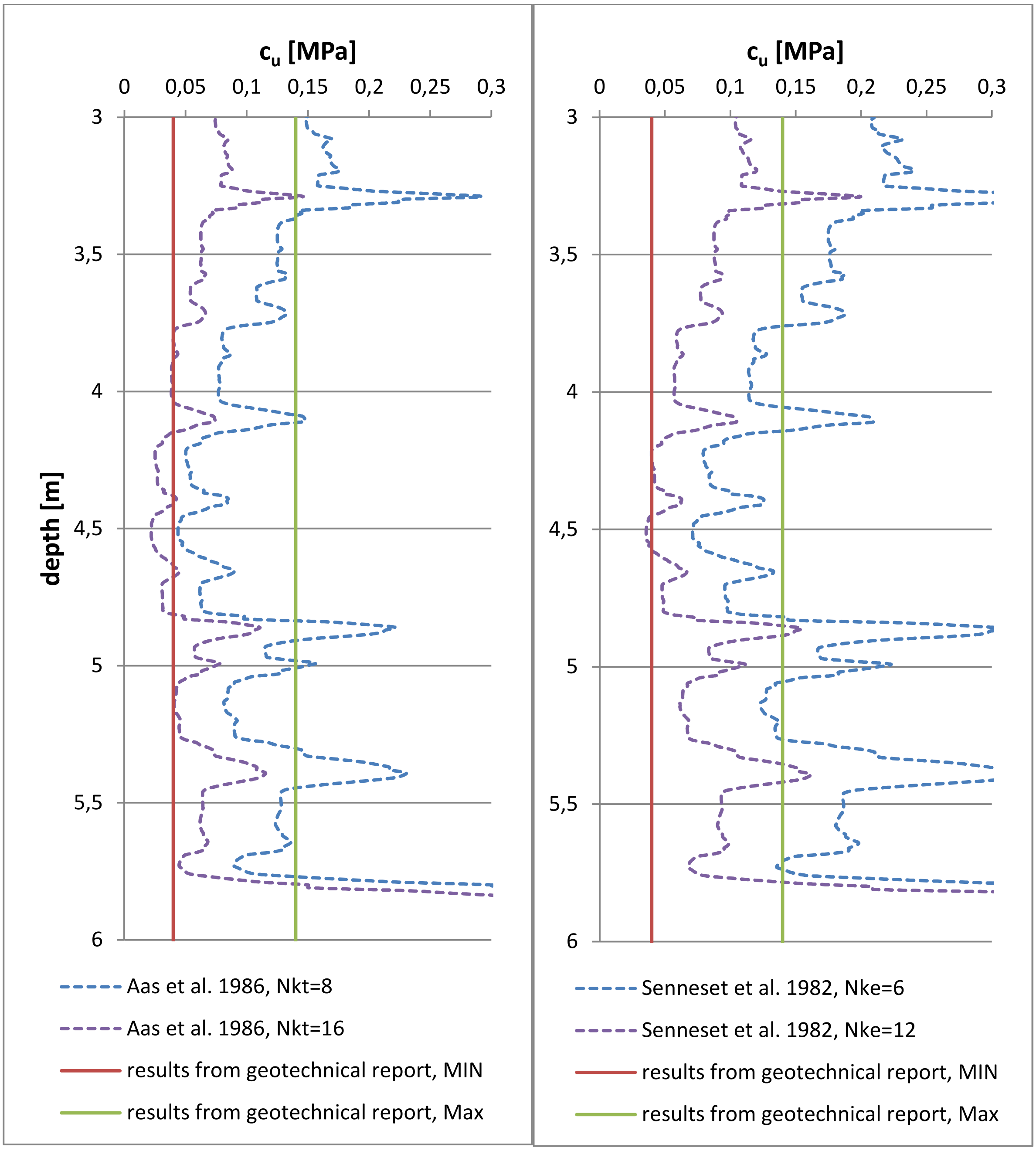


Figure 96: Correlations for c_u, data project 8;

2.13.3 Estimation of soil type

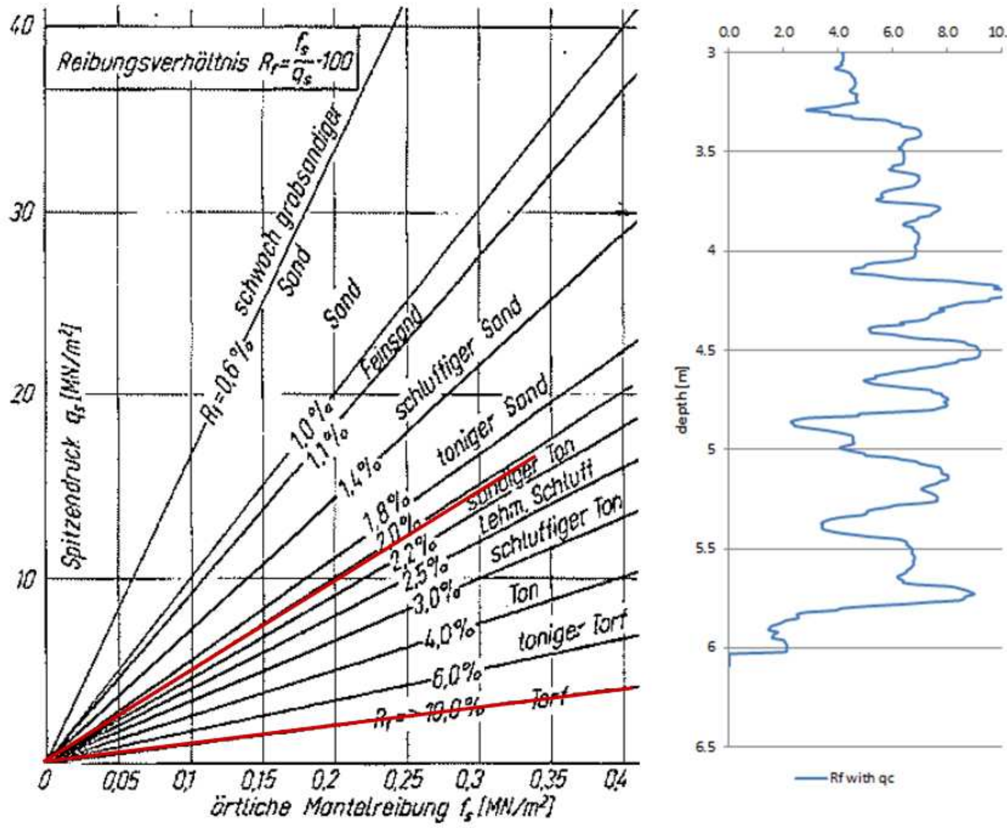


Figure 97: Fugro soil type chart and friction ratio (2- 10%);

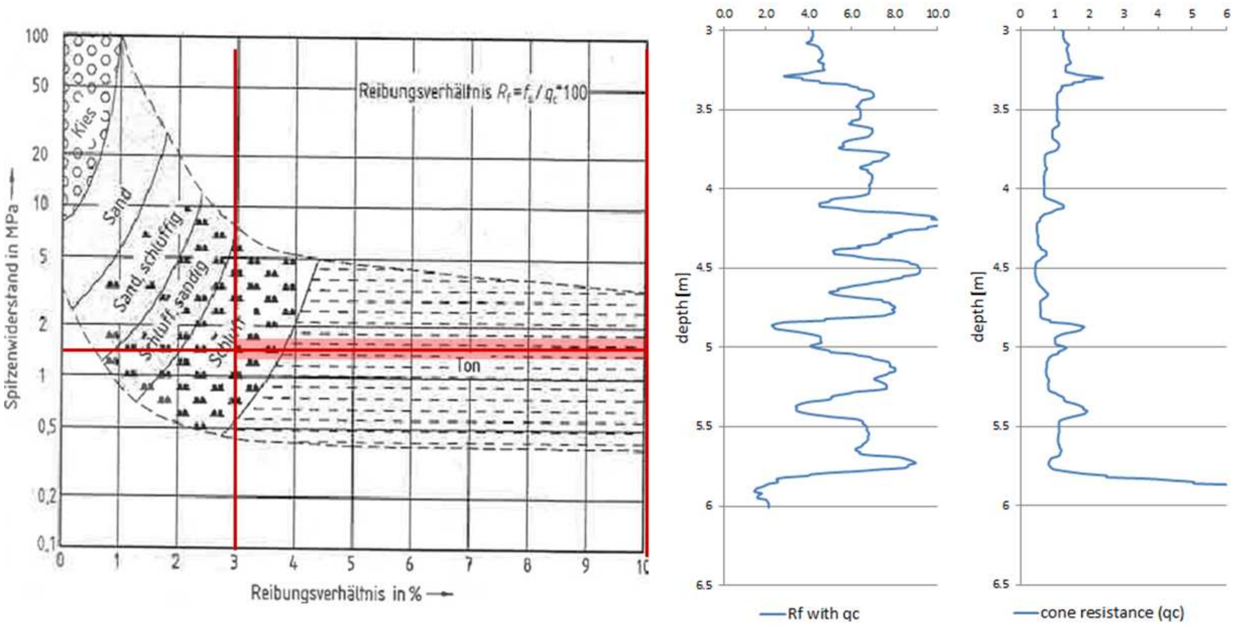


Figure 98: Soil type chart in DIN 4094-1 Appendix C 1, with a friction ratio value between ~3- 10% and a q_c of about 1,4MPa.

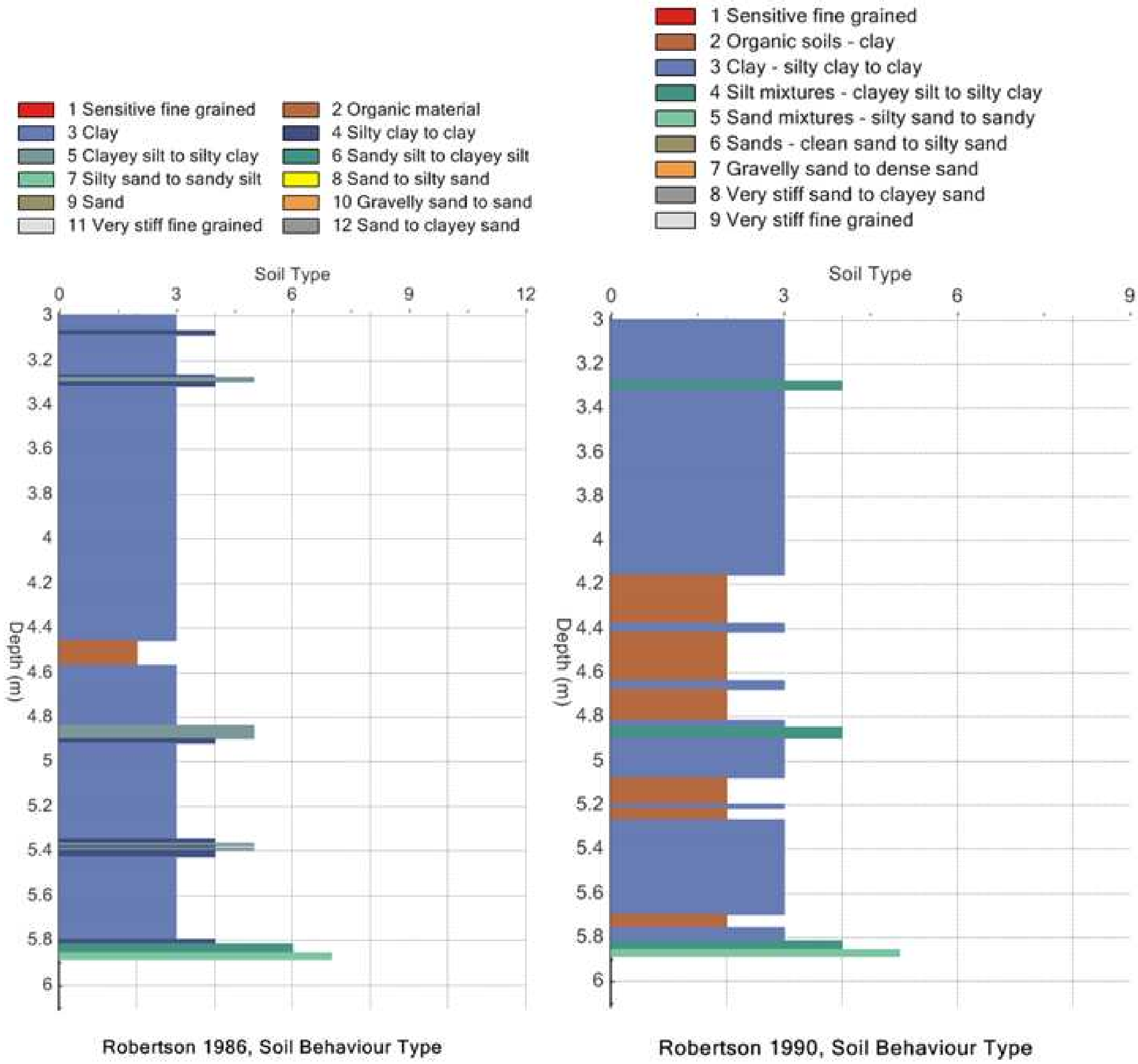


Figure 99: Soil type classification using Robertson 1986 and Robertsom 1990 (Project 8)

2.14 Project 9

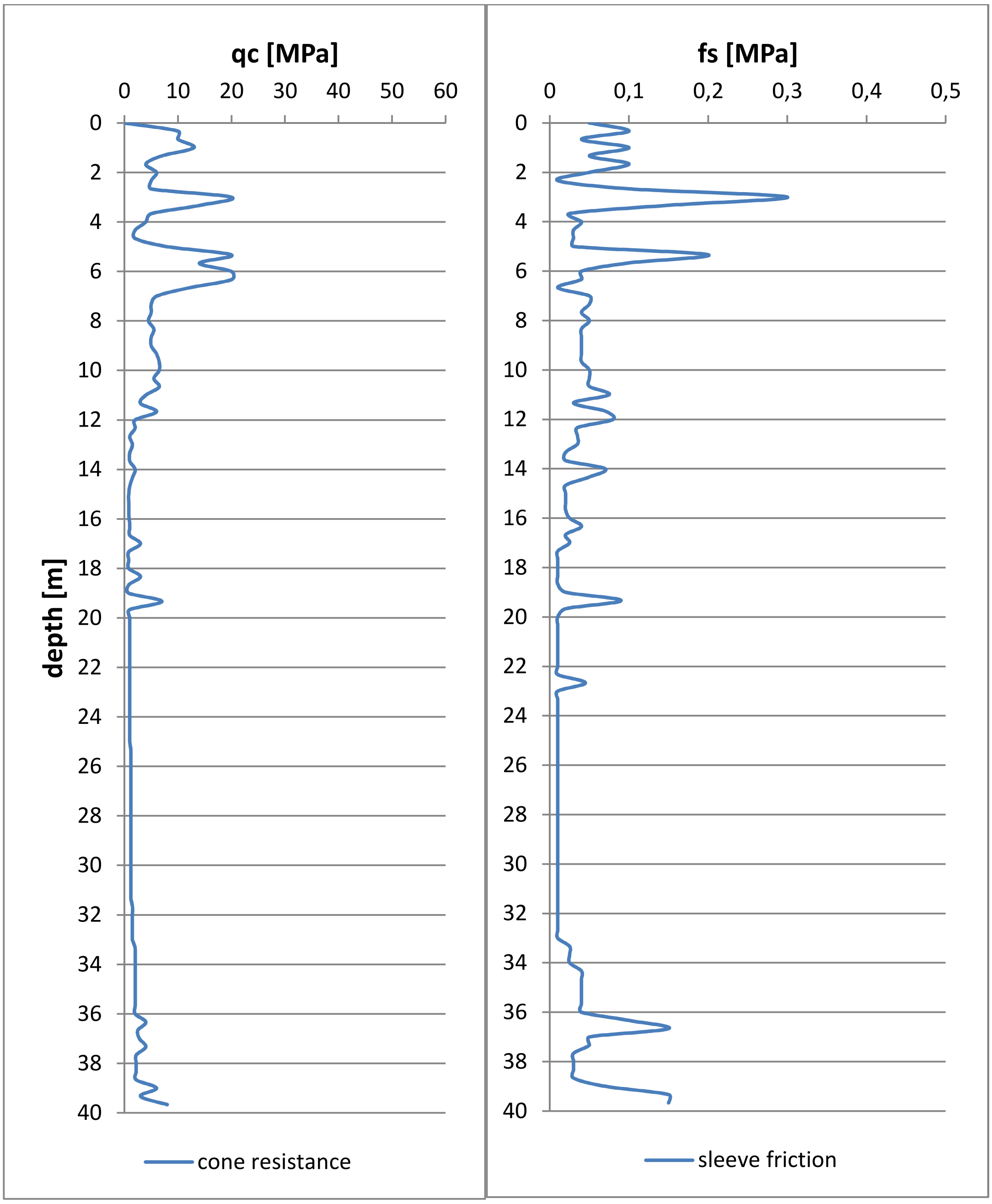


Figure 100: Input parameters for project 9;

For the estimation of the mean constrained modulus, the following values have been chosen:

Table 16: Assumed α - values for the mean constrained modulus (DIN 4094-1, D- 6)

Layer	α_{min}	α_{max}
Anschüttung	5	
schotter	6.00	
Feinsand/ Schluff	3,5	2
Seeton	2	6

Table 17: layers, description and geotechnical parameters

Layer	Depth [m]	Soil description	φ´ [°]	Es [MPa]	c_u [KPa]
Anschüttung	0-5	Inhomogen: Kies sand Schluff	30	---	0
Schotter	5-7	Kies sandig schluffig	35	30- 40	0-10 ("je nach Zustand")
Feinsand/ Schluff	7- 19,3	Feinsand/ schluff	22-27	5- 15	0- 5 ("je nach Zustand")
Seeton	19,3- 33	Schluff, gernig Tonig	22,5	Ca. 10 *("im entwässerten Zustand")*	0- 10

2.14.1 Estimation of Es

For the first 5 meters, information on the constrained modulus is missing, which may be difficult to estimate because of building waste that was found in that area (geotechnical report).

In greater depths, results from the geotechnical report fit best to the MIN values of the mean constrained value given in the DIN 4094- 1; D-6. The Maximum value of the mean constrained modulus and the stress- dependant constrained - modulus, as introduced in the DIN 4094- 1 D-5, result in much higher values in depth of 5 to 11m.

Also, correlations from Robertson 2009 and Mayne 2001, show higher values in lower depth but match with results of the geotechnical report in greater depths.

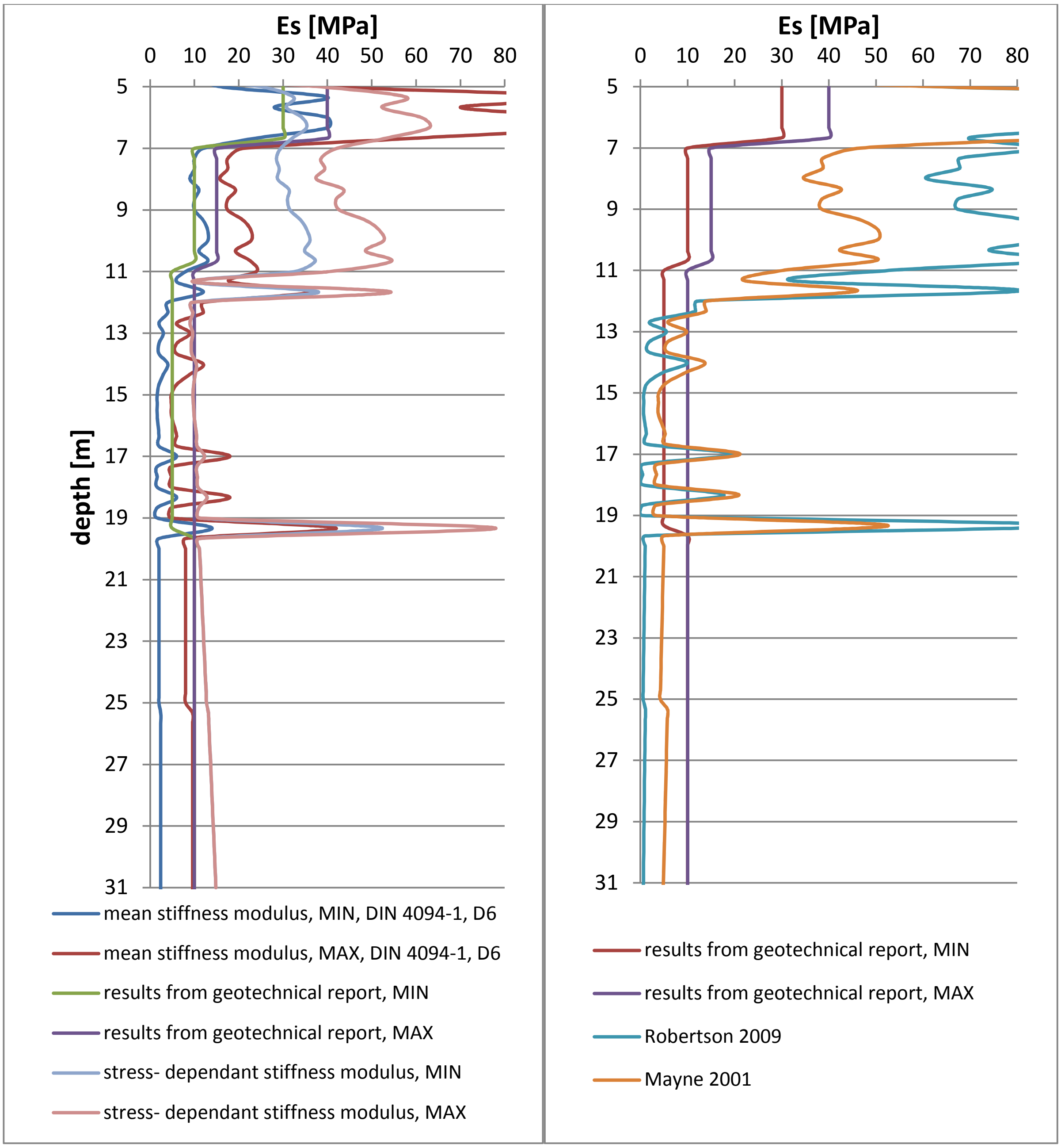


Figure 101: Correlations for Es, data: project 9;

2.14.2 Estimation of c_u

There is no information available for the undrained shear strength in the "Seeton" layer, but a Plasticity Index I_P, which is within a range of 10- 15%, is provided. However, one can compare the results using theoretical (Vesic 1975 and Baligh 1975) and empirical approaches (Aas et al. 1986) using the total cone resistance. In this project the two other approaches (effective cone resistance and excess pore pressure) are not available because the CPT was conducted without additional measurement of the dynamic pore pressure (u_2).

For the theoretical solution, IR got linked with I_P and OCR (Powell et al. 1988: OCR~1) after Keaveny and Mitchells´ (1996) diagram which give a value of IR~250 (compare Figure 102) and result in a N_{kt}-value of 11,25 for Vesic and 17,5 for Baligh.

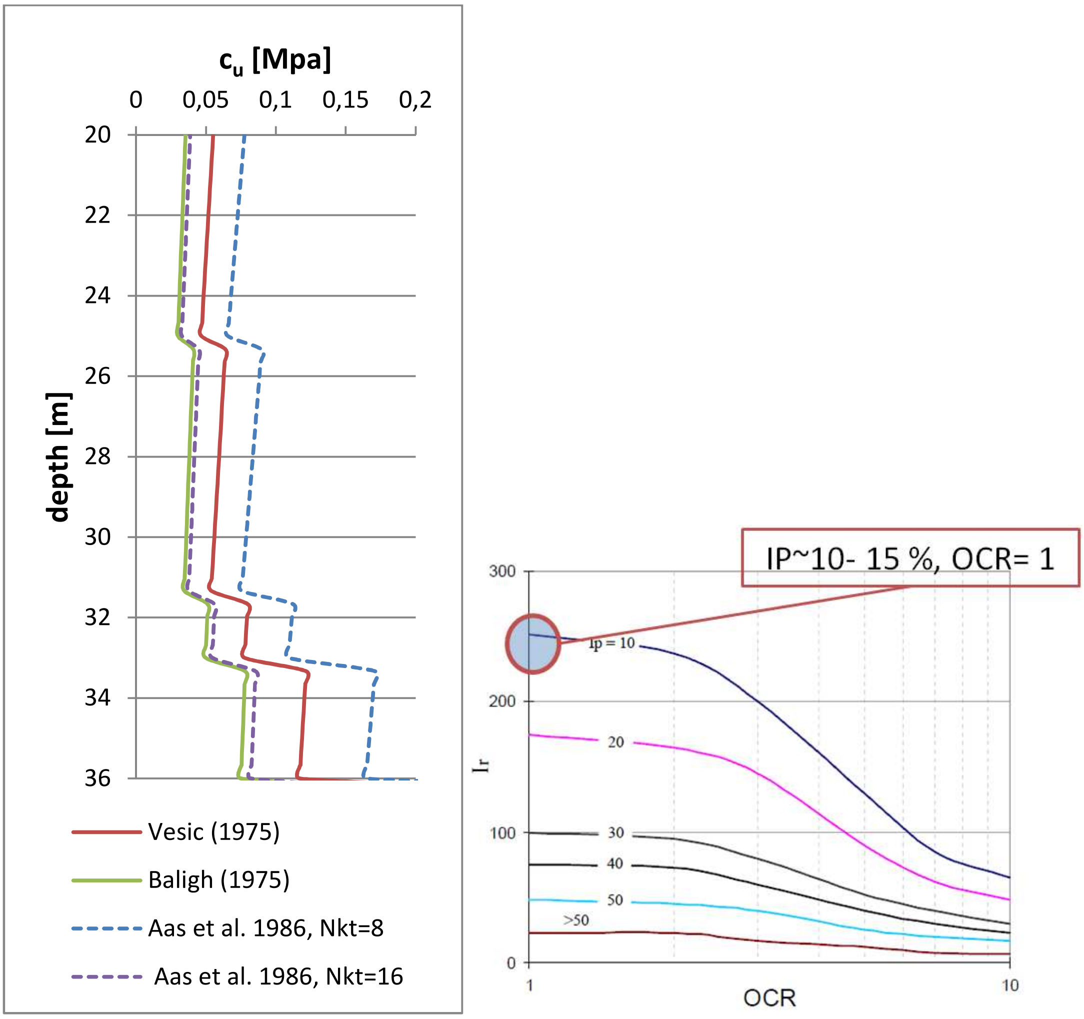


Figure 102: left: theoretical and empirical c_u correlations, determination of IR after Keaveny and Mitchells (1996)

2.14.3 Estimation of φ´

In that project, closer investigations on the estimation of φ´ in clays will be provided. Two approaches are provided by Senneset et al. 1989 and Mayne 2006 for correlating φ´ in clays.

The correlation of Senneset et al. 1989 shows values up to 15° less than those recommended in the geotechnical report, whereas Mayne 2006 doesn´t provide any results, which is due to certain requirements on the application of that correlation (values from NOVOCPT software, compare Figure 103).

For the sake of completeness, results for Sands (depth 0- 11m) are shown in Figure 103.

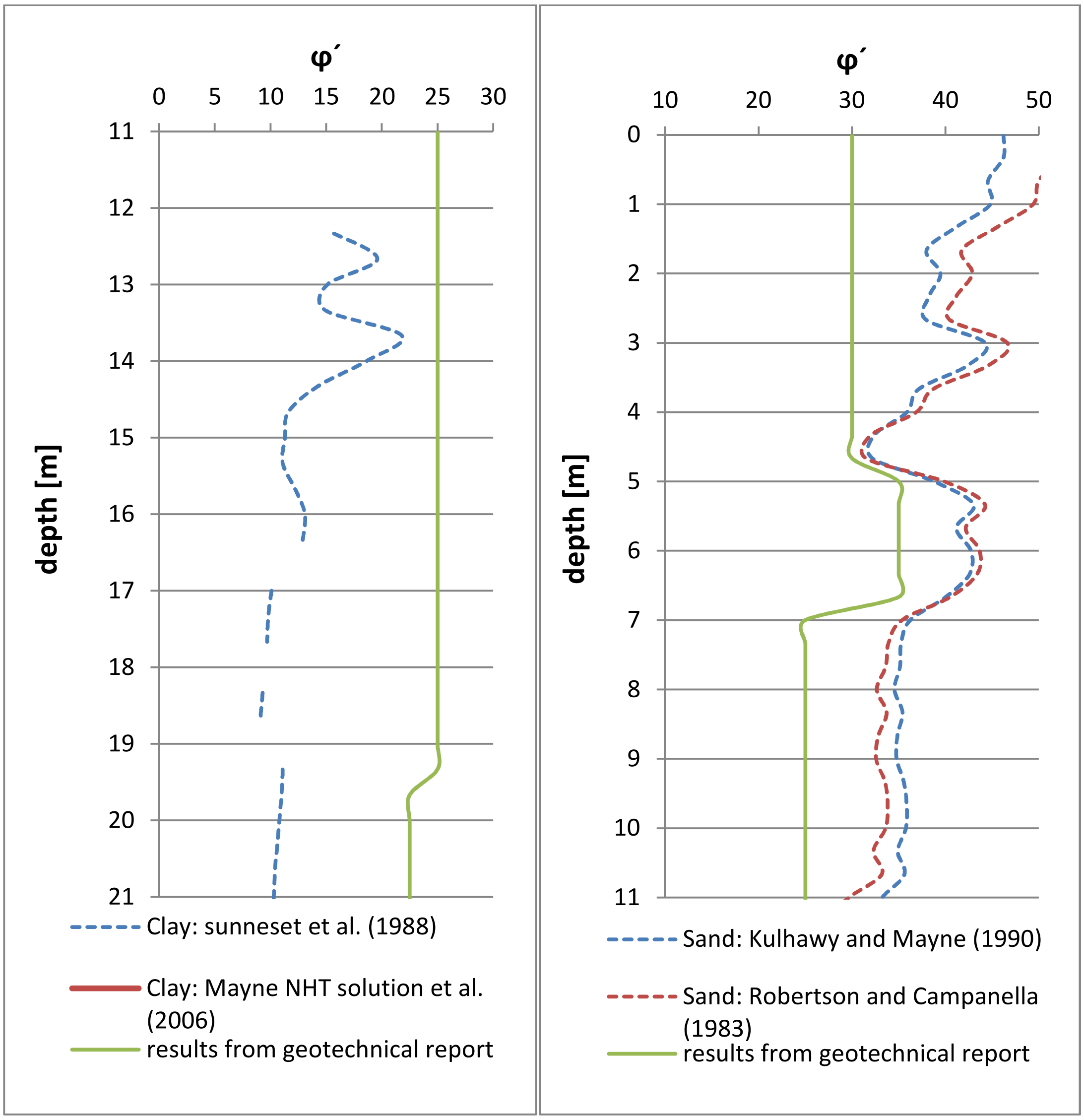


Figure 103: Correlation on φ´, data: project 9;

2.15 Literature on CPT

2.15.1 Papers

Schnaid F., In-situ Testing in Geomechanics: The Main Tests, Taylor & Francis, New York, 2009;

Schnaid F., Geo- characterisation and properties of natural soils by in situ tests Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering, Osaka, Japan, 12-16 September, 2005, 2006 pp. 3-45;

Robertson P. K. and Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 4th Edition, 2010;

Meigh A. C., Cone penetration testing: methods and interpretation, Butterworths, CIRIA, 1987;

Simonini, P., Ricceri, G. & Cola, S. 2007. Geotechnical characterization and propoerties of the Venice lagoon heterogeneous silts. Proc. 2nd International Workshop on Characterization and EngineeringProperties of Natural Soils, Singapore, November 2006: 2289-2327.

Lunne T., Robertson P. K., Powell J.J.M., Cone penetration testing in geotechnical practice, Blackie Academic/Routledge Publishing, New York, 1997;

Kim K., M. Prezzi, R. Salgado, Interpretation of Cone Penetration tests in Cohesive Soils; Joint Transportation Research Program Technical, Report Series, Purdue University, 2006;

Fellenius, B. H., and Eslami, A., Soil profile interpreted from CPTu data. “Year 2000 Geotechnics” Geotechnical Engineering Conference, Asian Institute of Technology, Bangkok, Thailand, November 27 - 30, 2000;

Schaap, L. H. J. and Zuidberg, H. M., mechanical and electrical aspects of the electric cone penetrometer tip, proceedings oft h 2nd European Symposium on penetration testing, ESOPT- II, Amsterdam, 2, 841-51, Balkema Pub. , Rotterdam, 1982;

Mayne P. W., Christopher B. R., and DeJong J., Manual on Subsurface Investigations, National Highway Institute Publication No, FHWA NHI-01-031, Federal Highway Administration, Washington, DC, 2001;

Powell J J M, Session Report 1: CPT Equipment & Procedures, 2nd International Symposium on Cone Penetration Testing, California, USA, May 2010;

Long M., Regional report for Northern Europe, CPT Equipment & Procedures, 2nd International Symposium on Cone Penetration Testing, California, USA, May 2010;

Butcher A.P., Campanella R.G., Kaynia A.M., Massarsch K.R., Seismic cone downhole procedure to measure shear wave velocity - a guideline prepared by ISSMGE TC10: Geophysical Testing in Geotechnical Engineering, International Society for Soil Mechanics and Geotechnical Engineering,

Mayne P. W., NCHRP Project 20-05 Topic 37-14 Cone Penetration Testing State-of-Practice, 2007;

Jacobs P., CONE PENETRATION TESTING (CPT), Simplified Description of the Use and Design Methods for CPTs in Ground Engineering, Fugro, Feb. 1996;

Poulsen R., Nielsen B. N., Ibsen L. B., Effect of drainage conditions on cone penetration testing in silty soils, Pan- Am CGS Geotechnical Conference, 2011;

Robertson P.K., Interpretation of CPT – a unified approach, Gregg Drilling & Testing Inc., California, Canadian Geotechnical Journal, May, 2009;

Robertson P. K., Soil classification using the cone penetration test, Canadian Geotechnic journal, Vol. 27, 1990, pages: 152- 158;

Robertson P.K., Cabal K.L., Estimating soil unit weight from CPT2nd International Symposium on Cone Penetration Testing, CPT'10, TC-16 of ISSMGE, USA, 2010,

Schneider J.A., Session Report 2: CPT Interpretation, 2nd International Symposium on Cone Penetration Testing, CPT'10, TC-16 of ISSMGE, USA, 2010;

Młynarek Z., Wierzbicki J., Stefaniak K., CPTU, DMT, SDMT results for organic and fluvial soils, 2nd International Symposium on Cone Penetration Testing, CPT'10, TC-16 of ISSMGE, 2010;

MAYNE P. W., Cone Penetration Testing, NCHRP SYNTHESIS 368, A Synthesis of Highway Practice, National Cooperative Highway Research Program, 2007 Transportation Research Board;

Hutchinson, J. N., Reading the ground: morphology and geology in site appraisal. Quarterly Journal of Engineering Geology and Hydrogeology 34 (1) (2001), S. 7–50.

Mayne P.W., Peuchen J. and Bouwmeester D., Soil unit weight estimation from CPTs, 2nd International Symposium on Cone Penetration Testing, CPT'10; California, 2010;

Liao T. & Bell K.R., CPT in glacial soils after deep excavation, 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, USA, May 2010;

Bellotti R., Ghionna V.N., Jamiolkowski M., and Robertson P. K., Design parameters of cohesionless soils from in-situ tests, in Proceedings of Transportation, Research Board Conference, Washington, 1989;

Kulhawy F.H., Mayne P.W., Manual on estimating soil properties for foundation design, Report EL-6800, Electric Power Research Institute, Palo Alta, CA, August: 306, 1990;

Mayne P.W., Overview Paper: In-situ test calibrations for evaluating soil parameters, Characterisation & Engineering Properties of Natural Soils, Vol. 3 (Proc. Singapore 2006), Taylor & Francis Group, London: 1602-1652, 2007;

Landva A., Korpijaakko E.O., Pheeney P.H., Geotechnical Classification Of Peats And Organic Soils, Testing of Peats and Organic Soils: A Symposium, 1983;

Powell J.J.M., Quarterman R.S.T. & Lunne, T., Interpretation and use of the piezocone test in UK. Penetration testing in the UK, Thomas Telford, London, 47-52, 1988;

Jaeger R.A., DeJong J.T., Boulanger R.W., Low H.E., Randolph M.F., Variable penetration rate CPT in an intermediate soil, 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, USA, May 2010;

Rix G.J., Stoke K.H., Correlation of initial tangent modulus and cone resistance, Proceeding of Int. Symposium on Calibration Chamber Testing, pages: 351-362, 1992;

Witt K. J., Grundbau-Taschenbuch, Teil 1: Geotechnische Grundlagen, Ernst & Sohn, 7. Auflage, Berlin 2008;

Clayton C. R. I., Matthews M. C., Simons E., Site Investigation, Department of Civil Engineering, University of Surreysecond edition, September 6, 1995;

Butcher A. P., Campanella R.G., Kaynia A.M. and Massarsch K. R., Seismic cone downhole procedure to measure shear wave velocity - a guideline prepared by ISSMGE TC10: Geophysical Testing in Geotechnical Engineering, 16th Internat. Conference on Soil Mechanics and Geotechnical Engineering, Osaka, 2005;

Robertson, P.K., Campanella R.G., Gillespie D. and Rice A., Seismic CPT to Measure In-Situ Shear Wave Velocity, ASCE, Journal of Geotechnical Engineering, Vol. 112, No. 8, August 1986, pp. 791-804, 1986;

Elkateb T.M., Ali H.E., CPT-SPT correlations for calcareous sand in the Persian Gulf area, 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, USA, May 2010;

2.15.2 software manuals

USER GUIDE, Datgel CPT Tool gINT Add-In 2.2, DCT-UG-001 - 2.12.2 November 2011, (http://www.datgel.com/pdfs/UserGuide/DCT-UG-001_2.12.2_Datgel_CPT_Tool_User_Guide.pdf)

2.15.3 Standards and Guidelines

EN ISO 22476- 1, Geotechnical investigation and testing -- Field testing -- Part 1: Electrical cone and piezocone penetration test

DIN 4094- 1, 2002- 06 Baugrund - Felduntersuchungen - Teil 1: Drucksondierungen;

ASTM D 5778- 07, Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils;

ASTM D 3441- 05, Standard Test Method for Mechanical Cone Penetration Tests of Soil;

BS 1377 Part 9, 1990, Soils for engineering purposes- CPT;

SGF Report 1:93 E, Swedish standard for cone testing, Swedish Geotechnical Society;

ISSFME 1989, International reference test procedure for cone penetration test (CPT)

EN 1997-2:2010: Entwurf, Berechnung und Bemessung in der Geotechnik Teil 2: Erkundung und Untersuchung des Baugrunds;

NEN 5140- 1996, Geotechnics, Determination of the cone resistance and the sleeve friction of soil, Electric penetration test;

3. Standard Penetration Test

3.1 Introduction

Throughout the world, the Standard penetration test (SPT) is the most popular and cost effective Insitu test to get relevant information about the subsoil. It is the oldest and most frequently used test for geotechnical exploration.
The test method describes the procedure for driving a split-barrel sampler to obtain a representative disturbed soil sample for identification purposes, and measure the resistance of the soil to penetration of the sampler (ASTM D1586 – 11, page 1). The SPT is conducted at the bottom of a soil boring. At regular depth intervals, the drilling process is interrupted to perform the SPT. Generally, tests are taken every 0.76 m at depths shallower than 3 meters and at intervals of 1.5 m thereafter. The head of water in the borehole must be maintained at or above the ambient groundwater level to avoid inflow of water and borehole instability (Mayne et al. 1990).
The Test- setup and the performance is described in a different section because of the varying equipment and process.
General information on procedures and equipment of SPT will be briefly covered. However, the focus will be on problems like standardisation and energy transfer.

On the correlation of SPT results, the EN ISO 22476- 3 mentions that the SPT will be mainly used to determine the parameter of cohesionless soils. Nonetheless, important data of other soil types may be obtained. DIN 4094-2 states that in coarse- grained soils correlations between SPT results and geotechnical parameters can be derived. For fine grained or mixed soil types with major cohesive parts, the evaluation of the results should only be performed with good knowledge of the local conditions.

3.1.1 Problem of standardisation

First attempts to standardize the SPT were made in the early 1930 in the USA. The SPT is still today the most common used Insitu test for bearing capacity and stabilisation investigations. The most acknowledged and cited standards are the ASTM 1586 for Northern America and the BS 1377 for the UK. Besides that countries, Australia, Brazil, India or Japan have developed their own standards for SPT. The first international Harmonisation for SPT was succeeded by the Technical Committee TC 16 of the ISSMFE with the document: "International Reference Test Procedure for SPT". The IRTP was used as a base for the EN ISO 22476-3. For reasons of comparison, the German standard DIN 4094-2 ("Bohrlochrammsondierungen") will be introduced. Moreover, this paper will cover the differences within the standards mentioned above.

3.1.2 Advantages and disadvantages of SPT

The advantages of the SPT are that it is relatively quick and simple to perform and it is widely available. It is relatively cheap and provides, with one procedure, both a (disturbed) sample (Note: according to the German standard DIN 4094-2 no sample will be obtained) and a soil test result. The SPT also provides a useful index of the relative strength and compressibility of the soil. In addition, the test is able to penetrate through relatively difficult materials such as dense layers, gravels, and fills.

The disadvantage of the SPT is that it has many sources of error, both random and systematic. The accuracy of the test is in large parts dependent on the details of the procedure followed and the equipment used by the drilling crew, so that knowledge of the drillers form a critical factor in the test accuracy (compare Kulhawy& Mayne 1990).

3.2 Test- setup

As there are some variations in the test setup the following Standard Penetration Test setup will be described matching the IRTP, 1989 (variations will be noted).

The SPT consists of dropping a hammer weighing 63, 5 kg on to a drive head from a height of 760mm. The number of blows (N) necessary to achieve a penetration by a sampler of 300mm (after its initial penetration under gravity and below a seating drive of 150mm) is regarded as the penetration resistance. If the 300mm penetration can´t be achieved by 100 blows the test has to be terminated (IRTP for SPT, 1989). Note: The BS 1377- 9 suspends the test already after 50 blows whereas the EN ISO 22476- 3 mentions that the number of blows can be increased from 50 to 100 in weak rock.

3.2.1 Equipment

Not just the procedure is far from standardized- also a complete classification of SPT systems is difficult to find in literature.

SPT equipment consists of 3 main components:

- split spoon (sampler)
- rods
- hammer

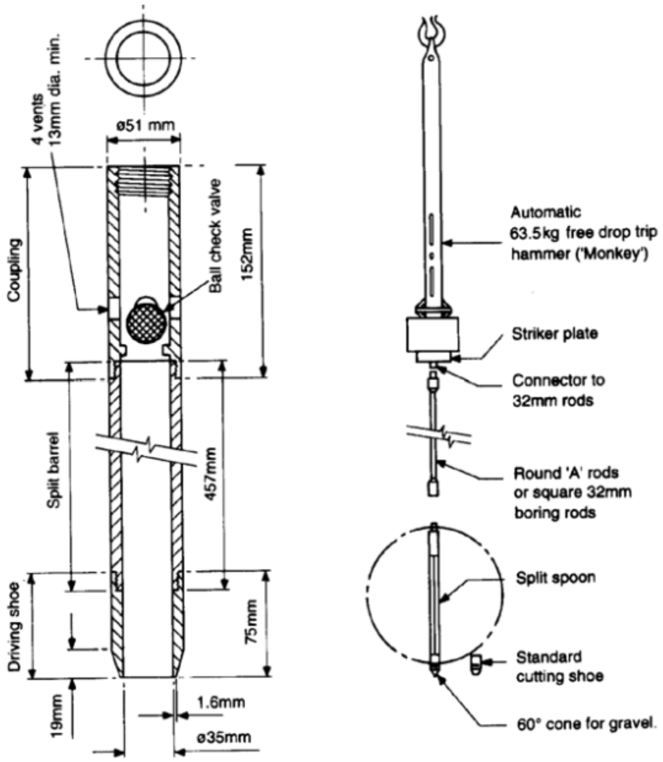


Figure 104: Equipment for standard penetration test (Clayton et al 1995)

The steel rods connect the spilt spoon (sampler) with the hammer. The left figure shows an example of the sampler (Dimensions in accordance with the IRTP for SPT).

3.2.2 Hammer- system

As the main differences on energy transfer result from the type of hammer used, it will be described in a separate section. Only the American standard introduces two hammer- systems (donut hammer and safety hammer) that are widely used in geotechnical practise. Whereas, almost all other standards don´t even mention any systems available. The IRTP 1989 just mentions the weight and the tolerance (63,5±0,5 kg) of the hammer. It gives further information about the drive head but doesn´t specify any other details ("suitable design"). Hence, various hammer- systems introduced by Clayton et al. 1995 will be presented. Most other papers introduce mainly American- orientated systems.

Clayton et al. 1995 classifies SPT- hammers- ystems in four main categories and subdivides the slip rope hammer in three more subcategories:

<u>Automatic trip hammers</u>: Automatic trip hammers (Fig. 9.1), which are standard in the UK, are also used in Israel, Australia and Japan. This is the best type of hammer, because the energy delivered per blow is consistent. Tests on a Dando automatic trip hammer have been conducted which gave an average energy of 73% of the free-fall energy, with a standard deviation of only 2.8%.

<u>Hand-controlled trip hammers</u>: Hand-controlled trip hammers are not widely used. The weight is lifted by hoist and then tripped to give a free fall. Inconsistencies can occur if the driller is careless in assessing how high to lift the weight.

Slip-rope hammers: Slip-rope hammers are widely used over much of the world, including the USA, Japan, and South America. Common types of slip-rope hammer are the safety hammer, pinweight hammer and the donut hammer. The weight is lifted by a rope which passes over a sheave on the top of the mast of the drilling rig, and is pulled via a cathead. To deliver consistent energy the operator not only has to lift the weight repeatedly to the correct height, but also has to release it from the cathead in a consistent manner. This is extremely difficult. Energy is lost in friction as the rope slips over the rotating cathead, and also as the sheave is turned. The amount of energy lost on the cathead depends upon its condition, and how many turns of rope the operator uses.

Hand-lifted hammers: Hand-lifted hammers are almost identical today to those used in the USA in the 1920s and 1930s, which were described above. They are not widely used, except in relatively undeveloped countries (Clayton et al. 1995).

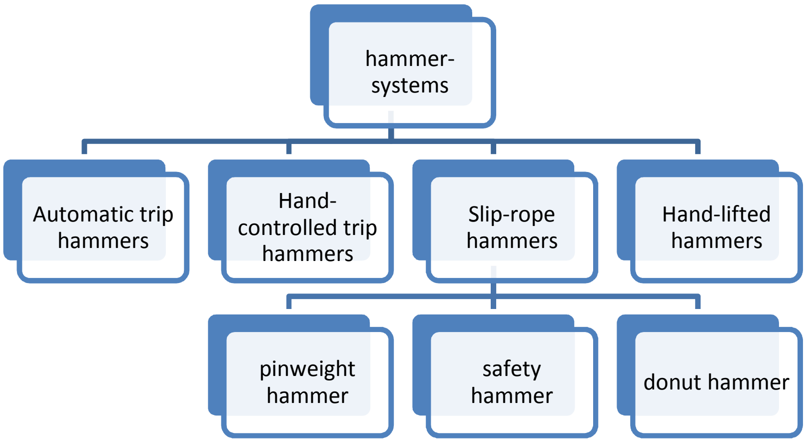


Figure 105: Hammer- systems in use (Clayton et al 1995);

The most used types of hammer can be seen in Figure 106:

- donut hammer,
- safety hamme,
- automatic hammer;

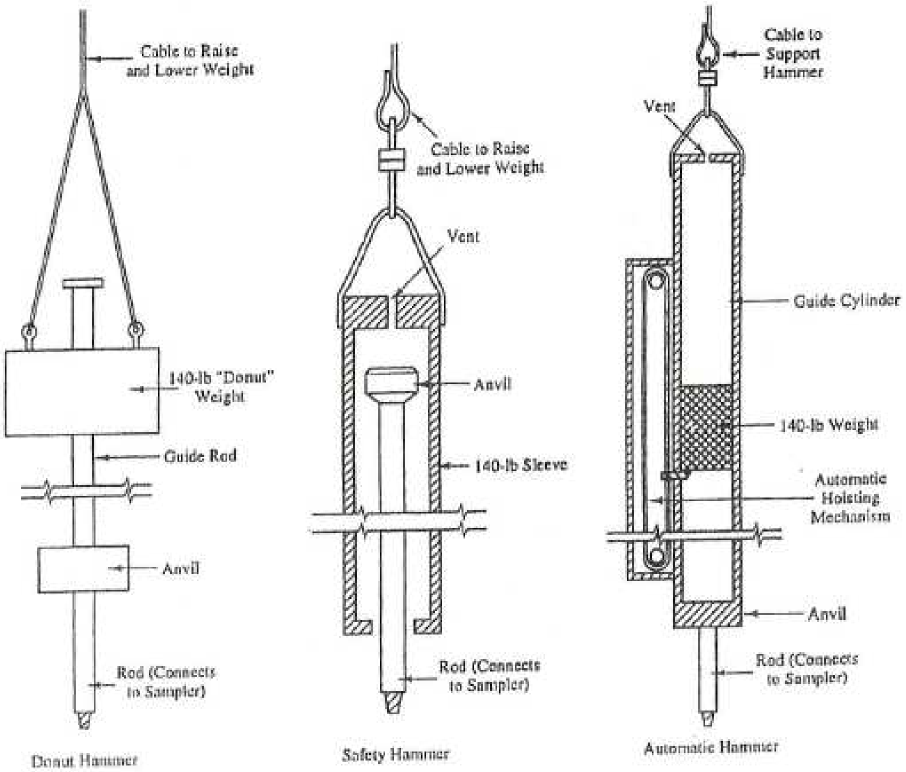


Figure 106: donut hammer, safety hammer and automatic hammer[3]

3.2.3 IRTP for SPT- and comparison to other standards

There are significant differences between the drilling techniques, SPT equipment and test procedures used in different countries. According to Clayton et al (1995), these differences have arisen because of adaptation to local ground condition, because of the relative difficulty to access the borehole sites, and because of the degree of sophistication of drilling and boring plant. They can have a major influence on the result of a Standard penetration test.

Generally, the SPT is widely dominated by the UK and the US. As a result numerous literature is available and often refers to their standards.

In Central European countries, the SPT still doesn´t receive widespread recognition in estimation soil properties. That is also reflected by standards available for SPT. Before implementing the EUROCODE, Austria had no specific standard for SPT. Germany offers a standard on "*Bohrlochrammsondierungen*", that is very similar to the Standard Penetration Test, but without getting a disturbed sample of the subsoil (details below).

Taking the IRTP 1989 as a reference, derivations are stated below:

- The internal diameter of the barrel of the sampler being 3 mm larger than the standard 35mm internal diameter of the drive shoe (USA).

[3] http://www.scribd.com/doc/48056653/SoilCharacterization

Note: The new ASTM D1586-11 advises that the N-values may differ as much as 10 to 30 % between a constant inside diameter sampler and upset wall sampler and refer to Practice D6066 for correction the upset wall sampler.
Also, the EN ISO 22476-3 (page: 5) also allows a 3mm great internal diameter for an optional installation of a liner but doesn´t provide a guideline to correct that factor.

- The use of a solid steel cone in lieu of the standard drive shoe for tests in gravel and weak rocks (UK, FRG and Australia).
  Note: The European standard allows the use of a 60° solid cone instead of drive shoe for gravelly sands (compare EN ISO 22476- 3, page: 5)
- Furthermore in the German standard (DIN 4094-2 “Bohrlochrammsondierungs”) the SPT doesn´t include sampling of soil at all. The drive shoe consists of a solid closed apex- cone with a diameter of 50,5mm.

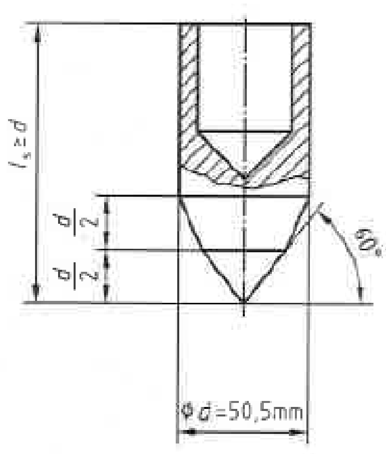


Figure 107: Design of conus (prDIN 4094-2)

3.2.4 Effects on the N- value (energy transfer)

While performing a Standard Penetration Test difficulties can occur concerning the interpretation of the results.

It is important to know what factors influence the N value. A very detailed list is provided by Sherif Aggour& Rose Radding 2001.

Schmertmann (1978) and Kovacs und Salomone (1982) and other authors identify that the most significant factor affecting the N value is the amount of energy delivered to the rods. They indicated that the energy delivered to the rods during an SPT test can vary from 30 to 80 % of the theoretical maximum (Sherif Aggour& Rose Radding 2001).

Energy delivery is a very important topic that has been discussed and analysed a lot in respective literature.

Chapter *Correction factor proposed by different authors*, will present ranges of proposed values by different authors. In advance, the factors that influence the energy transfer to determine the corrected N- value, will be outlined.

As already mentioned, the major components of test apparatus are the split spoon, the rods and the hammer with its influence on the energy transfer (Clayton 1995):

- Hammer design
  - Energy delivered to anvil by automatic trip hammers, handcontrolled trip hammers, slip- rope hammers and handlifted hammers,
  - Influence of anvil mass,
  - Influence of wooden cushion blocks;
- Rods
  - Rod whip in flexible and bent rods,
  - Length of rod string,
  - Rod weight and stiffness,
  - Energy losses in loose couplings;
- Split- spoon design
  - Use of catchers,
  - Lack of linear in US larger i. d. samplers,
  - Vent area and ball- check valve details,
  - Penetration resistance of the cone compared to the shoe;

According to Clayton et al. 1995 the split spoon has few impact on the energy delivered, whereas the hammer- system has the most influence on the energy transferred.

It is already common knowledge that the N value has to be attenuated due to energy losses. The IRTP for SPT, 1989 recommends the N value to be adjusted to 60% of the nominal kinetic energy of 474 Joules. But the problem is that the exact value of the energy transferred to the rods is not known and varies a lot- mainly due to the different SPT hammer- systems in use.

3.2.5 Correction factors proposed by different Authors

In literature an abundance of correction factors for energy transfer can be found. As a result of that most corrections are made on a basis of the already "corrected" value (60% of nominal kinetic energy).

The intention of this chapter is to give an overview and not to value the proposed factors with its ranges.

The most general equation to describe energy losses is:

$$N_{60} = N_f * n_1 * n_2 * n_3 * n_4 * n_5 * n_6$$

It includes correction factors for the equipment in use (no overburden corrections).

Note: In the appendix in EN ISO 22476-3 a similar but simplified equation is introduced to describe energy losses (will be introduced later on).

For the additional correction of the overburden pressure one term (C_N) must be added to the equation:

$$N_{60} = N_f * n_1 * n_2 * n_3 * n_4 * n_5 * n_6 * c_N$$

With:

N_f= measured value in the field,
n_1=hammer energy correction factor,
n_2= rod length correction factor,
n_3= liner correction factor (sampler with/without liner),
n_4= borehole diameter correction factor,
n_5= anvil correction factor,
n_6= blow count frequency correction factor,

C_N= overburden correction factor,

The overburden correction factor will be introduced separately of the factors.

3.2.5.1 Overburden correction factor

In order to compare blow counts in different depth, measured blow counts should be adjusted to a standard overburden pressure of 100KN/m². The penetration resistance of cohesionless soils depend heavily on the confining pressure. For instance, for the same sand, a SPT performed at a shallow depth will have a lower blow count than for an SPT performed at a great depth (MGregor et al. 1998).

In addition since the SPT brings the soil to failure, and because the strength of granular soil will be strongly dependent on effective stress level, it will be necessary to correct 'N' values (Clayton et al. 1995).

Schnaid 2009 summarises overburden correction factors proposed by different authors (compare Figure 108). The EN ISO 22476-3 also proposes the use of an overburden correction factor. In the appendix, the use of the correction factor proposed by Skempton (1986) is recommended with clear limits set for the relative density (see Table 18).

Skempton (1986)	$C_N = \frac{200}{100 + \sigma'_{v0}}$	kPa	Seed *et al.* (1983) D_r = 40–60% NC sand
Skempton (1986)	$C_N = \frac{300}{200 + \sigma'_{v0}}$	kPa	Seed *et al.* (1983) D_r = 60–80% NC sand
Peck *et al.* (1974)	$C_N = 0.77 \log\left(\frac{2000}{\sigma'_{v0}}\right)$	kPa	NC sand
Liao and Whitman (1985)	$C_N = \sqrt{\frac{100}{\sigma'_{v0}}}$	kPa	NC sand
Liao and Whitman (1985)	$C_N = \left[\frac{(\sigma'_{v0})_{ref}}{\sigma'_{v0}}\right]^k$	–	k = 0.4–0.6
Skempton (1986)	$C_N = \frac{170}{70 + \sigma'_{v0}}$	kPa	OC sand OCR = 3
Clayton (1993)	$C_N = \frac{143}{43 + \sigma'_{v0}}$	kPa	OC sand OCR = 10
Robertson *et al.* (2000)	$C_N = \left(\frac{\sigma'_{v0}}{\sigma_{atm}}\right)^{-0.5}$	kPa	NC sand

Figure 108: Overburden correction factors (Schnaid 2009);

Table 18: Korrekturfaktor CN für einen wirksamen Überlagerungsdruck σv´ bei Sanden (EN ISO 22476- 3);

Sandart	Bezogene Lagerungsdichte ID [%]	Korrekturfaktor [CN]
Normal belastet	40- 60	$\frac{200}{100 + \sigma_V{}'}$
	60- 80	$\frac{300}{200 + \sigma_V{}'}$
vorbelastet	-	$\frac{170}{700 + \sigma_V{}'}$

3.2.5.2 Additional correction

Incorporating correction due to groundwater in sands, according to Melzer 1971 and DIN 4094-2:

Melzer 1971 conducted a study on the measurement of relative density and compared the N- values above and in groundwater. He concluded for the same relative density, N values above the groundwater level have been higher than in groundwater. He proposed a linear relationship that corrects the N values in groundwater with those above and got incorporated in the German Standard (DIN 4094-2). Melzer 1971 argues that the number of blows is smaller below ground water than above because the effective unit weight of the sand in a submerged state is smaller than a dry or wet state. Furthermore, the result of the dynamic action of the penetrating sampler causes a quick sand effect, at least in very loose to medium-dense sands, resulting in decreased penetration resistance. Therefore, the number of blows measured below the ground-water level in medium and coarse sands should be corrected for these influences before an estimate of relative density is made.

DIN 4094- 2 states: ”In coarse grained soils, the penetration resistance in the same soil is lower in the groundwater, than above the groundwater table, whereas for fine grained soils the N_{10} is similar or shows even higher N10 values due to the capillary actions.” It further provides the correlation for sands:

Uniformed-graded sands (U≤3, 3≤Nk≤50)

SPT:

$$N_{30ü} = 1{,}1 * N_{30u} + 5$$

Well graded gravel- sand mixtures (U≥6, 3≤Nk≤50)

SPT:

$$N_{30ü} = 1{,}1 * N_{30u} + 5{,}9$$

Besides the DIN 4093-3, all other standards like the EN, BS or the IRTP from the ISSMFE don´t provide any information about corrections in groundwater.

NOVOSPT similarly doesn´t take that effect of groundwater (proposed in DIN 4094-2) into account, but provides another correction for saturated fine and silty sands, to account for dynamic pore pressure effects (Meyerhof 1965):

$$N = 15 + \frac{(N' - 15)}{2} \; for \; N' > 15$$

Where: N´=measured blow count,

N=corrected blow count;

3.2.5.3 Correction factors depending on the equipment used

Here, correction factors depending on the equipment are listed in Table 19 (excluding the overburden correction factor):

Table 19: correction factor by parameter (SHERIF AGGOUR et al. 1998);

Length of Drill Rod		Robertson & Wride (1997)	Seed (1984) Per McGregor and Duncan (1998)	Bowles (1996)	Skempton (1986)
Length over 30 m	(+100 ft)	Less than 1	1	1	1
'10 – 30 m	(30–100 ft)	1	1	1	1
'6 – 10 m	(20–30 ft)	0.95	1	0.95	0.95
'4 – 6 m	(13–20 ft)	0.85	1	0.85	0.85
'3 – 4 m	(10–13 ft)	0.75	1	0.75	0.75
'0 – 3 m	(0–10 ft)	–	0.75	0.75	0.75

Corrections for Blow Rate (CBF)	Decourt, 1990 per McGregor and Duncan (1998)	
	Frequency of Hammer Blows	Bdf
Less than 20	10–20 blows/minute	0.95
Greater than 20	10–20 blows/minute	1.05

Anvil	Tokimatsu (1988) Per McGregor and Duncan (1998)	Skempton (1986)
Small (4.4 lbs)	0.85	0.7 – 0.8
Large (26.5 lbs)	0.7	0.6 – 0.7
Safety 5.5 lbs	0.9	0.7 – 0.8

Bore Hole Diameter	Robertson & Wride (1997)	Bowles (1996)	Skempton (1986)
Pamameter		N4	
60 – 120 mm	1	1	1
150 mm	1.05	1.05	1.05
200 mm	1.15	1.15	1.15

Sampler	Robertson & Wride (1997)	Bowles (1996)	Skempton (1986)
No linear	1.1 – 1.3	1	1.2
With liner: loose sand	1	0.9	1
With liner: dense sand, clay	1	0.8	1

Hammer Type	Seed (1984) per McGregor and Duncan (1998)	Robertson & Wride (1997)	Bowles (1996)
			n1 * =
Automatic	1.67	0.8 – 1.5	1.14 – 1.43
Pulley Safety Hammer	1	0.7 – 1.2	1 – 1.14
Donut	0.75	0.5 – 1.0	0.64

* where n1 = (Er/70) example for ER = 80% – 100% n1 = 1.14 – 1.43

It is apparent that the hammer type correction factor is the most significant value for the energy transfer. The energy delivered from the hammer depends on the way the hammer is lifted and released and on the design of the hammer. According to SHERIF AGGOUR et al. 1998 the energy transfer efficiency can be between 30% and 90%, depending on the type of hammer used.

Consequently, analysing the influence of the hammerassembly, Table 20 shows the range of published correction values for the main types of hammers (SHERIF AGGOUR et al. 1998).

Table 20: Range of published correction values (corresponding to the N60 value!);

Hammerassembly	donut	Safety	automatic
	0,5- 1,0	0,7- 1,2	0,8- 1,67

Another attempt of a guideline in selected countries is summarized in Clayton et al. 1995 by referring to further literature.

Country	Hammer type	Release mechanism	Average rod energy ratio (%)	Source references*
Argentina	Donut	Cathead	45	1
Brazil	Pin weight	Hand dropped	72	3
China	Automatic donut	Hand trip	60	1
	Donut	Dropped	55	2
	Donut	Cathead	50	1
Colombia	Donut	Cathead	50	3
Japan	Donut	Tombi	78-85	1,4
	Donut	Cathead, 2 turns + special release	65-67	1,2
UK	Automatic	Trip	73	5
USA	Safety	Cathead, 2 turns	55-60	1,2
	Donut	Cathead, 2 turns	45	1
Venezuela	Donut	Cathead	43	3

* (1) Seed *et al.* (1985); (2) Skempton (1986); (3) Decourt (1986), (4) Riggs (1986), (5) Clayton (1990).

Figure 109: Measured SPT- rod energy ratios (Clayton et al. 1995)

3.2.6 Note on energy transfer in IRTP and the EN

The IRTP for SPT, 1989 doesn´t determine the use of one specific hammerassembly. It states that the "hammer should drop with minimal resistance" and "a mechanical release mechanism which will ensure that the hammer has a constant free fall of 760mm has to be installed". The IRTP for SPT, 1989 further states: "The energy transferred on impact shall be maximised by a suitable design of drive head."

Additionally, the EUROCODE doesn´t pay much attention to the hammertype used. EN ISO 22476- 3 Appendix A provides a guideline to deal with the energy losses that is less accurate than the equation introduced above.

$$N_{60} = \frac{ER}{60} * \lambda * C_N * N$$

$\boldsymbol{\lambda}$ stands for the correction factor for the energy losses because of the drill rod length in sands (see Table 21)

It has been shown that when the length of the drill rods is less than 10m, a considerable amount of energy is reflected back in the rod reducing the energy available for driving the sampling tube into the ground, thus it is recommended that the N values should be corrected for short lengths of rods (Sherif Aggour et al. 2001).

E_R is the Energy ratio of the test equipment (examples see Figure 110). The measured energy ratio E_R is used to normalize the recorded SPT N values to an accepted standard efficiency of 60% (N60).

C_N is the already introduced correction factor for overburden pressure in sands.

N is the blow count measured in field.

Table 21: correction for rod length for sands (EN ISO 22476- 3; page 10)

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

Concerning energy delivery, the IRTP for SPT, 1989 quotes that "In situations where comparisons of SPT results are important, calibrations shall be made to evaluate the efficiency of the equipment in terms of energy transfer. In such cases N values shall be adjusted by calibration to a reference energy of 60 per cent of the nominal kinetic energy in a 63,5 kg hammer after free fall of 760mm namely 474 Joules. The reference energy shall be measured immediately below the drive head."

	Hammer	Release	ER_r: %	$ER_r/60$
Japan	Donut	Tombi	78	1·3
	Donut	2 turns of rope	65	1·1
China	Pilcon type	Trip	60	1·0
	Donut	Manual	55	0·9
USA	Safety	2 turns of rope	55	0·9
	Donut	2 turns of rope	45	0·75
UK	Pilcon, Dando,	Trip	60	1·0
	old standard	2 turns of rope	50	0·8

Figure 110: Examples of different energy ratios (different countries, various hammer and release mechanism) (Skempton 1986)

3.2.6.1 Conclusion on energy transfer

All standards allow the use of various equipment. In practise one can find several types of hammers and lifting or dropping mechanism, as well as different dimensions and installations on the equipment (e.g. sampler with/ without liner). As a result, the same type of hammer can be operated differently. In respective literature, one can find drill rigs manufactured by different manufacturers. Thus, the combination of hammer type and drill rig type results in a matrix of systems. These systems introduce different amounts of energy per blow into the drill rod, which explains the wide range of published correction factors. The effect of this wide range on the N values is very pronounced. For example, the N values, using an automatic hammer could be multiplied by 0.8 or multiplied by 1.67- a range of 87%. Such a wide range in values is not acceptable in design. Hence, the correction factor for each drill rig should be determined from actual energy measurements (Sherif Aggour et al 2001).

3.2.6.2 Energy Measurement Methods

A complete description on energy measurement methods is provided by Clayton (1995).

Basically, there are two approved methods of calculating the maximum transferred energy to the drill rods (compare Sherif Aggour et al. 2001).

The first method uses the integration of the product of the force and velocity record over time (Force- Velocity method). For this method the transferred energy is determined by:

$$EFV = \int F_{(t)} * V_{(t)} dt$$

Where: F = the force at time t,
V = the velocity at time t;

The integration begins at impact (time the energy transfer begins) and ends at the time at which energy transferred to the rod reaches a maximum value. This method is theoretically sound and requires no correction factors.

The second method calculates transferred energy to the drill rod using the square of the force record (F- square method) referred to as EF2 and is as follows:

$$EF2 = (\frac{c}{EA}) \int {F_{(t)}}^2 dt$$

Where: c = stress wave propagation speed in the drill rod,
E = modulus of elasticity of the drill rod,
A = cross sectional area of the drill rod,
F = the force at time t;

The integration begins at the time of impact and ends at the time of the first occurrence of a zero force after impact. The force squared method requires the use of three correction constants and there is uncertainty associated with the use of these correction constants.

Without going into much detail, M. SHERIF AGGOUR concludes that in the first method, the force velocity method is unaffected by changes in cross-sectional area and is based on measured values. This method is believed to be exact.

Note: ASTM D4633-86 describes and prefers the second method. In contrast, the Eurocode attached the first methods as a recommendation in the EN ISO 22476-3.

3.3 Correlations on SPT results

The following geotechnical parameters are correlated in the following section:

- Relative density of sands,
- Friction angle of sands and silts,
- Undrained shear strength of clay,
- Stiffness modulus of sands and clays;

Since the long-time existence of the Standard Penetration Test, numerous correlations have been published or modified since then. To list all correlations would go beyond the scope of this thesis.

To give an outlook of the amount of existing correlations, one example on the friction angle φ´ (data: Enntal project KB 1) is presented below (interpretation using NOVOSPT software). The correlations scatter in a range between 26,6- 45°!

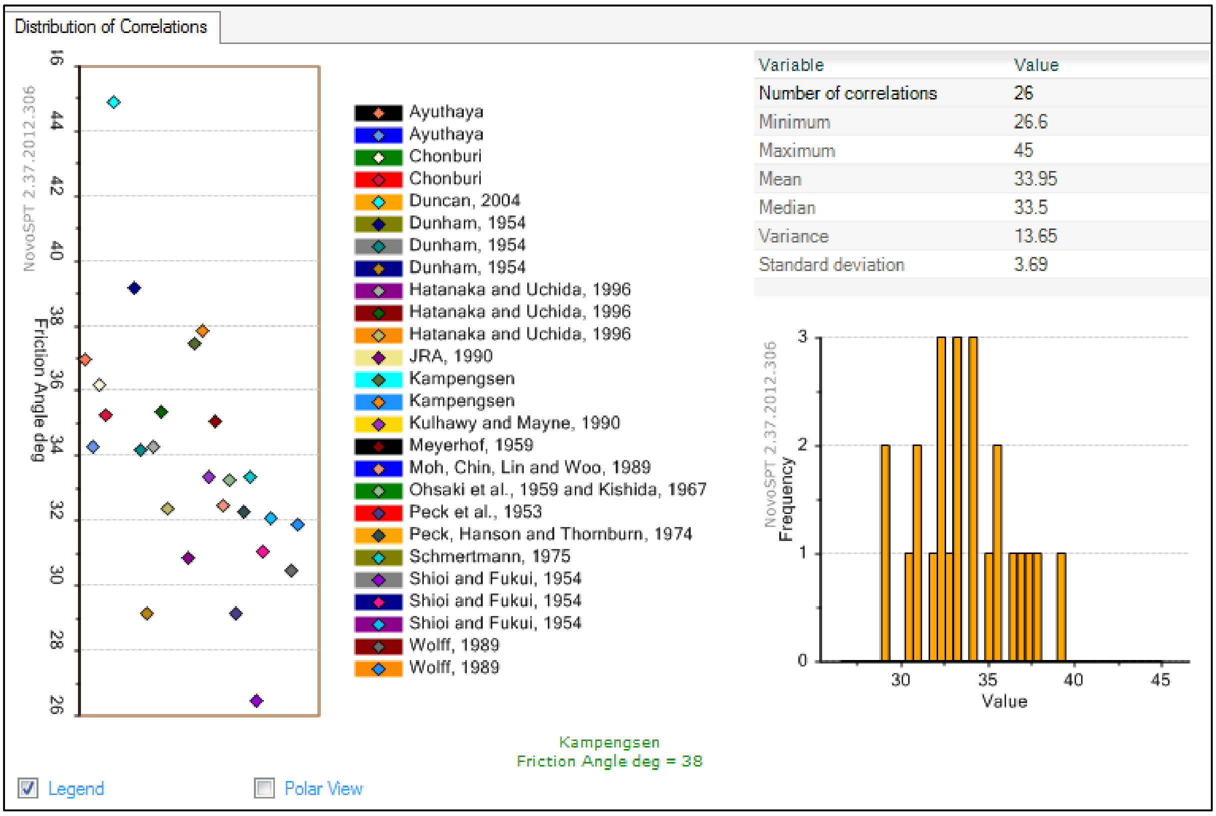


Figure 111: 26 (!) available correlations on φ´ (data: project 12), varying from 26,6- 45° (screenshot of interpretation of NOVOSPT- software)

Complete lists of all existing correlations (including equations, diagrams, etc.) are shown in McGregor et al. 1998, page: 54 – 91.

Before realizing the problem of correcting the measured SPT- N value, correlations on the (uncorrected) N- value have been developed. As a result, recent researchers recommend the corrected N value (N_{60}) to be applied on older correlations (compare McGregor et al. 1998).

Clayton et al, 1995, published some recommended correlations on some geotechnical parameters (that will be introduced in this paper):

Table 22: recommended correlations by Clayton et al, 1995

Parameter and ground condition	Reference
Effective angle of friction of sand	Peck et al. 1974
	Mitchell et al 1978
Stiffness of sand (Young´s modulus)	Stroud 1989
Undrained shear strength of clay	Stroud 1974

This paper will focus on correlations/ ranges developed in German speaking areas, the recommended correlations by Clayton et al. 1995 and additionally mention the reference (from NOVOSPT) for the other correlations.

3.3.1 Relative density of sands

Several ranges have been published on the relative density of sands.

In the following, ranges from several European- guidelines, recommendation or standards will be presented:

Table 23: after Deutsche Gesellschaft für Geotechnik, EA-Pfähle, 2nd Edition 1990; (and same values than Duncan and Buchignani, 1976)

	Very loose	loose	Medium dense	dense	Very dense
$(N_1)_{60}$	1-4	4-10	10-30	30-50	>50
I_D [%]	0-15	15-35	35-65	65-85	85-100

Table 24: correlation between $(N1)_{60}$ and relative density of sands (EN ISO 1997-2 F.3;)

	Very loose	loose	Medium dense	dense	Very dense
$(N_1)_{60}$	0-3	3-8	8-25	25-42	42-58
I_D [%]	0-15	15-35	35-65	65-85	85-100

EN ISO 1997-2 F 3 further states that for fine sands N- values should be attenuated in a rate of 55:60 and increased in a rate of 65:60 for coarse sands.

The DIN 4094- 2 distinguishes the grade of uniformity in sands:

Table 25: calculation of ID according to DIN 4094- 2, with the same limits applied as in EN 1997- 2; F3

<table>
<tr><td colspan="2"></td><td colspan="2">Uniformed- graded sands U≤3</td><td colspan="2">Well graded gravel- sand mixtures U≥6</td></tr>
<tr><td colspan="2">Relative density of sands</td><td colspan="2">$I_D = 0{,}1 + 0{,}385 * \log(N_{30})$</td><td colspan="2">$I_D = -0{,}03 + 0{,}455 * \log(N_{30})$</td></tr>
<tr><td colspan="6"></td></tr>
<tr><td></td><td>Very loose</td><td>loose</td><td>Medium dense</td><td>dense</td><td>Very dense</td></tr>
<tr><td>I_D [%]</td><td>0-15</td><td>15-35</td><td>35-65</td><td>65-85</td><td>85-100</td></tr>
</table>

No comment is given for U in the range of 3 to 6.

Table 26: relationship between SPT (N) and relative density of conhesionless soils (after: Foundation Engineering Handbook, 1990)

	Very loose	loose	Medium dense	dense	Very dense
$(N_1)_{60}$	<4	4-10	10-30	30-50	>50
I_D [%]	<0.2	0.2- 0.4	0.4- 0.6	0.6- 0.8	>0.8

A list of further correlations on relative density of sands (provided by NOVOSPT- software):

- Gibbs and Holtz, 1957
- Meyerhof, 1957
- Yoshida et al., 1988
- Idriss and Boulanger, 2003
- Skempton, 1986
- Cubrinovski and Ishihara, 1999 (3 equations)

3.3.2 Friction angle of sands and silts

EN ISO 1997-2, F.3 gives an example of deriving the friction angle from the relative density, in respect to the grain size and uniformity of the soil.

Table 27: correlation on effective friction angle φ´ derived from relative density of Quarz- sands, that table distinguishes in grain size and uniformity of sands (EN ISO 1997- 2; F3)

Relative density I_D [%]	**Fine- grained**		**Medium grained**		**Coarse grained**	
	Uniformed graded	**Well graded**	**Uniformed graded**	**Well graded**	**Uniformed graded**	**Well graded**
40	34	36	36	38	38	41
60	36	38	38	41	41	43
80	39	41	41	43	43	44
100	42	43	43	44	44	46

Another correlation derives the friction angle directly from the SPT N- value.

Table 28: after“ Deutsche Gesellschaft für Geotechnik“, EA-Pfähle, 2nd Edition 1990;

	Very loose	loose	Medium dense	dense	Very dense
$(N_1)_{60}$	1-4	4-10	10-30	30-50	>50
φ´	30	30	30-35	35-40	40-45

Table 29: recommendation after Duncan and Buchignani, 1976

	Very loose	loose	Medium dense	dense	Very dense
$(N_1)_{60}$	1-4	4-10	10-30	30-50	>50
φ´	<30	30-32	32-35	35-40	40-45

Table 30: recommendation of *Bundesanstalt für Wasserbau*[4]

$(N_1)_{60}$	<4	4-12	12-22	22-38	>38
φ´	30	30-35	35-37.5	37,5-40	≥40

[4] http://www.baw.de/de/die_baw/publikationen/merkblaetter/index.php.html

Comparison of correlations from CPTu, SPT, DP;

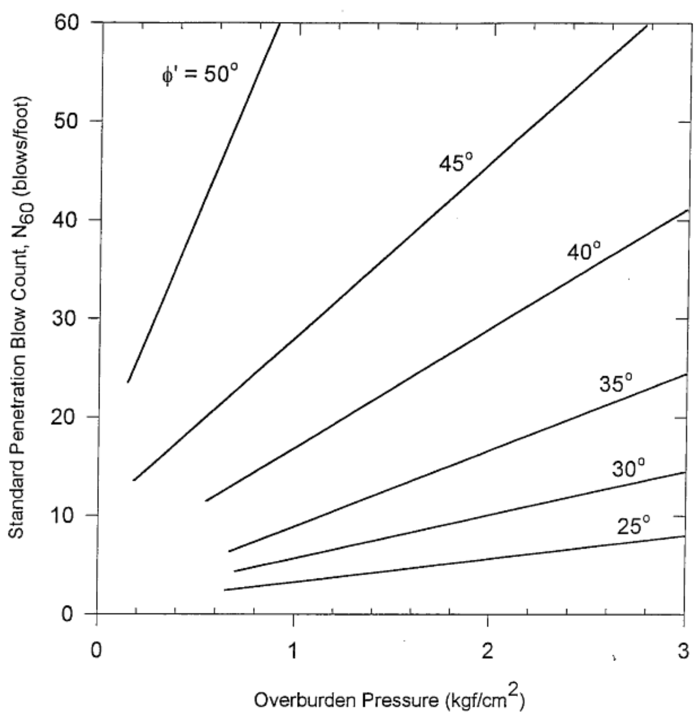


Figure 112: recommended procedure by Clayton et al 1995 to estimate the friction angle with regard to overburden pressure, (unit conversion: 1kgf/cm²=100KPa) (McGregor et al 1998)

A list of further correlations on friction angle of sands and silts (provided by NOVOSPT- software):

- Terzaghi, Peck and Mesri (2 equations)
- Hatanaka and Uchida, 1996 for Sands (2 equations)
- Ohsaki et al., 1959
- Japan Road Association, 1990
- Dunham, 1954 (3 equations)
- Shioi and Fukui, 1954 (2 equations)
- Meyerhof, 1959
- Peck, Hanson and Thornburn, 1974
- Bowles, 2002
- Mezenbach, 1961 (7 equations)
- Kampengsen (2 equations for N60 and N1,60)
- Chonburi (2 equations for N60 and N1,60)
- Ayuthaya (2 equations for N60 and N1,60)
- Wolff, 1989
- Kulhawy and Mayne, 1990
- Peck et al., 1953
- Duncan, 2004 (3 equations)
- Schmertmann, 1975
- Hettiarachchi and Brown, 2009

3.3.3 Undrained shear strength (c_u) of clay/ silts

Table 31: approximate values based on experience for standard applications, according to DIN 18122 (TUMünchen, lecture notes from Prof. Vogt [5])

according DIN 18122		c_u [kN/m²]	SPT N-value
Consistency	**Ic**		
Breiig	0-0.5	<20	<2
Weich	0.5-0.75	20-60	2-6
Steif	0.75-1	60-200	6-15
Halbfest- fest	>1	200-400	15-30
	>1.25	>400	>30

Table 32: approximate values of undrained shear strength proposed by Tschebotariff (1973) and Parcher and Means (1968), (compare McGregor et al 1998)

Tschebotariff (1973) c_u [kN/m²]	Parcher and Means (1968) c_u [kN/m²]	SPT N-value
30	---	<2
30-60	25-50	2-4
60-120	50-100	4-8
120-240	100-200	8-15
240	200-400	15-30
>450	>400	>30

Table 33: approximate values of undrained shear strength suggested from Terzaghi and Peck 1967, (compare McGregor et al 1998)

Consistency	c_u [kN/m²]	SPT N-value
Very soft	<25	<2
Soft	25-50	2-4
medium	50-100	4-8
Stiff	100-200	8-15
Very stiff	200- 400	15-30
hard	>400	>30

[5] http://www.gb.bv.tum.de/download/skript/vorl-g-e.pdf

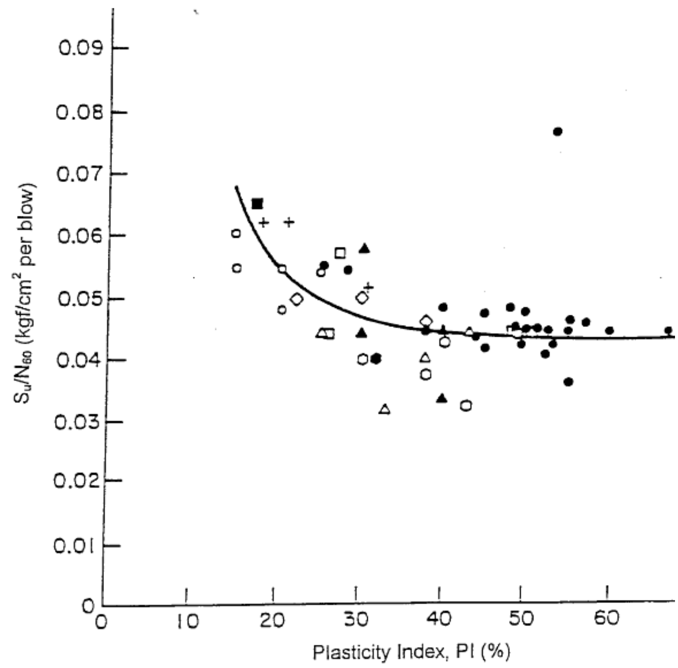


Figure 113: Stroud 1974, correlation for su, recommended by Clayton et al 1995 (from McGregor et al 1998)

A list of further correlations on undrained shear strength (c_u) of clay (provided by NOVOSPT-software):

- Meyerhof, 1956
- Peck et al., 1974
- Ghahramani and Behpoor, 1989
- Stroud, 1989
- Sowers, 1979 for all soil types
- Stroud and Butler, 1975
- Japanese Road Association
- Reese, Touma and O'Neill, 1976
- Kulhawy and Mayne, 1990
- Hara et al., 1974
- Hatef and Keshavarz, 2004
- Tavares, 1988
- Ajayi and Balogun, 1988
- Decourt, 1989
- Bowles, 1988
- Tavares, 1988
- Hettiarachchi and Brown, 2009

3.3.4 Constrained modulus E_S

Bundesanstalt für Wasserbau:

Table 34: recommendation from *Bundesanstalt für Wasserbau*[6]

SPT N30	Es [MN/m²]
<4	<15
4-12	15-50
12-22	50-80
22-38	80-100
>38	≥100

According to DIN 4094- 3:

$$E_s = v * p_a * [\frac{\sigma_{ü} + 0,5 * \Delta\sigma_z}{p_a}]^\wedge w$$

With:

The vertical stresses at the foundation level $\sigma_{ü} = \gamma * (d + z)$

E_S= constrained modulus

v=stiffness factor

w=stiffness exponent (w=0,5 for sands, and w=0,6 for clays of low and medium plasticity)

p_a= atmospheric pressure (p_a=0,1MPa)

$\Delta\sigma_z$= increase of the vertical stresses at the foundation level due to loading;

Above groundwater table, uniform graded sands: w=0,5

BDP: ($3 \leq N_k \leq 25$)

$$v = 217 * \log(N30) + 146$$

Above groundwater table, clay of low and medium plasticity, w=0,6 (0,75≤ Ic≤ 1,3), SR~0,8

BDP: ($3 \leq N_k \leq 23$)

$$v = 4 * \mathrm{N30} + 50$$

3.4 Projects

Applied correction:

As introduced, the measured N value has to be corrected. For the evaluation of correlations, the correction as presented in the EN ISO 22476-3 will be used and compared with the more detailed correction (including borehole diameter with/ without liner etc.), mentioned above and implemented in NOVOSPT. Furthermore, correction for sands below the groundwater table will be applied and results will be compared.

[6] http://www.baw.de/de/die_baw/publikationen/merkblaetter/index.php.html

For all projects energy measurements have not been performed, but effects will be investigated by varying the ER from 90 to 50%.

In contrary, to what is stated in the EN ISO 22476- 3 (overburden correction factor for sands and rod length correction factor for sands), NOVOSPT applies the correction (valid for sands) to all soil types, which is questionable.

3.4.1 Project 10 (mainly sands)

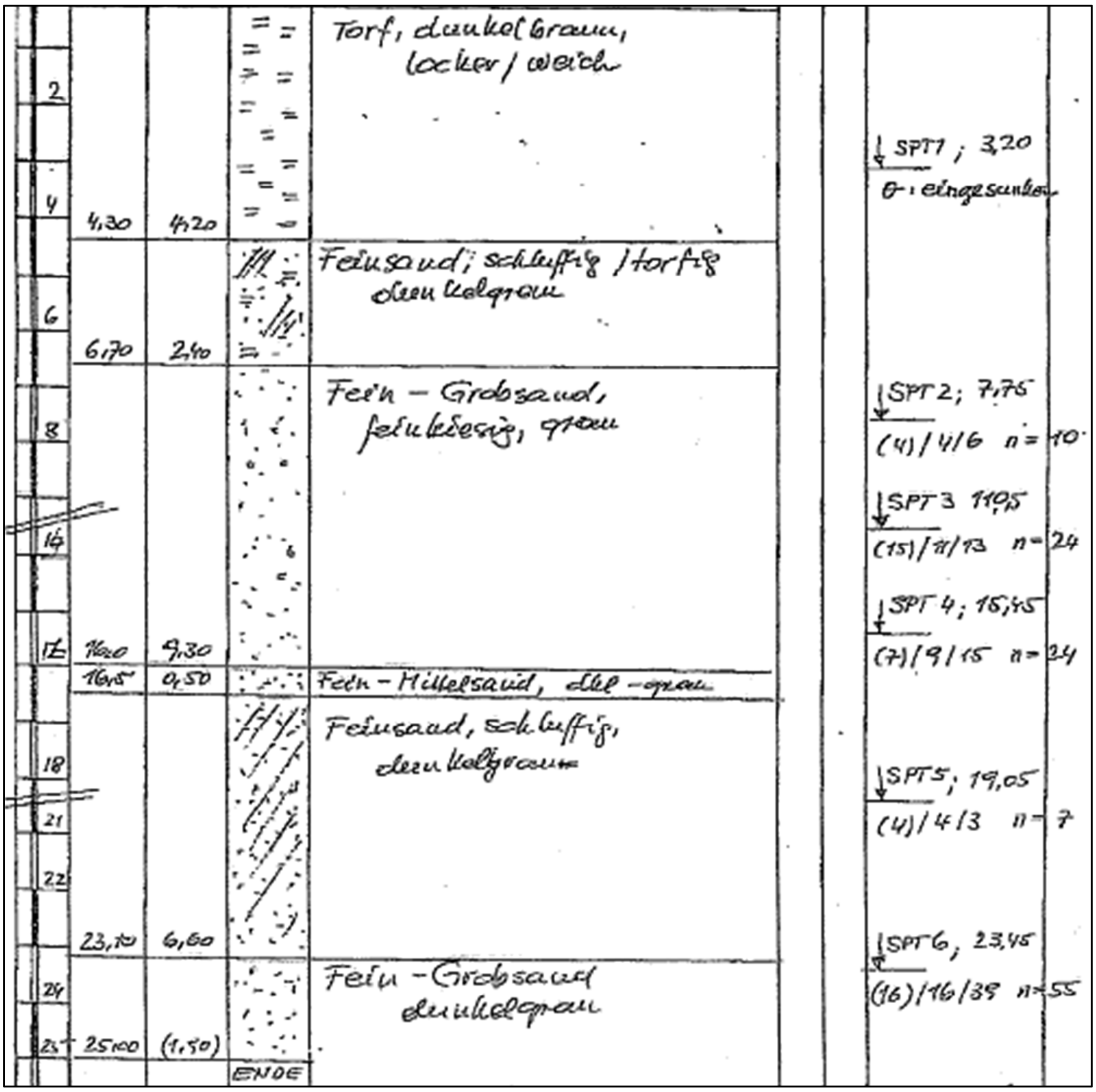


Figure 114: borehole and SPT blows (Project 10);

Unfortunately, for that project, results of the geotechnical report are not available.

Application of correction factors with a ER of 60%:

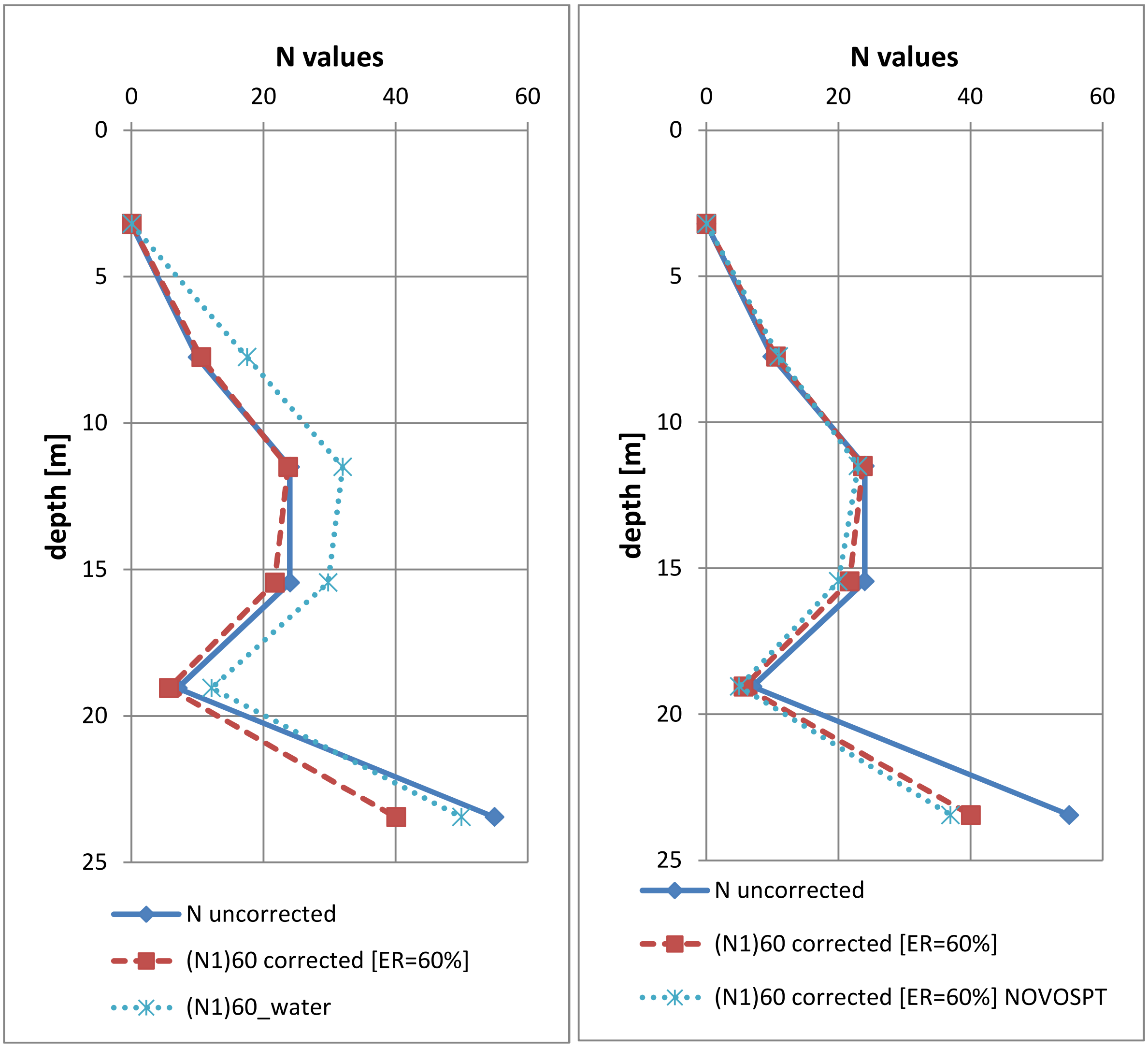


Figure 115: comparison of uncorrected and corrected N- values;

The small differences in the (N1)60 value occure due to the additional application of further corrections (borehole, liner etc.).

Notably are corrections due to groundwater in sands (as proposed by DIN and Melzer 1971) are more significant. In the left figure the differences at a depth of 23m are about 10 blows, that can have a significant effect when correlating to further geotechnical parameters.

For the sake of completeness the correction factor for overburden pressure and rod length are shown in Figure 116.

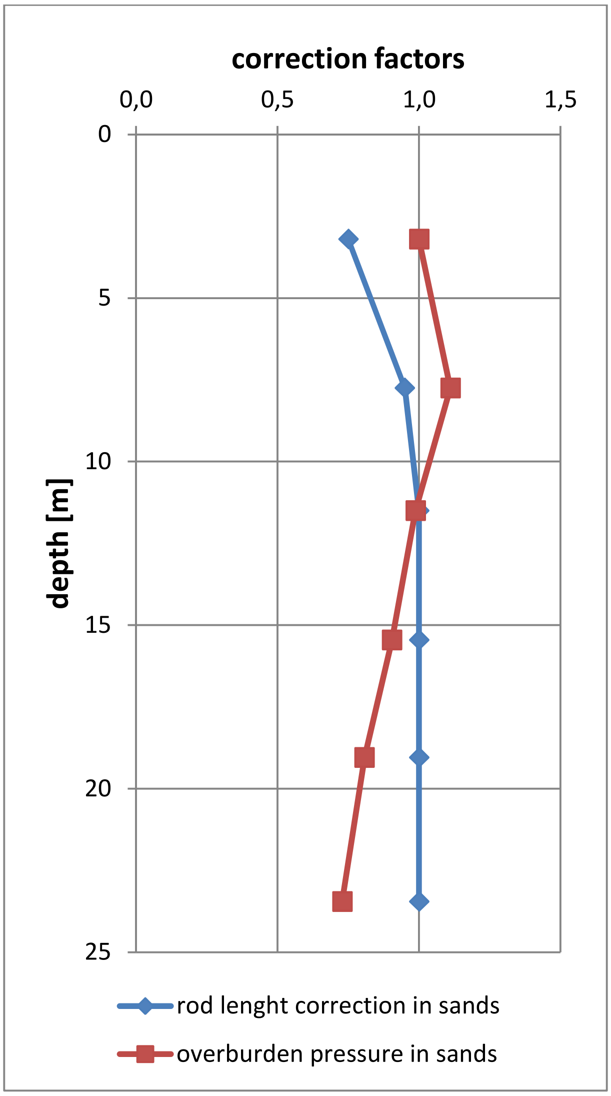


Figure 116: correction factors over depth

The significance of energy measurement will be shown in this project. The following figure shows the differences in applied Energy- ratio: ER=50% (for example a donut hammer) and ER= 90% (for example a automatic tip hammer).

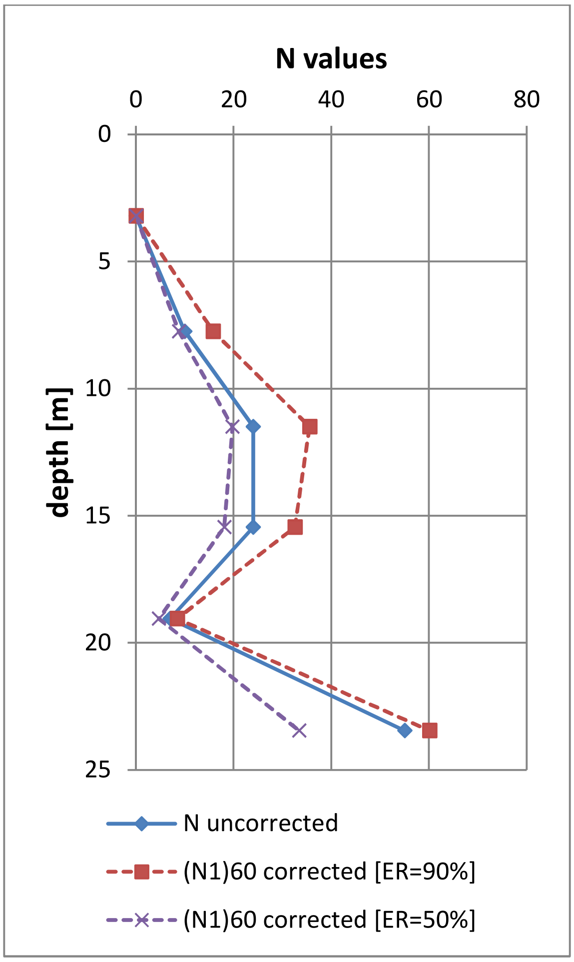


Figure 117: SPT n- blows with different Energy ratios ER=50-90;

Here, one can see the most significant factor concerning SPT corrections. An Energy ratio of 50% showed almost half or the N- values compare to an Energy ratio of 90%. This highlights the importance of energy measurements.

For this project, just correlations in sand can be applied, that are:

- Relative density
- Friction angle
- Constrained modulus

As there are certain papers or standards that publish ranges for parameters like the relative density or the friction angle for certain N- blows (as introduced), the mean value (in legend marked with *MEAN VALUE* at the end) will be taken for the evaluation.

3.4.1.1 Relative density

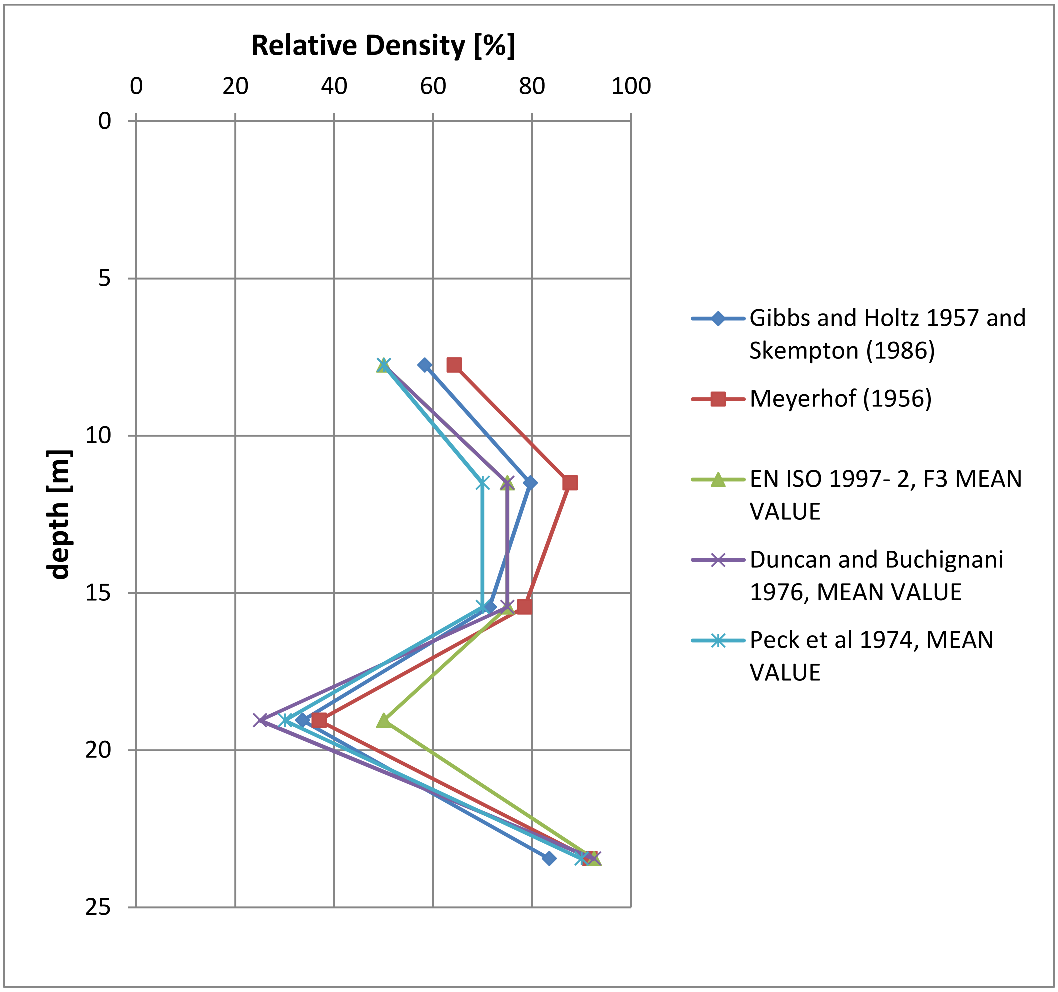


Figure 118: comparison of various correlations for the relative density;

Softwares like the NOVOSPT have a larger amount of correlations available. Some of these correlations have certain requirements or just fit to specific geographical areas. The usefulness of plotting all existing equations and the ability to compare the mean values or the standard derivation is questionable, if knowledge on the corrections does not exist. However, for the sake of completeness one example on the relative density over depth that is plotted in Figure 119, and two examples on the variation of results at depth 11.5m and 19.05m (Figure 120) are presented below.

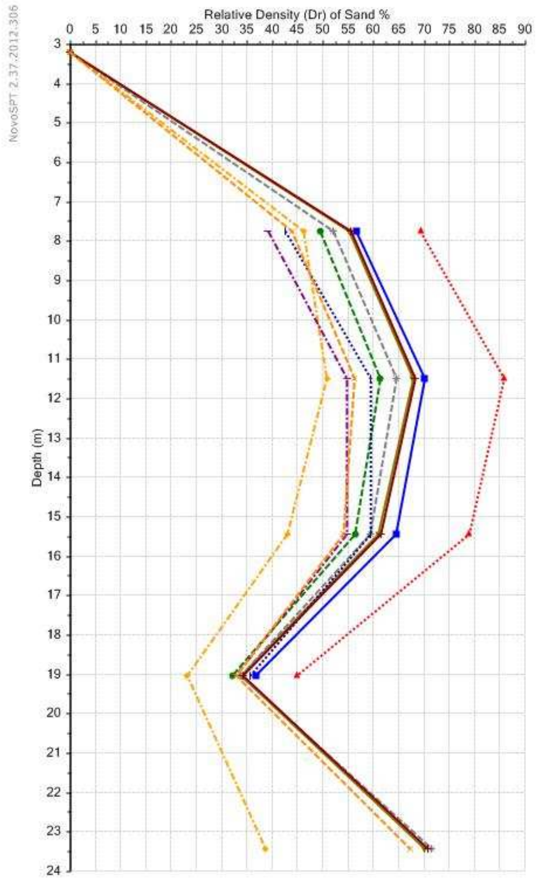


Figure 119: Correlations for the relative density in NOVOSPT

Number of correlations	10
Minimum	50.8
Maximum	85.9
Mean	63.97
Median	63
Variance	99.7
Standard deviation	9.98

Number of correlations	10
Minimum	23.2
Maximum	44.9
Mean	34.02
Median	33.85
Variance	28
Standard deviation	5.29

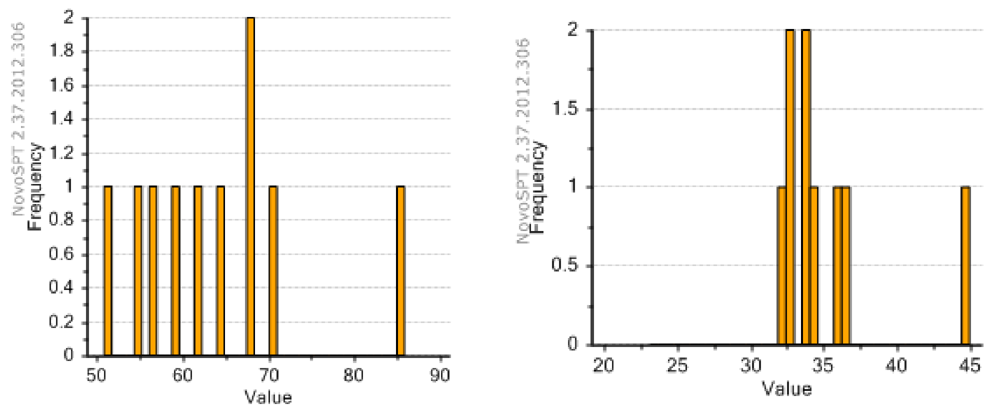


Figure 120: variations of the applied correlation (50-85) at a depth of 11,5m (left) and (35-44) at a depth of19.05m (right);

3.4.1.2 Friction angle and constrained modulus

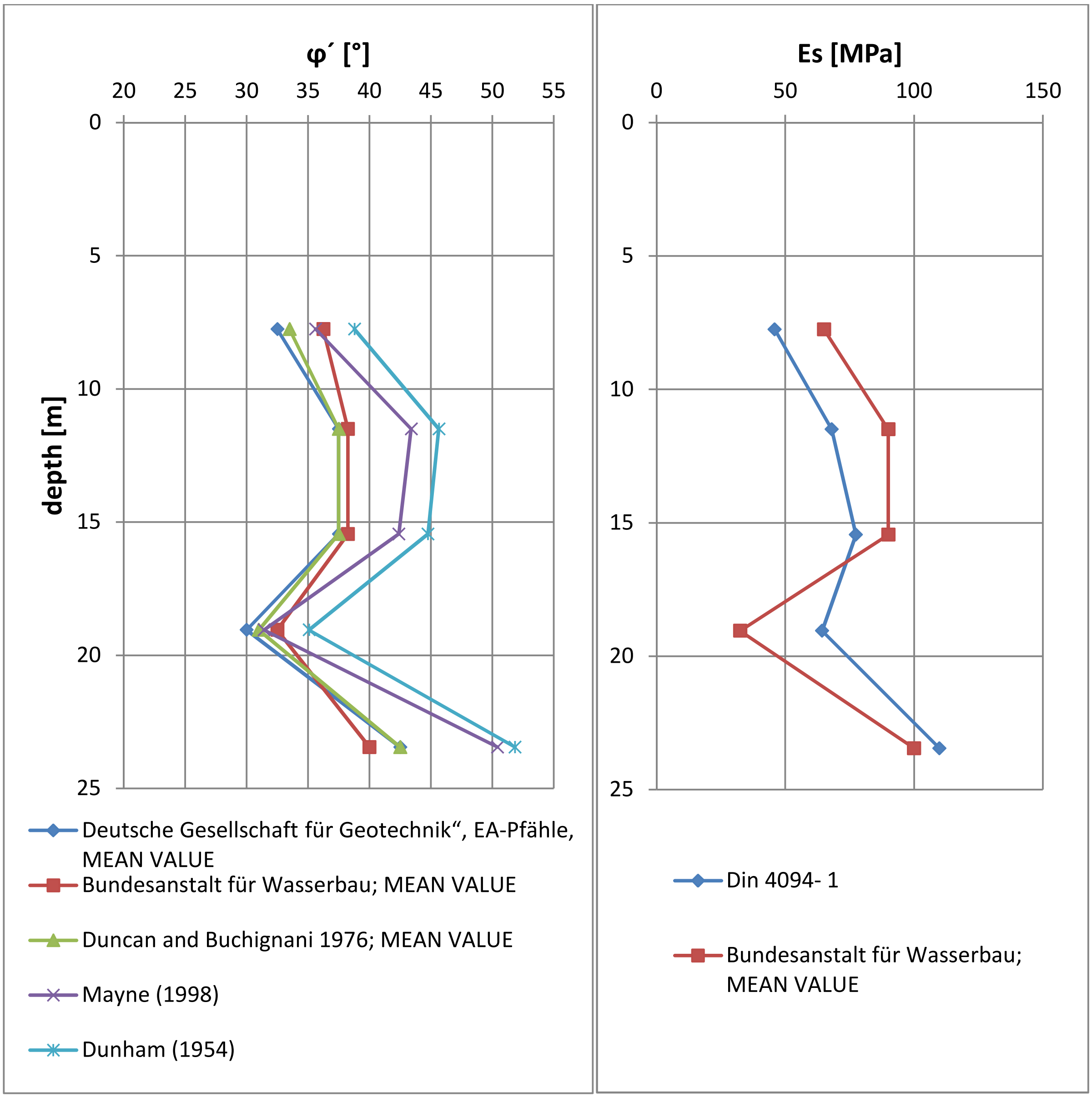


Figure 121: correlation to the friction angle (left) and constraint modulus (right);

3.4.2 Project 11 (including clay)

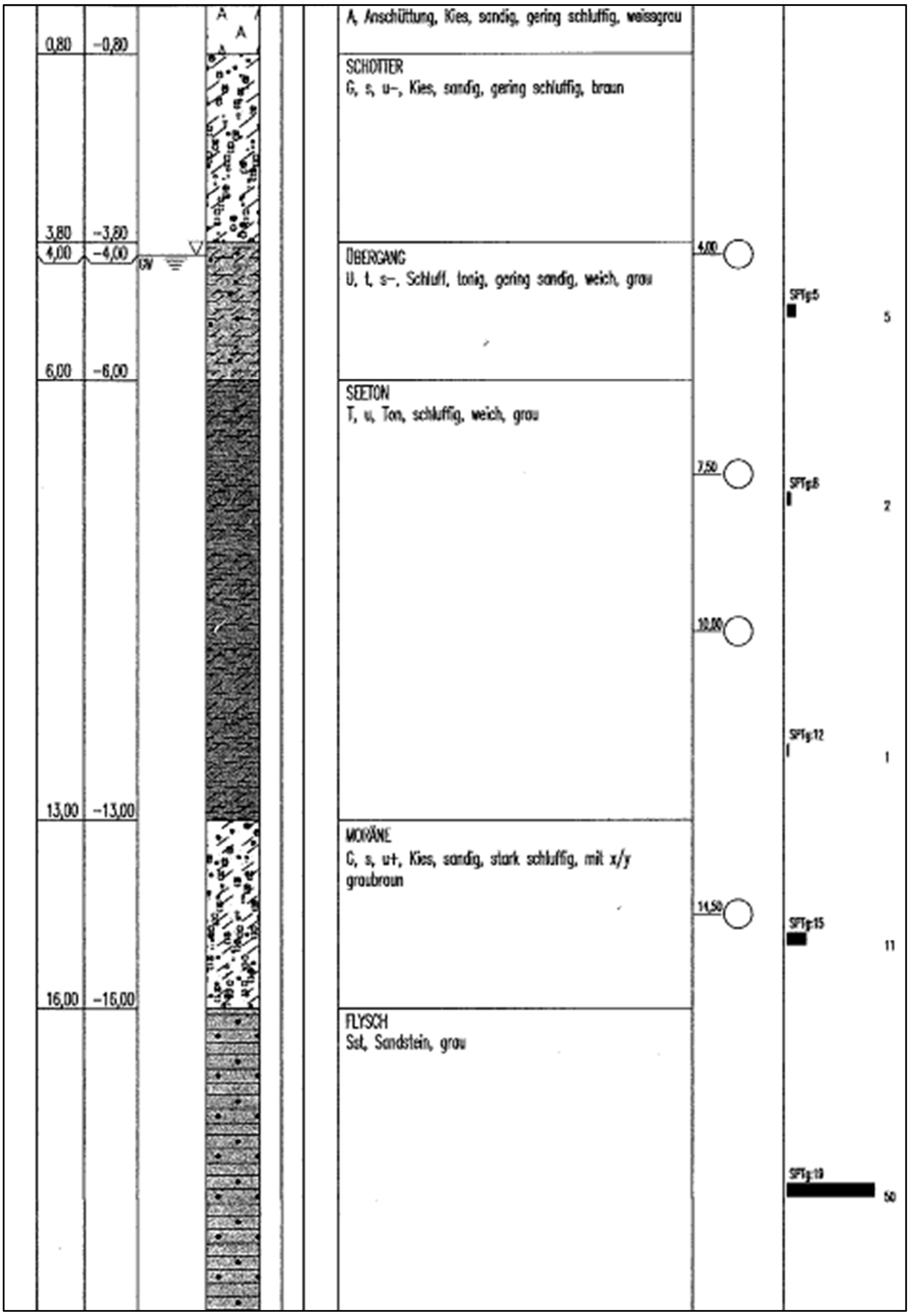


Figure 122: Borehole and SPT N- values;

Also for this project, information parameters derived from SPT in the geotechnical report are not available.

The Energy ratio was again assumed to be 60%. The corrected N value is calculated using the Eurocodes´suggestion and compared with the more detailed approach (NOVOSPT). As the first SPT measurement was taken in a silt layer (and the two following have been conducted in clay), the correction for the overburden pressure was not applied for the calculation following the Eurocode (Skempton 1986). Still, differences seem to be small.

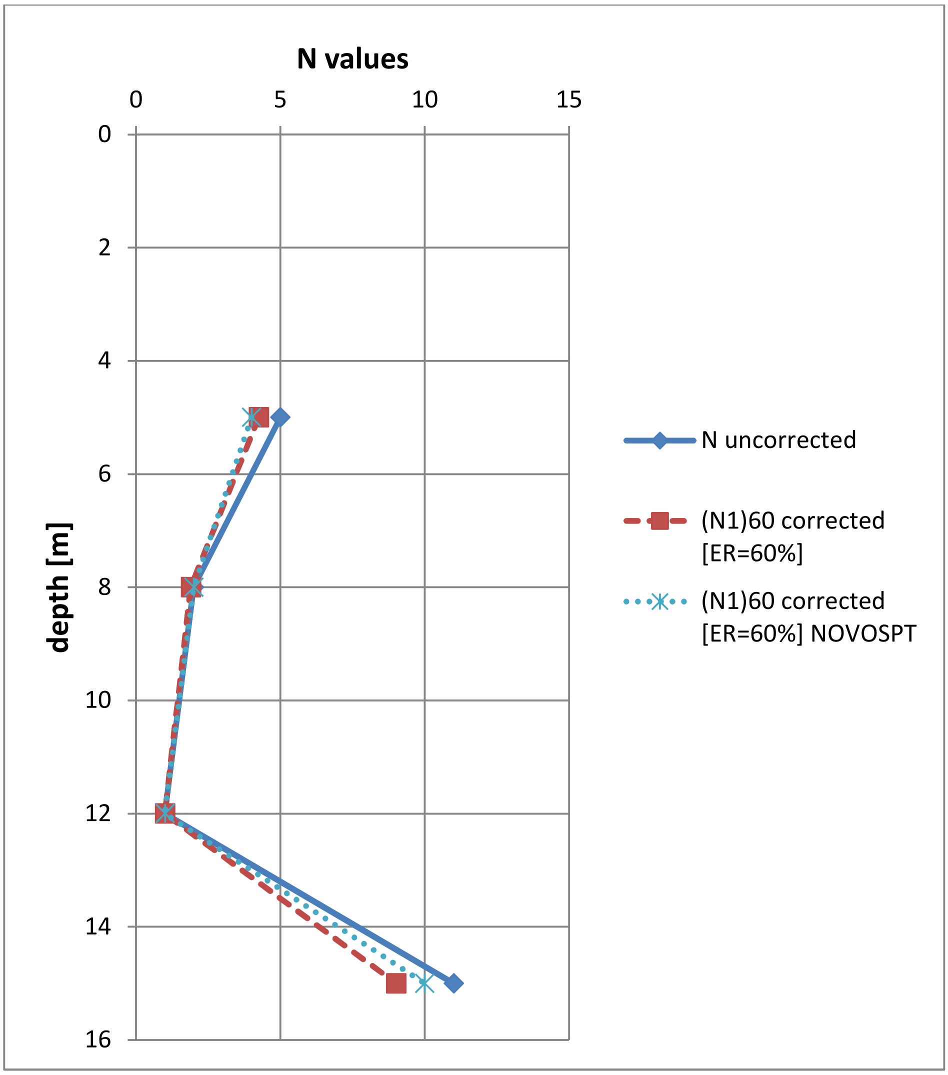


Figure 123: comparison of the uncorrected and corrected N value;

3.4.2.1 Undrained shear strength and constrained modulus

The undrained shear strength could be determined in the clay layer where two SPT tests have been performed. The constrained modulus was obtain in the clay layer and the sand layer (depth 15m, DIN 4094-1).

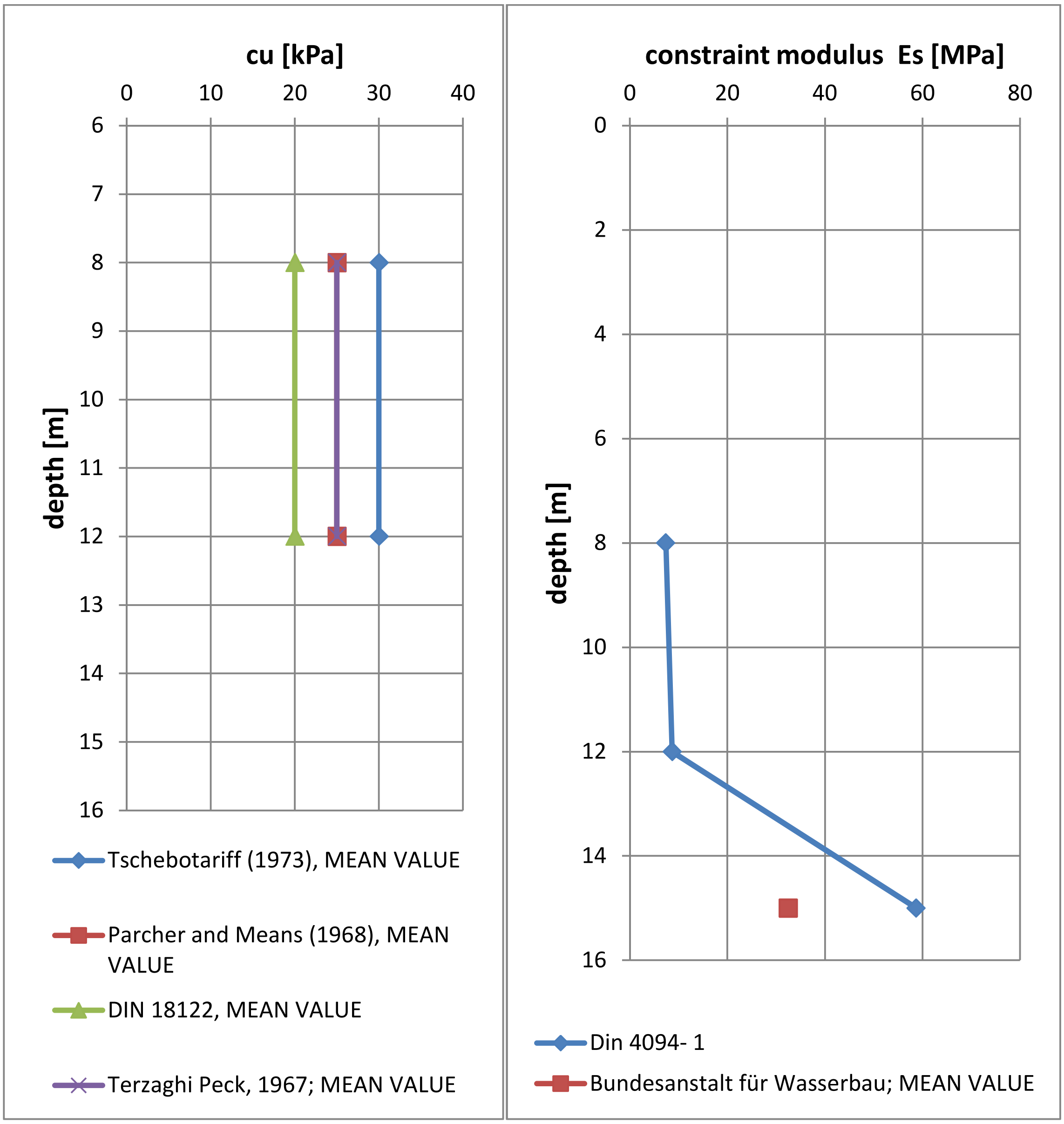


Figure 124: undrained shear strength (left) and the constrained modulus for the clay layer (at 8m and 12m) and at depth 15 a correlation for sand (right);

3.4.3 Conclusion

To evaluate the SPT blows, information of the soil type must be provided beforehand: For accurate correlations, prior knowledge of the soil density, the coefficient of uniformity and the grain size distribution is required for each layer.

For the correction of the measured N- value, the energy ratio must be known prior to evaluation. No comment about the correction for sands in groundwater, which has a significant influence on SPT results, exists in the EN or the IRTP.

It is very hard to correlate actual geotechnical parameter like, constrained modulus E_S, friction angle φ´ and undrained shear strength, if the corrected N- value is not known. As for the CPT, knowledge about the local ground condition is indispensable for the evaluation.

A large amount of correlations exist. For the user it is difficult to select the “right” one. The results of existing correlations vary in a wide range.

Most correlations are just applicable for one soil type and have restrictions (coefficient of uniformity/ relative density etc.).

3.5 Literature on SPT

3.5.1 Papers

Mayne P. W., Stress-strain-strength-flow parameters from enhanced in-situ tests, Proceedings, International Conference on In-Situ Measurement of Soil Properties & Case Histories [In-Situ 2001], Bali, Indonesia, pp. 27-48, 2001;

Clayton C.R.I., The Standard Penetration test (SPT): Methods and Use, Construction Industry Research and information Association, CIRIA Report 143, 1995;

Mayne P. W., Christopher B. R. and DeJong J., Manual on Subsurface Investigations, National Highway Institute Publication FHWA NHI-01-031, Federal Highway Administration, Washington, DC, 1990;

SHERIF AGGOUR M. AND ROSE RADDING W., Standard Penetration Test Correction, the bridge engineering software and technology (best) Center, Department of civil and environmental engineering, university of Maryland, final report, 2001;

McGregor Jeffrey A. and Duncan J. Michael, Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice, Report of a study performed by the Virginia Tech Center for Geotechnical Practice and Research, October 1998;

Schnaid F., Insitu testing in Geomechanics, The Main tests, Taylor und Francis; London and New York, 2009;

Deutsche Gesellschaft für Geotechnik e.V. (Hrsg.), EA-Pfähle, Empfehlungen des Arbeitskreises "Pfähle", Ernst&Sohn, Berlin, 2007;

Hsai-Yang Fang, Foundation Engineering Handbook, 2nd edition, Springer, New York, 1990;

SKEMPTON A. W., Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing and overconsolidation Geotechnique 36, No. 3, 425-447, 1986;

Clayton C. R. I., Matthews M. C. and Simons N. E., Site Investigation, Department of Civil Engineering, University of Surreysecond edition, September, 1995;

Melzer K. J., Standard penetration test and relative density, Army Engineer Waterways Experimfnt Station, Vicksburg, Mississippi, 1971;

Kulhawy F.H., Mayne P.W., Manual on estimating soil properties for foundation design, Report EL-6800, Electric Power Research Institute, Palo Alta, CA, August: 306, 1990;

3.5.2 Standards and Guidelines

ASTM D1586 – 11, Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils;

BS 1377 Part 9, 1990, Soils for engineering purposes- SPT;

ISSFME 1989, International reference test procedure for Standard Penetration Test (SPT)

EN ISO 22476- 1, Geotechnical investigation and testing -- Field testing -- Part 3: Standard penetration test;

DIN 4094- 1, 2002- 06 Baugrund - Felduntersuchungen - Teil 1: Bohrlochrammsondierungen;

EN 1997-2: 2010, Entwurf, Berechnung und Bemessung in der Geotechnik, Teil 2: Erkundung und Untersuchung des Baugrunds;

4. Dynamic Probing

4.1 Description of the test

The dynamic test consists of driving a cone into the ground, via an anvil and extension rods, with successive blows of a free fall hammer. The number of blows required to drive the cone each successive 10 cm (N_{10}) is recorded, so creating a record of blows/10 cm with depth. When driving is halted to add a further extension rod (every metre) the torque, required to rotate the extension rods that are already in the ground, is measured to assess the friction on the rods from the soil. The free fall hammer may be raised by hand (only practical with lightweight hammers) or by a device incorporating a motor driven continuous chain with an automatic latch and release arrangement for the hammer (Butcher et al. 1995).

To minimise friction on the rods and for a better detection of the cone resistance, the cone has a slightly bigger diameter than the rods.

A configuration of the cone and the connection to the rods as presented in the ISO 22476- 2, is shown in Figure 125.

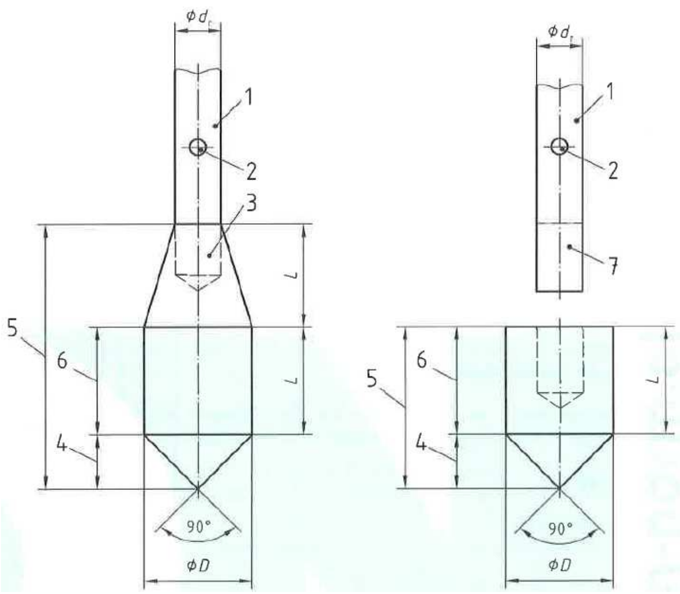


Figure 125: left: example of a cone and connection to the rods, “Type 1”, right: alternative configuration (as lost cone) (ISO 22476: 2)

4.2 Equipment and its variations

Figure 126 presents the equipment standardized in the EN ISO 22476- 2. The Eurocode introduces most available variations of the dynamic probing (DP) equipment including DP light (DPL), DP medium (DPM), DP Heavy (DPH) and two types of DP SuperHeavy (DPHS-A, DPHS-B; Fig. 6). Other standards (DIN 4094-3, BS 1377-9, IRTP (1989)) present only selected DB types (see Table 17 for details).

Geräte für Rammsondierungen	Symbol	Einheit	DPL (leicht)	DPM (mittel)	DPH (schwer)	DPSH (superschwer) DPSH-A	DPSH (superschwer) DPSH-B
Rammvorrichtung							
Rammbärmasse, neu	m	kg	10 ± 0,1	30 ± 0,3	50 ± 0,5	63,5 ± 0,5	63,5 ± 0,5
Fallhöhe	h	mm	500 ± 10	500 ± 10	500 ± 10	500 ± 10	750 ± 20
Amboss							
Durchmesser	d	mm	50 < d < D_h [a)]	50 < d < D_h [a)]	50 < d < 0,5 D_h [a)]	50 < d < 0,5 D_h	50 < d < 0,5 D_h [a)]
Masse (max.) (einschließlich Führungsstange)	m	kg	6	18	18	18	30
90°-Sondenspitze							
Nennquerschnittsfläche	A	cm^2	10	15	15	16	20
Spitzendurchmesser, neu	D	mm	35,7 ± 0,3	43,7 ± 0,3	43,7 ± 0,3	45,0± 0,3	50,5 ± 0,5
Spitzendurchmesser, abgenutzt (min.)		mm	34	42	42	43	49
Mantellänge (mm)	L	mm	35,7 ± 1	43,7 ± 1	43,7 ± 1	90,0 ± 2 [b)]	51 ± 2
Höhe des Kegels		mm	17,9 ± 0,1	21,9 ± 0,1	21,9 ± 0,1	22,5 ± 0,1	25,3 ± 0,4
max. zulässiger Verschleiß an der Sondenspitze		mm	3	4	4	5	5
Gestänge [c)]							
Masse (max.)	m	kg/m	3	6	6	6	8
Außendurchmesser (max.)	d_a	mm	22	32	32	32	35
Gestängedurchbiegung [d)]:							
unterste 5 m		%	0,1	0,1	0,1	0,1	0,1
restliche Länge		%	0,2	0,2	0,2	0,2	0,2
spezifische Arbeit je Schlag	mgh/A	kJ/m^2	50	100	167	194	238

Anmerkung: Die angegebenen Toleranzen sind Herstellungstoleranzen.
[a)] D_h Durchmesser des Rammbären, bei rechteckiger Ausbildung wird die kleinere Länge als Durchmesser angenommen.
[b)] Nur für verlorene Sondenspitze.
[c)] Die maximale Gestängelänge darf 2 m nicht überschreiten.
[d)] Abweichung der Stange von der Vertikale.

Figure 126: Available types of dynamic probing (EN ISO 22476- 2: 2005)

Table 35: Overview of DP variation introduced in various standards

Standard	DPL	DPM	DPH	DPSH- A	DPSH- B
EN ISO 22476- 2	+	+	+	+	+
DIN 4094- 3	+	-	+	+	-
BS 1377-9	-	-	+	-	+
IRTP from ISSMFE	+	+	+	-	+

+ This type of DP is introduced and described.

- This type of DP is NOT introduced.

In the United States, DP and its variations, as introduced above, are not mentioned in a standard. Instead a Dynamic cone penetrometer (DCP) is used in the United States that functions in a similar way but has different dimensions and weights. A short description of the DCP is in the following:

Short description of the DCP:

The device consists of two 16 mm diameter rods, with the lower rod containing an anvil, a replaceable 60° pointed tip, and depth markings every 5, 1 mm. The upper rod contains an 8 kg drop hammer with a 575 mm drop distance, an end plug for connection to the lower rod, and a top grab handle. All materials (except the drop hammer) are stainless steel for corrosion resistance. An optional depth reading device can be attached to eliminate the need to measure penetration depth at ground level (compare Burnham et al 1993).

Data from a DCP test is processed to produce a penetration index (PI) which is simply the distance the cone penetrates with each drop of the hammer. The PI is expressed in terms millimetres per blow. The penetration index can be plotted on a layer strength diagram, or directly correlated with a number of common pavement design parameters (compare Burnham et al 1993).

Presentation of DP results:

DP results are usually presented as blows/ 10cm penetration (N_{10}) for the DPL, DPM, DPH, and blows/ 20cm (N_{20}) for the DPSH (Butcher et al. 1995).

In exceptional cases (outside these ranges), when the penetration resistance is low, e. g. in soft soils, the penetration depth per blow can be recorded. In hard soils where the penetration number is very high, the penetration for a certain number of blows can be recorded. The precision of measuring the total depth of penetration (tip of the cone) shall be ± 0,02mm (IRTP for DP, ISSMFE 1989).

According to the BS 1377: 1990 and the IRTP 1989, the results also include torque readings taken when extension rods are added, to assess rod friction.

The effect of rod friction on the N_{10} values can be significant but no comment on that effect in made in standards or the IRTP (compare Butcher el al 1995).

According to the IRTP 1989, the N_{10} values can be used to calculate the unit point resistance (r_d) and the dynamic point resistance (q_d) applying the equation:

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

$$q_d = \frac{M}{(M + M')} * r_d$$

Where: r_d (unit point resistance) and q_d (dynamic point resistance) in Pa, kPa or MPa

M is the mass of the hammer

M´ is the total mass of extension rods, the anvil and the guiding rods

H is the height of fall

e is the average penetration per blow (0,1/N_{10} for DPL, DPM, DPH and 0,2/N_{20} for DPSH)

A is the area at base of the cone

g is the gravity acceleration;

The value of r_d is an assessment of the driving work done in penetrating the ground and further calculation, to produce q_d values, modifies the r_d value to take account of the inertia of driving the rods and hammer after impact with the anvil. The calculation of r_d includes the different hammer weights, the height of fall and the different cone sizes. The different sizes in numbers of extension rods are included in the calculation of q_d and so allow comparison of different equipment configurations (Butcher et al 1995).

Problem and effect of rod friction:

Several approaches are proposed in papers, standards and guidelines to either eliminate or evaluate extension rod friction. A few of them will be introduced here.

The DP equipment with a cone/rod diameter ratio exceeding about 1.3 leads to results in cohesionless soils that are little or not at all influenced by skin friction. However, for clayey soil types, skin friction develops along the rod length due to the annulus around the rod squeezing in or collapsing. Therefore, the penetration blow count will be the sum of blow counts to overcome the cone driving resistance plus a certain number of blows (Ns) that may be considered as the equivalent to the additional blows required to overcome skin friction effects. The unknown contribution from extension rod friction has been a major difficulty in evaluating dynamic cone penetration test data (compare Abuel-Naga et al. 2011).

The IRTP 1989 argues: To eliminate skin friction, drilling mud can be injected through holes in the hollow rods near the cone. The holes have to be directed horizontally or slightly upwards. The injection pressure should be sufficient so that the drilling mud fills the annular space between the sol and the rod.

Instead of the hollow extension rods (OD=22mm) of the DPL, solid rods of OD= 20mm are being used (IRTP 1989).

The EN ISO 22476- 2 gives an example of a DPM- profile with and without the use of drilling mud and shows the differences in N10 values:

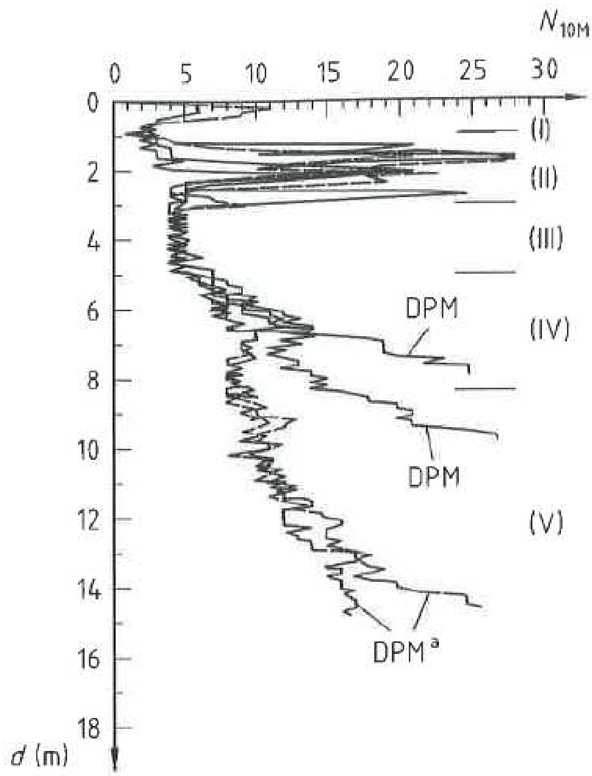


Figure 127: Differences in N10 values, with- (DPM[a]) and without (DPM) drilling mud; I- crust, II- gravel, III- rearranged weathered clay, IV- weathered clay, V- unweathered clay, (EN ISO 22476- 2);

Butcher et al. 1995 conducted DP- tests (with and without drilling mud) at three different tests sites and measured the torque resistance at all tests, including those where bentonite drilling was used. A correction was established where the penetration and torque values were constant. This gave the correction of the torque equivalent to 1 blow of the driving hammer. By dividing the measured torque by the torque correction, the number of blows needed to overcome the rod friction can be calculated and then subtracted from the recorded N10 value.

Butcher et al 1995 concluded that torque corrections obtained at stiff clay sites show good similarity for similar equipment. Instead, torque corrections for soft clay had been difficult to establish.

Instead of eliminating extension rod skin friction, Dahlberg & Bergdahl (1974) suggested a correction factor for the effect of skin friction. The torque T required to turn a group of extension rods in the soil can be used to determine the number of blows Ns required to overcome rod skin friction as follows (compare to ABUEL-NAGA et al 2011):

$$N_s = \frac{2T\alpha e}{DM_0gh}$$

Where:

e is the standard penetration depth increment (100 or 200 mm),

D is rod diameter,

M_0 is the hammer mass,

h is hammer drop height,

g is gravitational acceleration (9,806 m/s²) and

α is the ratio between the average unit rod skin friction in driving and rotation.

Some field studies by Bergdahl (1979) and Scarff (1988) concluded that α could vary from 1 to 4. Based on a laboratory study, Unal & Kayalar (2001) indicated that α depends on soil type and suggested that α= 1 for sand, 2 for silty sands and 4 for clays. Therefore, choosing the right value of a can be the challenge that could limit the usefulness of the equation. Moreover, for multilayer soil profiles, it will be difficult to predetermine the depths at which the value of α should be changed since the dynamic cone penetration testis a blind test that does not provide information about soil type (compare ABUEL-NAGA et al 2011).

Cope 2010 performed a study on the dynamic point resistance. He claimed that the larger the values of M´, the less accurate and more divergent the result will be. As the number of the rods in the string increases, the value of (g/A)*(M+M´) increases significantly. So Cope 2010 concluded that the term – T/(A*r) should be added to the equation of qd, being:

$$q_d = \left(\frac{g}{A}\right) * \left[\frac{M^2}{(M+M')} * \left(\frac{h}{e}\right) + (M+M')\right] - \frac{T}{A*r}$$

With the additional parameters:

T is the torque required to rotate the string,

R is the radius of the rods;

He further stated that for shallow depths probed by the oversized cone, there would have been little rebound (squeezing) of the displaced soil. However, for great depths of rebounding soils, friction on the rods becomes very significant. The term T/[A*r] gives a direct estimation of the value of the resisting skin friction force acting on the rods, which is subtracted from the loading act over the projected area of the cone. This assumes (reasonably) that the frictional force acting on the rod surface has the same value whether it be resisting rotation or vertical motion.

Still researchers don´t agree, of how to avoid or incorporate skin friction in corrections.

4.2.1 General Deviations from the IRTP

“Some light penetrometers have hammers of 20 kg mass (e. g. Bulagrian State Standard 8994-40); in some countries cones with base areas of 5cm² are used (e. g. Belgium, German Standard DIN 4094). In Australia light penetrometers without cones are used during quality controls of compacted sands. Some medium penetrometers have hammers of 20 kg mass, and heights of fall of 20cm are being used in some countries (e. g. DIN 4094 of FRG, and Switzerland). Also, a height of fall of 50cm is used in some instances of the DPSH (e. g. Finland). In France besides the DPSH, the DPA of the ISSMFE European Recommendation Standard is also used a reference test (Proc. IXth intern. Conf. on Soil Mech. And Found. Eng., Vol. III, p. 110, Tokyo, 1977), e. g. the diameter and the shape of the cone are slightly different” states from the IRTP, 1989.

In regard to the extension rods of DSPH, the IRTP 1989 recommends to increase the OD from 32 to 36mm (this suggestion comes from France, Spain and Sweden). In the case of DPL, DPM and DP, the numbers of blows are occasionally counted per 0,2m depth interval of penetration. For DPSH, a 0,3m depth interval is used occasionally.

4.2.2 Choosing the right equipment

The type of penetrometer, which should be used for investigations, is strongly influenced by the following factors:

- Topography (accessibility of the area of investigation)
- Geology (general information about soil types etc.)
- Necessary depth of penetration
- Necessity of key penetration tests in connection with depth

Considering the tasks and influencing factors mentioned above, the process of selecting the right equipment has to be optimized under economic and technical aspects. Optimization can only be regional (Melzer et al. 1982).

Usually the investigations will start with the simplest equipment available which is suitable for the specific job. General rules concerning, for example, the best penetration equipment depths (DPL: 6 to 10m; DPH: 14-25m; DPSH: 40m) may serve as guidelines. Additional influences on the results of penetration tests, such as skin friction, groundwater level, critical depth, etc. are being considered when selecting the equipment which is first to be used. When first preliminary results of the investigated site are available, the use of additional, maybe more sophisticated equipment which is more suitable for the problems encountered will be taken into consideration (Melzer et al. 1982).

For construction supervision, especially the lighter equipment will be preferred. For instance in the area of road construction, the DPL is mostly used and its results correlate well with the CBR (California bearing ratio, which is used for the evaluation of the mechanical strength of road subgrades) values (compare Melzer et al. 1982).

In most European countries, there is a tendency to use heavier DP equipment, due to the desired ability of penetrating through soft rock extreme rigid layers like glacial gravels. This tendency can be observed in Japan, Canada or the United States (Melzer et al. 2008).

Hence, most correlations were developed on the basis of the DPH or DPSH, which will be the focus in this thesis. However, inter- correlations within the DP equipment (from DPL to DPSH) will be introduced.

4.3 Correlations on DPH

Likewise the SPT, correlations for DP (here on DPH) are provided for the following parameter:

- Relative density of sands,
- Friction angle of sands and silts,
- Undrained shear strength of clay,
- Stiffness modulus of sands and clays;

4.3.1 Relative Density

Table 36: approximate ranges for the relative density of sands based on experience (subdivided into U<3 and U>3)[7]

		Very loose	loose	Medium dense	dense	Very dense
DPH- N10	U<3	0-1	1-4	4-13	13-24	----
	U>3	<5	5-15	16-30	30-40	----
I_D [%]		0-15	15-35	35-65	65-85	85-100

Table 37: empirical values based on experience for pile foundation applications[8]

	Very loose	loose	Medium dense	dense	Very dense
DPH- N10	<4	4-9	8-18	14-30	>25
I_D [%]	0-15	15-35	35-65	65-85	85-100

Table 38: correlations on relative density of sands (DIN 4094-3)

		Uniformed- graded sands U≤3	**Well graded gravel- sand mixtures U≥6**
Relative density of sands	DPH:	$I_D = 0{,}1 + 0{,}435 * \log(N_{10})$	$I_D = -0{,}14 + 0{,}550 * \log(N_{10})$
	DPL:	$I_D = 0{,}15 + 0{,}26 * \log(N_{10})$	Not applied

	Very loose	loose	Medium dense	dense	Very dense
I_D [%]	0-15	15-35	35-65	65-85	85-100

4.3.2 Friction angle on sands and silts

Table 39: effective friction angle of coarse grained soil as a function of the relative density and coefficient of uniformity (EN 1997- 2, G2, Dynamic Probing)

Soil type	**Grain size distribution**	**Relative density ID [%]**	**Friction angle [°]**
Fine grained sand, sand, sand and gravel	Uniformed graded (U<6)	15- 35 (loose)	30
		35-65 (medium dense)	32.5
		>65 (dense)	35
Sand, sand and gravel, gravel	Well graded (6≤U≤15)	15- 35 (loose)	30
		35-65 (medium dense)	34
		>65 (dense)	38

[7] http://www.gb.bv.tum.de/download/skript/vorl-g-e.pdf, TUMünchen, lecture notes from Prof. Vogt
[8] John Wiley & Sons, EA-Pfähle, Empfehlungen des Arbeitskreises „Pfähle", Ernst& Sohn, Berlin 2007

Table 40: recommendation from *Bundesanstalt für Wasserbau*[9]

DPH N10	φ´ [°]
<2	<30
2-6	30-35
6-11	35-37.5
11-19	37.5-40
>19	≥40

4.3.3 Correlation to the undrained shear strength

Stiff clays (Butcher et al 1995) in kPa:

$$c_u = (\frac{q_d}{22})$$

Soft clays (Butcher et al 1995) in kPa:

$$c_u = \left(\frac{q_d}{170}\right) + 22$$

Table 41: approximate values based on experience for standard applications, according to DIN 18122 (TU München, lecture notes from Prof. Vogt [10])

according DIN 18122		c_u [kN/m²]	DPH N10 value
Consistency	**Ic**		
Breiig	0-0.5	<20	<2
Weich	0.5-0.75	20-60	2-5
Steif	0.75-1	60-200	5-9
Halbfest- fest	>1	200-400	9-17
	>1.25	>400	>17

4.3.4 Constrained modulus

According to Bundesanstalt für Wasserbau:

Table 42: recommendation from *Bundesanstalt für Wasserbau*[11]

DPH N10	E_S [MN/m²]
<2	<15
2-6	15-50
6-11	50-80
11-19	80-100
>19	≥100

[9] http://www.baw.de/de/die_baw/publikationen/merkblaetter/index.php.html
[10] http://www.gb.bv.tum.de/download/skript/vorl-g-e.pdf
[11] http://www.baw.de/de/die_baw/publikationen/merkblaetter/index.php.html

According to DIN 4094- 3:

$$E_s = v * p_a * [\frac{\sigma_{ü} + 0{,}5 * \Delta\sigma_z}{p_a}]^\wedge w$$

With:

The vertical stresses at the foundation level $\sigma_{ü} = \gamma * (d + z)$

E_S= constrained modulus

v= stiffness factor

w= stiffness exponent (w=0,5 for sands, and w=0,6 for clays of low and medium plasticity)

p_a= atmospheric pressure (p_a=0,1MPa)

$\Delta\sigma_z$= increase of the vertical stresses at the foundation level due to loading;

Above groundwater table, uniformed staged sands: w=0,5

DPH: ($3 \leq N_k \leq 10$)

$$v = 249 * \log(N10) + 161$$

DPL: ($4 \leq N_k \leq 50$)

$$v = 214 * \log(N10) + 71$$

Above groundwater table, clay of low and medium plasticity, w=0,6 ($0{,}75 \leq I_C \leq 1{,}3$), S_R~0,8

DPH: ($6 \leq N_k \leq 13$)

$$v = 6 * \mathrm{N10} + 50$$

DPL: ($6 \leq N_k \leq 19$)

$$v = 4 * \mathrm{N10} + 30$$

4.3.5 Intercorrelations (DPH to DPL)

For correlation within DP equipment, DIN 4094- 3 provides the following:

Uniformed-graded sands, (U≤3) above groundwater table:

$$N_{10DPL} = 3 * N_{10DPH}$$

$$N_{10DPH} = 0{,}3 * N_{10DPL}$$

4.4 Projects

Additional correction on DP:

Besides the correction for skin friction (only) the German standard introduces a correction when DP is conducted below groundwater in sand or gravels.

The DIN 4094-3 states that in coarse grained soils, the penetration resistance in the same soil is lower below the groundwater table, than above. In contrast for fine grained soils the N10 is similar or shows even higher N10 values due to the capillary actions.

Uniformed-graded sands (U≤3, 3≤Nk≤50)

DPH:

$$N_{10\ddot{u}} = 1{,}3 * N_{10u} + 2$$

DPL:

$$N_{10\ddot{u}} = 2 * N_{10u} + 2$$

Well graded gravel- sand mixtures (U≥6, 3≤Nk≤50)

DPH:

$$N_{10\ddot{u}} = 1{,}2 * N_{10u} + 4{,}5$$

Note: ü= above groundwater table, u= below groundwater table;

4.4.1 Project 12

The DP (used equipment: DPH) was conducted close to a borehole (compare Figure 128), where different layers were identified in terms of soil type. Additional SPT measurements have been performed. Torque measurements haven´t been conducted in Project 12. Hence, the skin friction correction will be neglected.

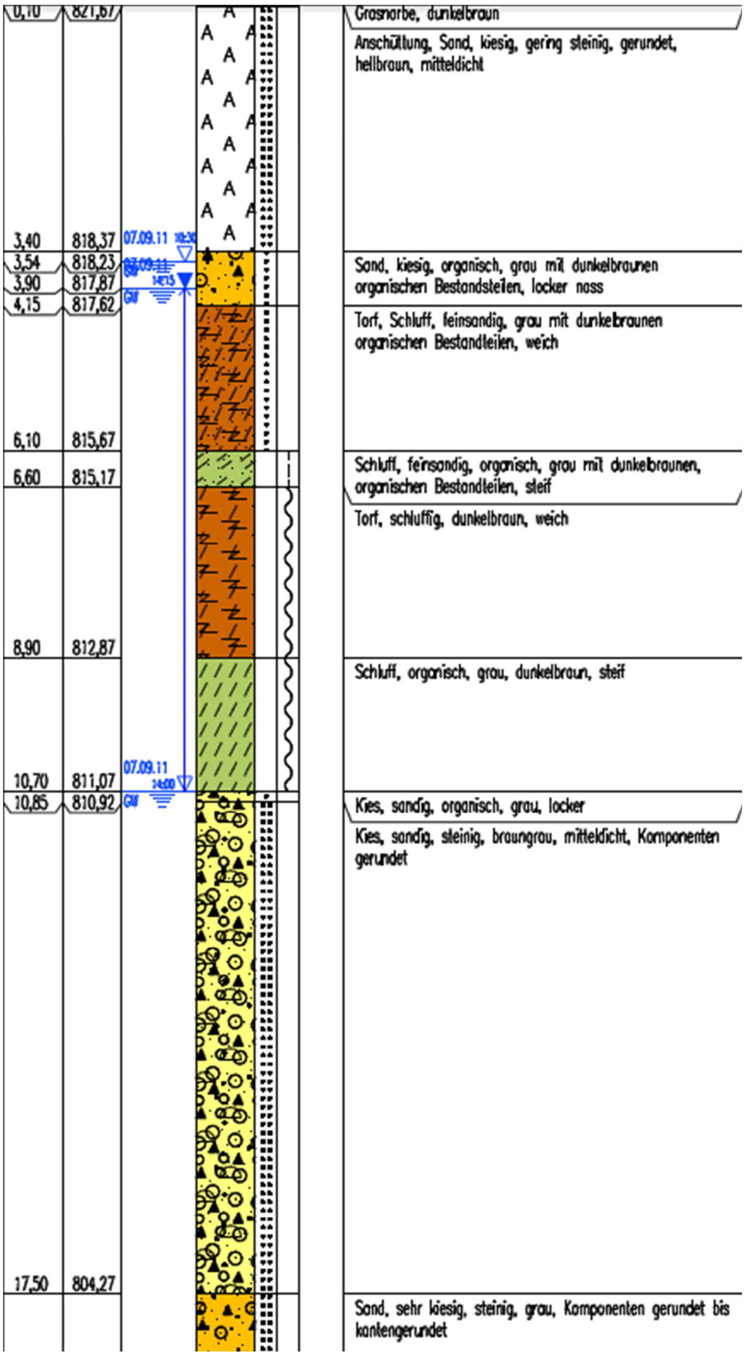


Figure 128: Borehole of project 12;

Parameters from the geotechnical report (for further calculations) will be compared with the derived correlations from DPH. E_S and c_u have been derived using CPT- tests. Information on the test method of determining the effective friction angle was not mentioned in the geotechnical report.

Table 43: description of the subsoil with geotechnical parameters;

Depth [m]	Soil type	φ´ [°]	E_S [MPa]	c_u [KPa]
0- ~3,4	Sand/ gravel	30-35	20-40	0
3,4-~10,7	Peat, silt	18-22	2- 10	2,5- 7,5
10,7-~20	Sand/ gravel	35- 40	80- 100	0

Correction on groundwater in sands/ gravel:

The first two figures show the effect on the N value, when applying the groundwater correction in sand- gravel mixtures (according to DIN 4094-3). The difference is up to 20 blows (in a depth of 13.9m), which effects results of further correlations. One example on the relative density is shown in Figure 129.

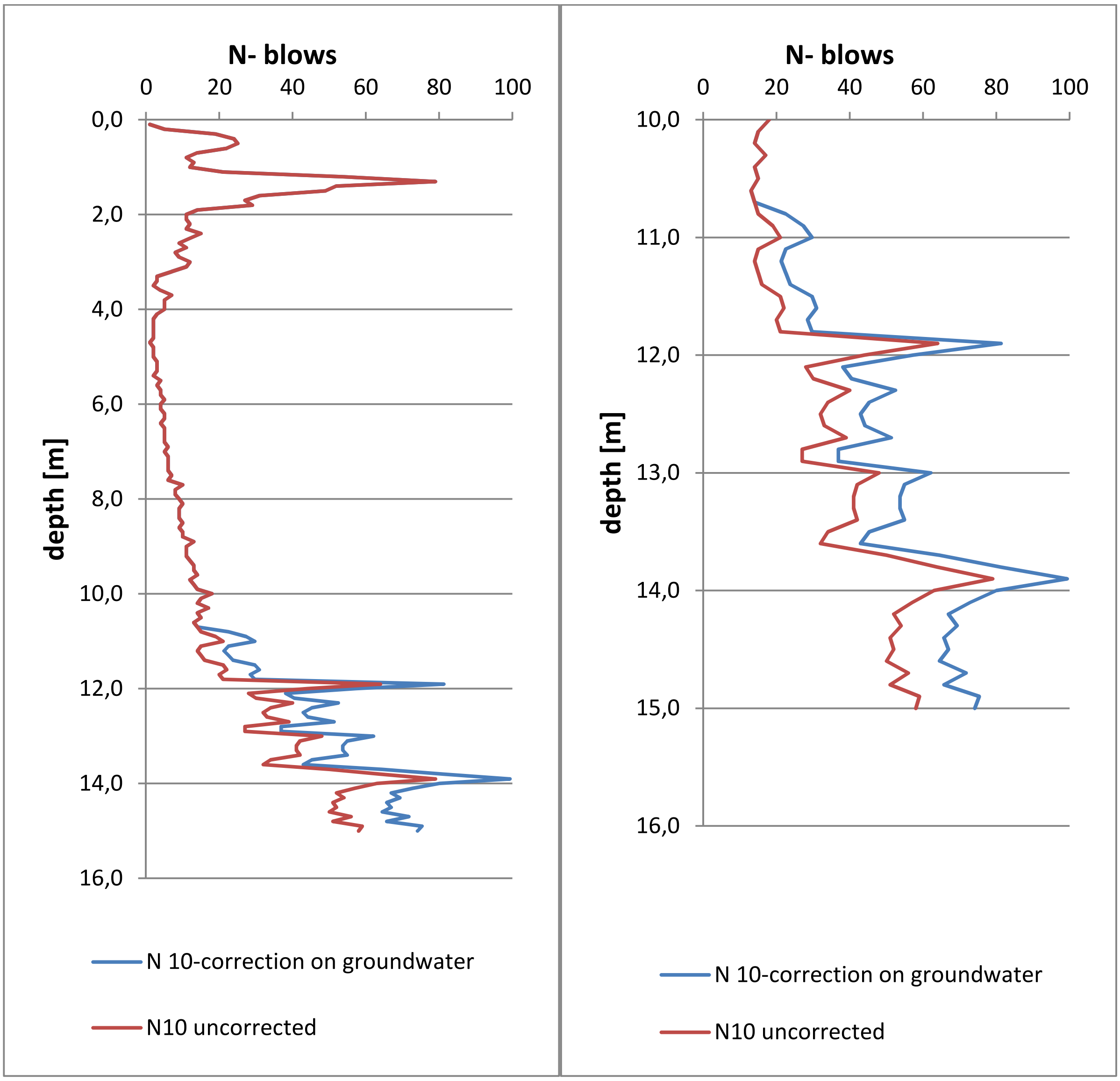


Figure 129: Corrected and uncorrected SPT N;

4.4.1.1 Dynamic point resistance and unit point resistance (q_d and r_d)

The proposed conversion to the unit point resistance and dynamic point resistance will be presented here:

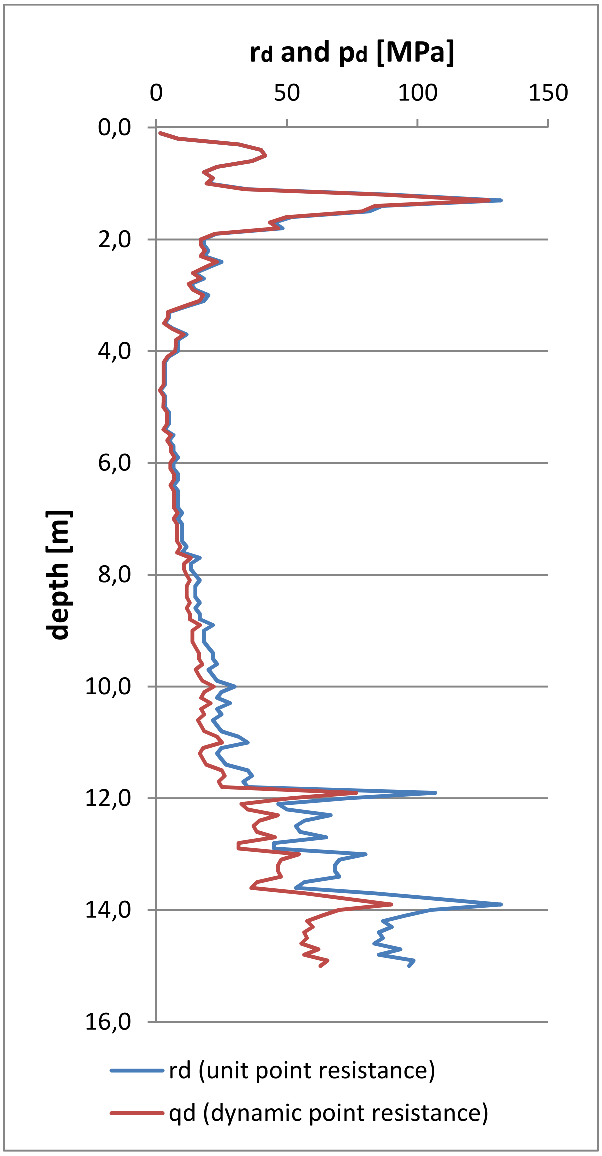


Figure 130: Unit- and dynamic point resistance at project 12;

4.4.1.2 Relative density

As, there is no information in the coefficient of uniformity provided for the first 5 meters, evaluation of the relative density in sands will be conducted for well graded and uniformed graded sands.

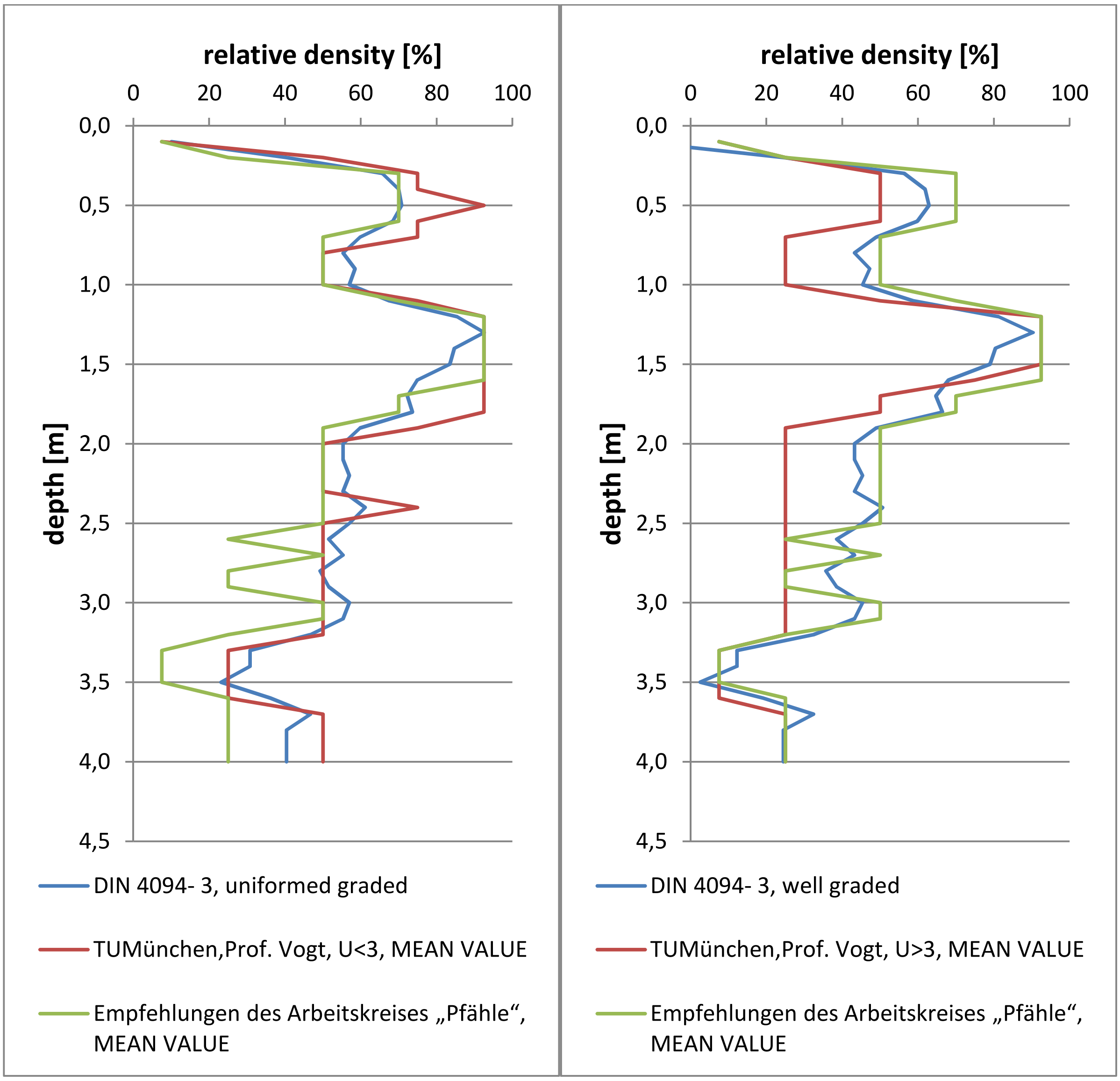


Figure 131: Evaluation of relative density in (left: uniformed graded and right: well graded) sands above groundwater level;

Figure 132 shows the effect of the applied correction for groundwater in a depth of 10,7- 15m for well graded sand/ gravel mixtures:

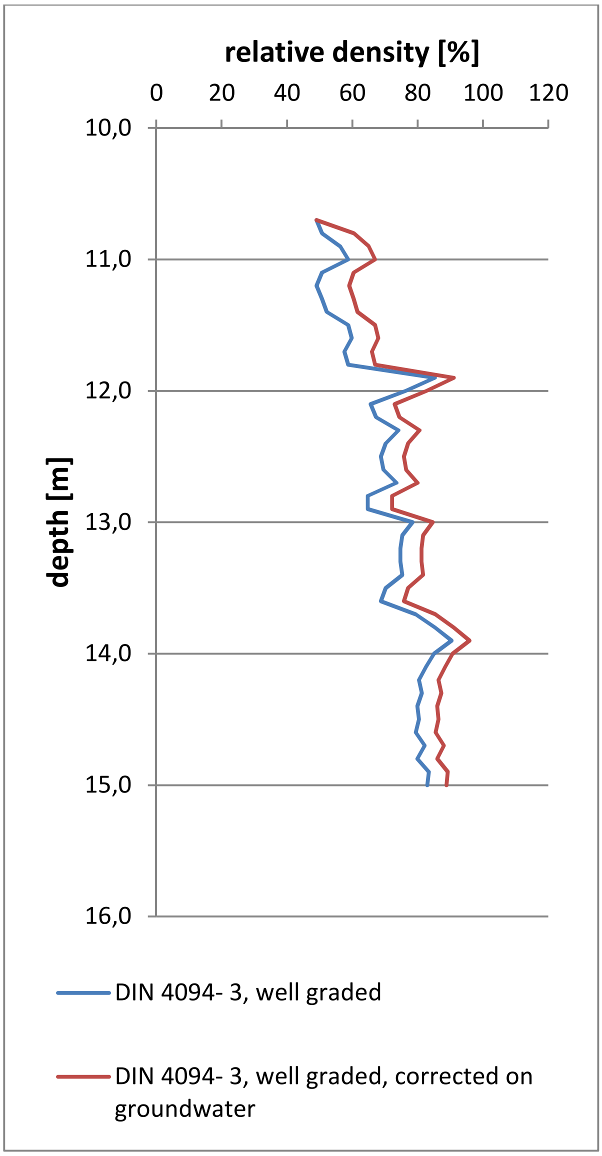


Figure 132: comparison of the evaluation for relative density without/ with groundwater correction;

4.4.1.3 Friction angle in sands and silts

Two correlations have been found for the friction angle φ´. The En 1997-2, G2 is derived from the relative density and distinguishes in uniformed graded and well graded sands. This correlations and the proposed range of 30-35° (for a depth from 0 to 4m, as stated in geotechnical report) for the friction angle show good accordance with the approach from EN 1997-2, G2. The recommended values from "*Bundesanstalt für Wasserbau*" are somewhat too high.

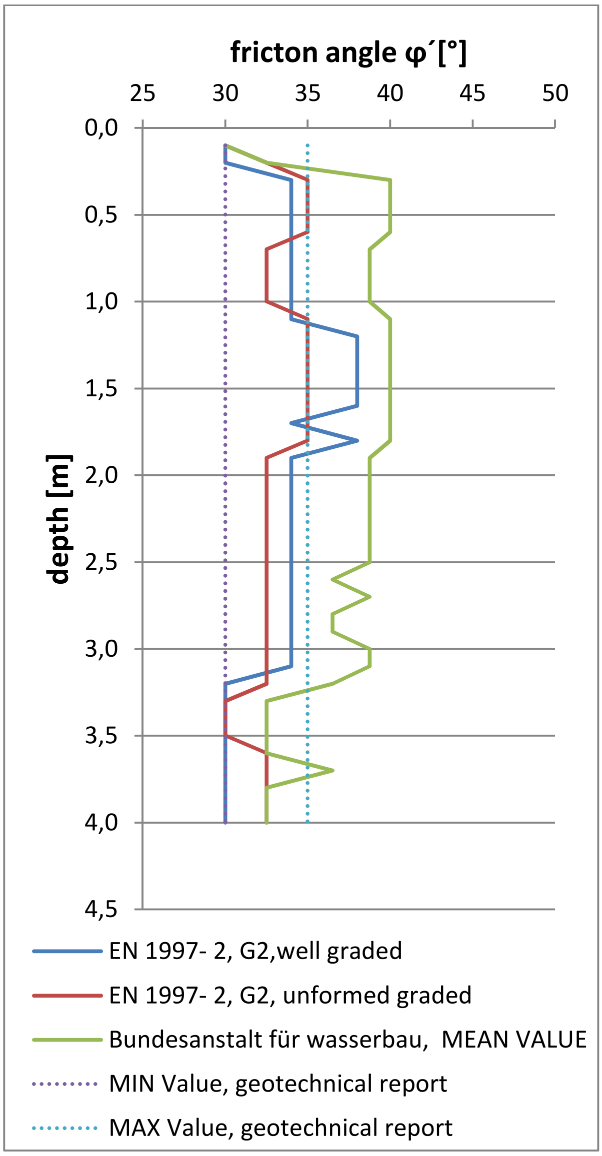


Figure 133: Comparison of correlations on the effective friction angle in sands

4.4.1.4 Constrained modulus

Also for the constrained modulus one correlation is provided in the Din 4094-3, but is just applicable to uniformed graded sands. This correlation and the proposed range of 20- 40MPa (for a depth from 0 to 4m, as stated in geotechnical report) show good accordance.

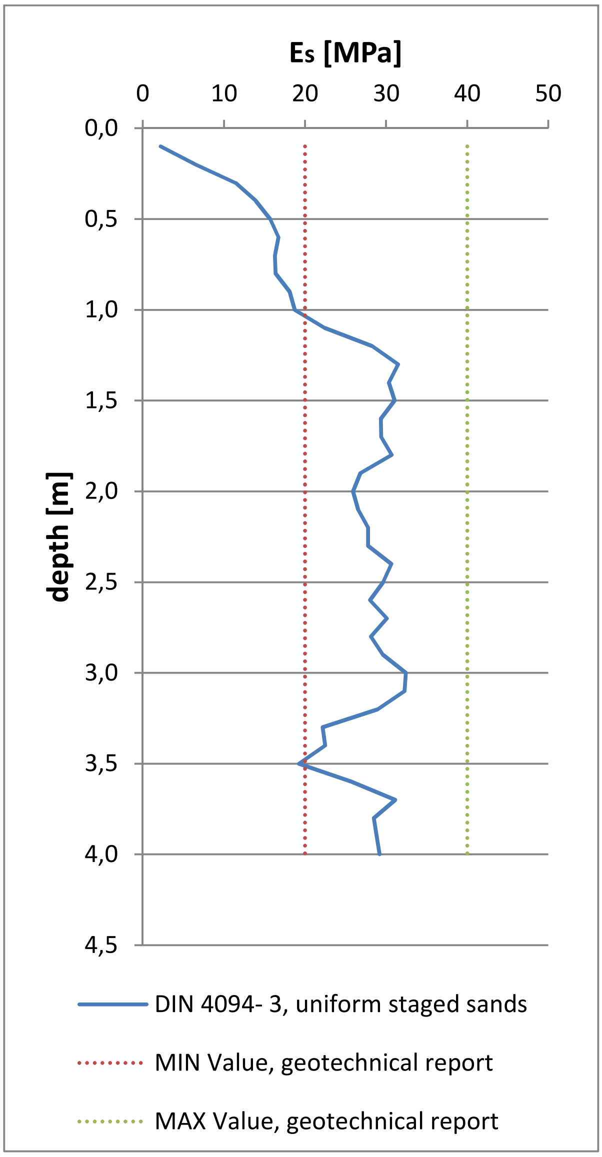


Figure 134: evaluation of the constrained modulus in sands (according to DIN 4094-3);

4.4.2 Conclusion

Torque measurements have been neglected in those DP- projects, which should be used to determine the effect of skin friction. The correction of N_{10} values in groundwater for sands should be taken into consideration, as there are differences in the evaluation of geotechnical parameter.

Generally, correlations to geotechnical parameter show good results for sands and clays. Correlation in other (intermediate) soils should be treated carefully.

As valid for all In- situ tests, knowledge about the subsoil in a certain area is of significant value.

4.5 Literature on DP

4.5.1 Papers

Butcher A.P, K McElmeel, J J M Powell, DYNAMIC PROBING AND ITS USE IN CLAY SOILS, ADVANCES IN SITE INVESTIGATION PRACTICE. PROCEEDINGS OF THE INTERNATIONAL CONFERENCE HELD IN LONDON ON 30-31 MARCH 1995, p. 383-95;

Burnham T. and D. Johnson, In Situ Foundation Characterization, Using the Dynamic Cone Penetrometer, Final Report, 1993, Minnesota Department of Transportation Office of Materials Research and Engineering;

Melzer K. J. et al: Dynamic penetration testing- State of art report, Proceedings of the second European symposium on penetration testing/ Amsterdam/ 1982;

K. J. Melzer, U. Bergdahl, Ed. Fecker, Grundbautaschenbuch, Grundbau-Taschenbuch, Teil 1 Geotechnische Grundlagen, edited by Witt K J, Ernst & Sohn, 7. Auflage, Berlin 2008;

Card G. B., Roche D. P., Herbert S. M., Applications of continuous dynamic probing in ground investigation; Field Testing in engineering geology, Proceedings of the 24th Annual Conference of the Engineering Group of the Geological Society Sunderland Polytechnic, 4- 8 September, 1998;

H. M. ABUEL-NAGA, M. HOLTRIGTER and M. J. PENDER, Simple method for correcting dynamic cone penetration test results for rod friction, Geotechnique Letters 1, 37–40, 2011;

Max Cope, Dynamic probe theory revisited, - effective interpretation of dynamic test results, Auckland New Zealand, 2010; http://www.docstoc.com/docs/77766375/DYNAMIC-PROBE-THEORY-REVISITED

4.5.2 Standards and Guidelines

ASTM D 6951 – 03, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications;

BS 1377 Part 9, 1990, Soils for engineering purposes- DP;

ISSFME 1989, International reference test procedure for Dynamic Probing (DP)

EN ISO 22476- 1, Geotechnical investigation and testing -- Field testing -- Part 2: Rammsondierungen;

DIN 4094- 1, 2002- 06 Baugrund - Felduntersuchungen - Teil 3: Rammsondierungen;

5. Correlations between DP and SPT

Correlation provided (SPT and DPH):

(Butcher et al. 1995):

$$`N` = 8 * N_{10} - 6$$

(DIN 4094- 3):

Coarse grained soils above Groundwater

$$N_{30} = 1{,}4 * N_{10}$$

$$N_{10} = 0{,}6 * N_{30}$$

Clay of low to medium plasticity:

$$N_{30} = 1{,}0 * N_{10} + 3$$

$$N_{30} = 0{,}5 * N_{10} + 2$$

Project 12

At Project 12, DP and SPT have been positioned nearby.

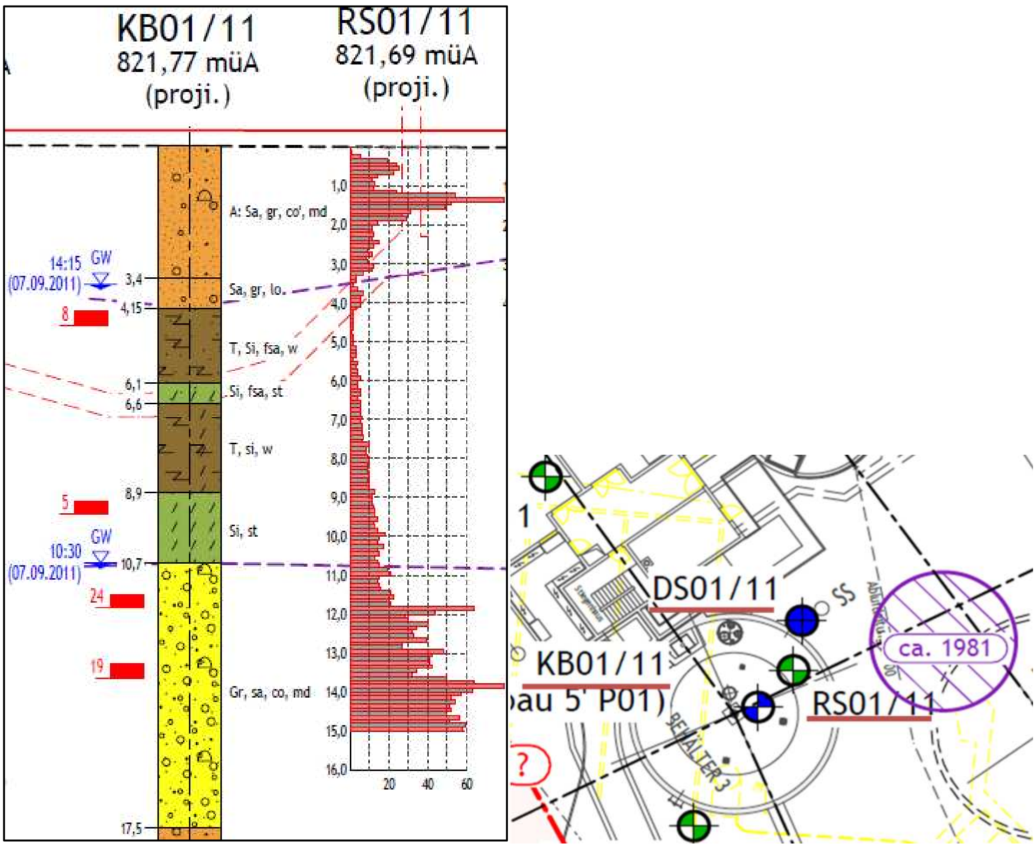


Figure 135: left: SPT measurements and DP; right: locating of the tests;

In the following part, correlations for the friction angle and the constrained modulus (from DP, SPT) will be compared in the sand/ gravel layer (depth 11- 15m).

The description of that layer is the following (according to the geotechnical report)

Table 44: description of geotechnical parameter;

Depth [m]	Soil type	φ´ [°]	Es [MPa]	c_u [KPa]
10,7-~20	Sand/ gravel	35- 40	80- 100	0

A comparison of the converted N value (from DP in sands using DIN 4094- 3) will be presented in the next figure:

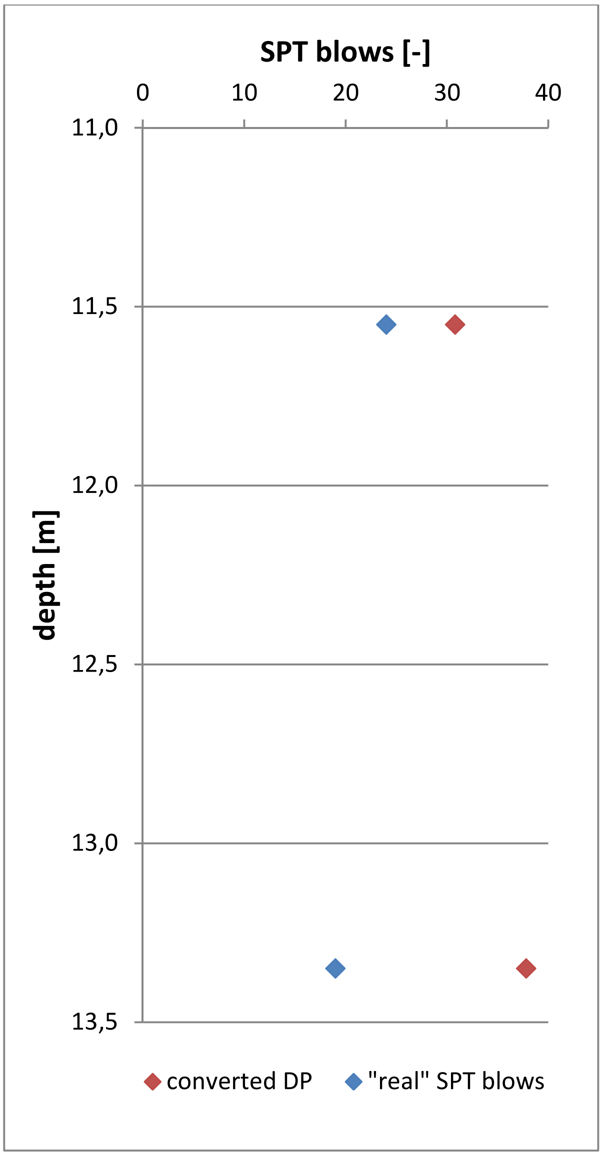


Figure 136: Comparison of the "real" SPT blows with the converted SPT blow from DP;

Note: Here both values have not been corrected for a certain energy ratio or account for overburden correction. The comparisons show big differences- especially for the depth of 13,35m.

The results of geotechnical parameter (c_u and E_S), including the proposed values from the geotechnical report, are shown in Figure 137.

Table 45: used SPT and DP correlations;

	SPT correlation	DP correlation
φ´	Mayne (1998)	EN 1997- 2, G2
E_S	DIN 4094-2	DIN 4094-3

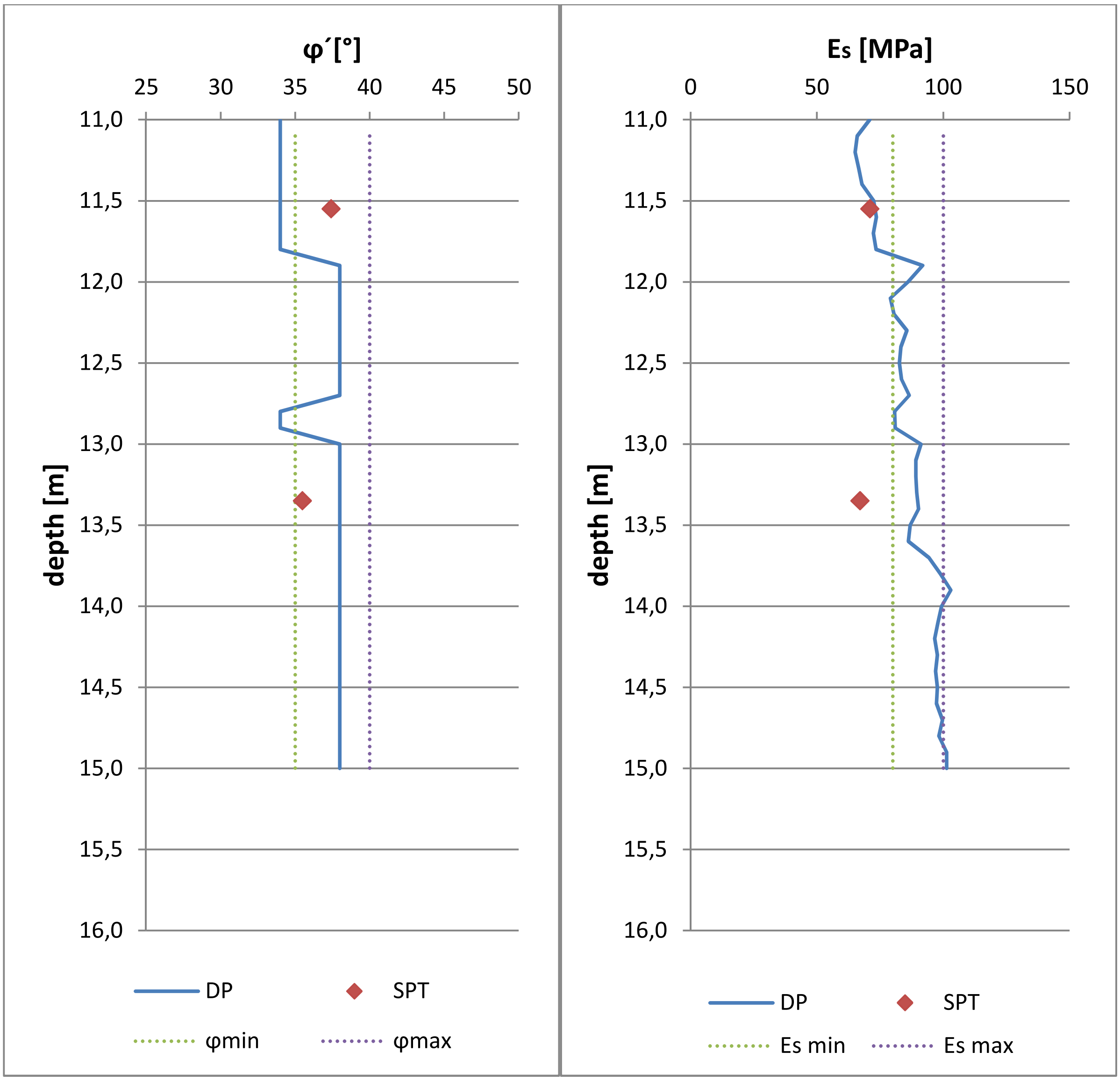


Figure 137: Comparison of DP and SPT results for the friction angle (left) and constrained modulus (right) including results from the geotechnical report;

Comparisons of SPT- and DP results show good accordance. They also correlate well with the values from the geotechnical report (Project 12).

6. Correlations between SPT and CPT

Correlations provided:

DIN 4094- 2 (just for sand/gravel):

Uniformed graded sands:

$$q_c = 0{,}5 * N_{30}$$

Well graded sands and well graded sand- gravel mixtures:

$$q_c = 0{,}7 * N_{30}$$

Uniformed and well graded gravels.

$$q_c = 1{,}1 * N_{30}$$

And another correlation (after Elkateb et al 2010):

$$N_{60} = \frac{q_c * 10^3}{8{,}5 * p_a * (1 - \frac{I_c}{4{,}6})}$$

Jefferies and Davies (1993) used for the Ic a modified soil behavioral type index I_C:

$$I_C = \sqrt{\{3 - \log\left(Q * \left[1 - B_q\right] + 1\right\}^2 + \{1{,}5 + 1{,}3 * \log(F)\}^2}$$

Jefferies and Davies (1993) even suggested that the above methods provide a better estimate of the SPT N values than the actual SPT test due to the poor repeatability of the SPT (Lunne et al 1997).

Wheras Robertson & Wride, 1998 used there soil behavioral type index I_{CRW}:

$$I_c = ((3{,}47 - \log(Q_t)^2 + (\log(F_r) + 1{,}22)^2))^{0{,}5}$$

Besides that, there are others correlations that require additional information like the Fines content (FS) or the mean particle size (D50 [mm]), that could be obtained from laboratory tests.

Project 11

SPT and CPT results will be compared in the Project 11 where these two tests have been conducted nearby. Constrained modulus, undrained shear strength and the equivalent SPT N value will be compared.

Table 46: corrected SPT blows

depth [m]	(N1)60 corrected [ER=60%]
5.00	5
8.00	2
12.00	1
15.00	11

Correlations proposed by DIN 4094-2, are verified in a depth of 13 to 16m (the only sand/gravel layer where a SPT test was performed). The corrected (ER=60%) N value at a depth of 15m is 11, which will be assumed as constant over the thickness of that layer (compare Figure 138, left).

The comparison of the “Equivalent SPT values” (Robertson & Wride, 1998) and “real” measured N values is presented in the next figure.

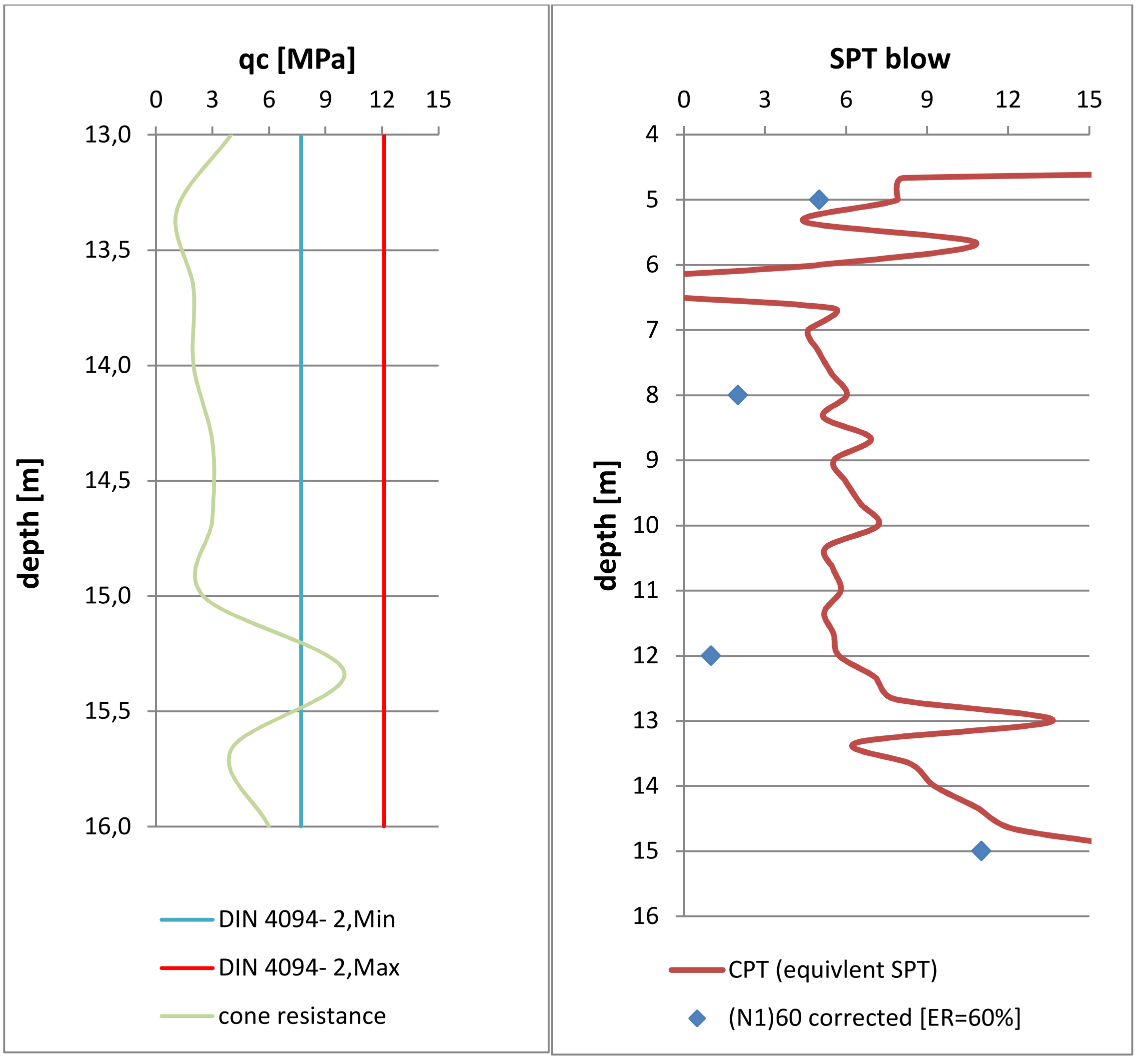


Figure 138: Evaluation of qc (left) and SPT blow (right);

Also the results of geotechnical parameter will be compared:

Table 47: used correlations

	CPT MIN	CPT MAX	SPT MIN	SPT MAX
Es	DIN 4094-1, D5		DIN 4094- 2	Bundesanstalt für Wasserbau
c_u	Aas et al. 1986, Nk=8	Aas et al. 1986, Nk=16	DIN 18122	Tschebotariff (1973)

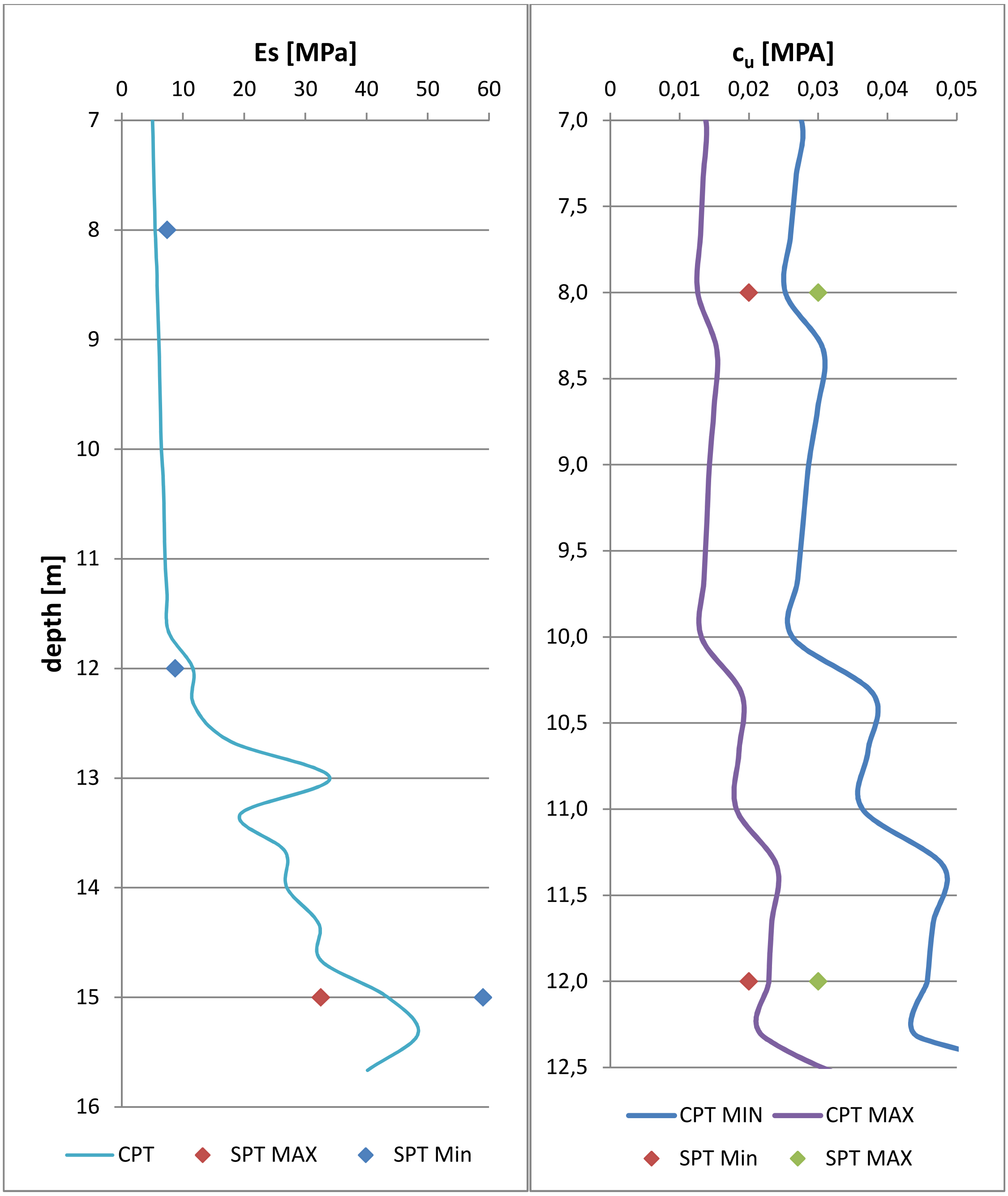


Figure 139: comparison of CPT and SPT results for the constrained modulus (left) and the undrained shear strength (right);

Note: It´s very difficult to compare the two Insitu- tests because of the uncertainty of the corrected N60- value (concerning correction factors and energy ratio).

7.Correlations between DP and CPT

Correlations provided (DP to CPT)

Clay:

Soft clay (Butcher et al 1995):

$$q_t = 0{,}24 * q_d + 0{,}14$$

Stiff clay (Butcher et al 1995):

$$q_t = q_d$$

Sand (DIN 4094-3):

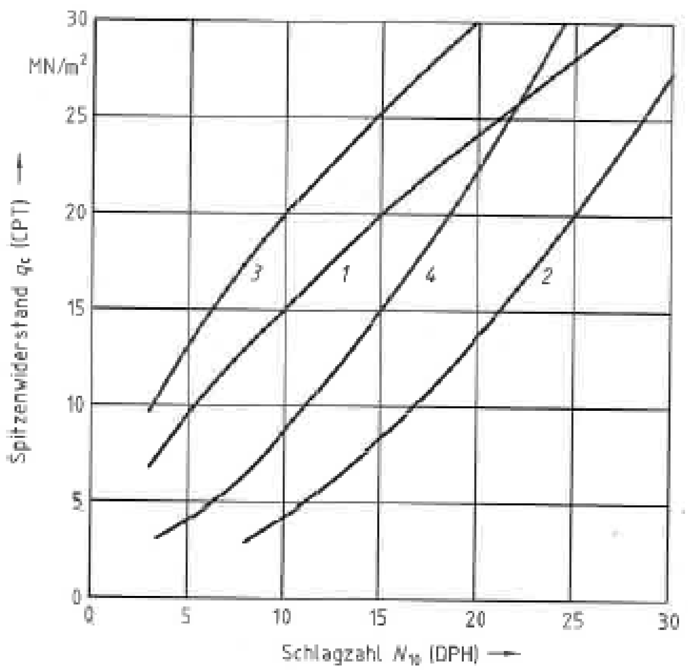


Figure 140: correlation of CPT and N10 value from DPH for various sands; 1: uniformed graded sands, 2: Well graded sand- gravel mixtures, and well graded gravel- sand mixtures, 3: Uniformed graded sands, 4: Well graded sand- gravel mixtures and well graded gravel- sand mixtures, DIN 4094- 3;

Unfortunately projects, where DP and CPT were conducted nearby, haven't been available.

8. List of Figures

9. List of Tables