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Graz University of Technology

Settlement estimation for auxiliary bridge foundations

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Error Magnitude in the constrained modulus

Master thesis

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Maximilian Weiß, B.Sc

Matr.-Nr.: 1231624

At the Technical University of Graz

Supervisor 1 University:	**Marte** Roman, Univ.-Prof. Dipl.-Ing. Dr.techn.
Supervisor 2 University:	**Liu** Qian, Ao.Univ.-Prof. Mag.rer.nat. Dr.rer.nat.
Supervisor Company:	**Willerich** Sebastian, Dipl. Dr.rer.nat.

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Preface of the author

In this thesis, a lot of people have been encouraging and supporting me. I would like to express my thanks to the following peoples.

I am very much indebted to my supervisors Professor Dr. Roman Marte and Professor Dr. Qian Liu for their continuous guidance through this research study and their understanding for the situation to write a thesis in a company. I am especially thankful to Dr. Sebastian Willerich and his colleague Emanuel Fritsch for their support and their will to show me everything in the work environment. Dr. Sebastian Willerich provided me with the data for this research study and always had an answer for my questions. I had a great time at the company Max Bögl and learned a lot, besides the topic of my thesis.

I would also take the opportunity to pay my thanks to my parents who gave me the possibility to study, my family and friends for their support throughout my thesis.

Kurzfassung

Diese Masterarbeit befasst sich mit den verschiedenen Ermittlungsmethoden des Steifemoduls aus Felddaten, welche im vorliegenden Fall zur Setzungsabschätzung von Hilfsfundamenten, die beim Bau von Brücken in der Verschubtechnik erforderlich sind. Die Untersuchungsmethoden wie Drucksondierungen (CPT), Bohrlochrammsondierungen (BDP, SPT) und Rammsondierungen (DP) werden begutachtet und mit den Baustellendaten der Firma Max Bögl wird gezeigt wie groß die Fehlerquellen beim Errechnen des Steifemoduls sind. Der Einfluss der Fehlergröße wird anhand von Setzungsberechnungen von Hilfsfundamenten an einem Baustellenbeispiel gezeigt.

Schlüsselwörter: *Steifemodul, CPT Datenberechnung, DPH Datenberechnung, Setzung von Hilfsfundamenten*

Abstract

This Master thesis covers up the different ways of determining the constrained modulus out of field data, which is used in the present case for settlement estimations of auxiliary foundations which are necessary to construct bridges with the roller launch method. The sampling methods Cone penetration test (CPT), Standard penetration test (SPT) and Dynamic Probing (DP) are investigated and with the construction site data of the company Max Bögl is shown how big the error magnitude in determining the constrained modulus is. The influence of the range is shown in a settlement analysis of auxiliary foundations on a construction site example.

Keywords: *Constrained modulus, CPT data, DPH data, Settlement of auxiliary foundations*

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List of symbols and abbreviations

Capital letters

C_c	Compression index
C_r	Recompression index
C_s	Swell index
E_b	Constrained modulus of the foundation
E_s	Constrained modulus
E_v	Constrained modulus of the Plate load test, “Verformungsmodul”
I_c	Soil type behavior index
K	Value which describes the interaction between soil and foundation
M_e	Swiss constrained modulus, “Zusammendrückungsmodul”
N_{10}	Number of blows for 10cm, DP
N_{20}	Number of blows for 20cm, DPSH
N_{30}	Number of blows for the last 30cm, SPT
R_f	Friction ratio, CPT

Small letters

a	Area ratio, CPT
e	Void ratio
f_s	Sleeve friction, CPT
h	Height
l	Length
p_a	Atmospheric pressure
q_c	Cone tip resistance, CPT
q_t	Corrected tip resistance, CPT
u	Pore water pressure, CPT
v	Stiffness factor
w	Stiffness exponent (w=0.5 for sands, and w=0.6 for clays of low and medium plasticity)
w_L	Water content

Greek letters

α	Regional factor
$\sigma_{ü}$	Vertical stress at the foundation level
$\Delta\sigma_z$	Increase of the vertical stresses at the foundation level due to loading
φ	Friction angle of the soil

Abbreviations

BDP	Borehole Dynamic Probing, German abbreviation for SPT
CPT	Cone Penetration Test
CPTU	Cone Penetration Test with pore water measurement
DIN	German Institution for standardization
DPH	Dynamic Probing heavy
DPSH	Dynamic Probing super heavy
DPL	Dynamic Probing light
DPM	Dynamic Probing medium
EN	European Institution for standardization
NC	Normal consolidated
OC	Over consolidated
OCR	Over consolidation ratio
SPT	Standard Penetration Test
UC	Under consolidated

1 Introduction

To write a thesis with a practical and economic background was very important for me, so I was lucky to find a company, which offered me exactly that possibility. Max Bögl is one of the largest construction companies in Germany and also the initiator of the topic of this thesis. During an internship I was able to collect the necessary data from their construction sites and I was shown the work environment. The Max Bögl Group is a German construction company which operates on a global scale. They employ over 6000 workers (2014) and thus are one of the largest employers in the Neumarkt region, located 30km east of Nürnberg. The annual turnover is about € 1.6 Billion (2014), making it the fourth biggest company in the German construction sector. They are working at several large construction site projects all over the world. One branch of business is the construction of large bridges.

Fig. 1 Bridge construction with the auxiliary foundations, Schierstein Bridge between Mainz and Wiesbaden, Germany

Nowadays steel parts are often used instead of concrete elements because of the weight. The prepared steel parts are erected on a supporting structure, which are founded on auxiliary foundations. In order to guarantee a safe and functional slide of the bridge construction they need stable auxiliary foundations on which they move the different parts into the right position.

The design and dimensioning of these auxiliary foundations causes problems because of uncertainties in the determination of the constrained modulus, which is an important parameter for the settlement estimation of these foundations. This leads to calculation problems of the dimensions of the auxiliary foundations and the prediction of the settlement. In the past it became obvious that by determining the constrained modulus of the soil with common in-situ and laboratory methods the respective results of the calculations were not representative and that normally the stiffness of the ground is underestimated, as described in *Soumaya (2005).*

The constrained modulus is needed for the standard calculation for a settlement analysis, since it describes the compaction behavior of soil when a force is acting upon it.

The E_s-modulus can have a large variability caused by the fact that a proper evaluation of the constrained modulus with laboratory and in-situ tests is quite difficult, because a soil underneath a foundation is normally not homogenous. Therefore the result depends on the composition of the ground and on the location where the soil is tested. These unpredictable aspects make this parameter even more difficult to determine.

2 Aim

For the construction of large bridges, auxiliary foundations are needed. To construct them it is necessary to understand the soil properties which are underneath and to describe the deformation behaviour of this soil due to the loads of the auxiliary foundations.

The aim of this thesis is to find error magnitudes in the different ways of determining the constrained modulus of field tests, literature and the pre examination of a project.

The stiffness-properties of the ground is usually underestimated. This leads to constructions with lower settlements than predicted. Through the data of much larger settlements the soil improvement is calculated to be much higher than actually needed. Soil improvement through additives or any mechanical compaction method leads to costs that are much higher than necessary. The findings of this work should make it possible to give a better prediction or at least a better understanding of the settlements of auxiliary foundations. The findings should lead to a result in a cost reduction for the uneconomically additional constructive and technical execution steps.

The steps of interpreting data and the resulting constrained modulus due to different calculation methods should be examined. Further on a settlement analysis should show the influence of the variability of the constrained modulus. Construction site data of Max Bögl are used to evaluate this process and help to find main error magnitudes.

3 State of the art

For economic and technical reasons the underground on a construction site is tested only partly at random places, which in advance were expected to be the best places to be examined. Starting from the results of these tests a subsurface model is established *(Kempfert & Raithel, 2012).* The state of the soil between the sampling points can only be estimated and so the ground conditions are only known to a certain degree, which leads to a risk known as geological risk. The geological risk gets an even higher meaning in a failure or damage scenario and would lead to bigger consequences.

The resulting damage may have occurred due to mistakes in the interpretation of the subsurface model. The geological risk is the owner's responsibility in Germany *(Prinz & Strauß, 2010).* The owner hires an expert for geotechnics to examine the construction site and minimize this geological risk. By handing out his geotechnical report, the geotechnical expert takes responsibilities for his explanations but not for the geological risk eventually. On this basis of data the construction is normally started. The expert may give advice for the construction when he considers up economically and efficiently aspects, but it must always be in accordance to the standards and the safety requirements.

"The geotechnical report describes the conditions of the ground for the execution of the service of the site. For example, soil and water conditions have to be described so that the contractor can assume the impact on the construction and on the execution. "
VOB, Part A: § 7

The exploration of the subsoil and the geological risk is closely linked. The geological risk is defined in *DIN 4020:*

„Geological risk is to be defined as a residual risk which is naturally found when taking advantage of a construction ground and can lead to unpredictable occurrences or difficulties, even if the construction ground owner has completely acted in accordance with his responsibilities to examine and describe the state of the construction ground as well as the ground water with all technical possibilities, and when the construction executer has followed his own duty of examination and consultation. As an example, the described difficulties can lead to construction damage or a delay in the construction."(Translated from the German *DIN 4020)*

Outcrops are samples, which only permit probability statements for the intervening areas so that the residual risk remains a geological risk. The geological risk becomes real when the contractor has fulfilled every task in his area of responsibility, but for example an unexpected soft layer in the ground leads to higher costs, shortcomings and damages *(Kempfert & Raithel, 2012)*.
These points are described in Germany by Law in §§ 644 and 645 Bundesgesetzbuch (BGB).
A geotechnical report is done by an expert in geology. The results of the investigation and testing of soil and groundwater conditions of a construction area and their evaluation in terms of the solution of a structural object according to the state of the art, contains information about the nature of the soil.
Since 2008 a geotechnical report is required as a basis for planning and execution of a construction in Germany. The base for the report is the *Euro Code 7*.
For design buildings and civil engineering structures, precise statements about the sustainability of the soil are needed for planners, engineers to dimension the foundation elements like piles, a foundation plate or footings. In order to plan the work's execution the contractor needs a document which provides him with a foundation for his work (excavation, groundwater conservation, waterproofing, impact on the environment, etc.).
To provide this data in the geotechnical report the geologist / geotechnical engineer needs several testing devices for the soil. These tests are performed with machinery and may lead to a variation of the testing results caused by the heterogeneity of the soil for instance. That is why most of the data in geotechnical reports are estimated very conservatively. This can lead to higher costs and demands improvements which aren't really necessary.

With the conservative assumptions there is a classical situation where the costs stand "against" the safety factor.

4 Foundations

4.1 Planning a foundation

A building's foundation transmits loads from buildings and other structures to the ground. Engineers design foundations based on the load characteristics of the structure and the properties of the soils at the site. *(Reuter et al, 1992)*

A geotechnical engineer’s tasks are as follows:

- To estimate the magnitude and location of the loads to be supported
- To develop an investigation plan to explore the subsurface
- To determine necessary soil parameters through field and lab testing
- To design the foundation in a safe but economical manner

The primary considerations for foundation design are the bearing capacity, and the ground movement beneath the foundations.

“In areas of shallow bedrock most foundations may act directly on bedrock. In other areas the soil may provide sufficient strength for the support of structures. In areas of deeper bedrock with soft overlying soils, deep foundations are used to support structures directly on the bedrock. In areas where bedrock is not economically available, stiff "bearing layers" are used to support deep foundations instead” *(Fang, 1991).*

There are two types of foundations:

- Shallow foundations
- Deep foundations

Shallow foundations are defined in *Coduto, (1994).*

“Shallow foundations are those that transmit structural loads to the near surface soils. These include spread footing foundations and mat foundations.”

Deep foundations are used to transfer loads from a structure above ground through upper weak strata of soil to a more competent one at depth, due to the fact that a shallow foundation would be both impractical and uneconomic. The most common form of deep foundation is provided by using piles (*Reuter et al, 1992).* Many building codes specify basic foundation design parameters for simple conditions, frequently varying by jurisdiction. But such design techniques are normally limited to certain types of construction and certain types of sites and are frequently very conservative. *(Kempfert & Raithel, 2012)*

4.2 Types of spread footings for bridges

A shallow foundation is a foundation body, which initiates the external loads exclusively or predominantly on a horizontal or inclined bottom surface in the soil. This causes an aerially distributed and mainly vertical ground deformation, which is referred to as the bearing pressure *(Coduto 1994)*. In shallow foundations the building loads are directly transmitted into the stable ground. Shallow foundations shall be designed and measured in accordance to *DIN 1054 (Permissible load of the ground).* The calculation bases include the design dossier of the type, shape and load of the structure and the description of the ground.

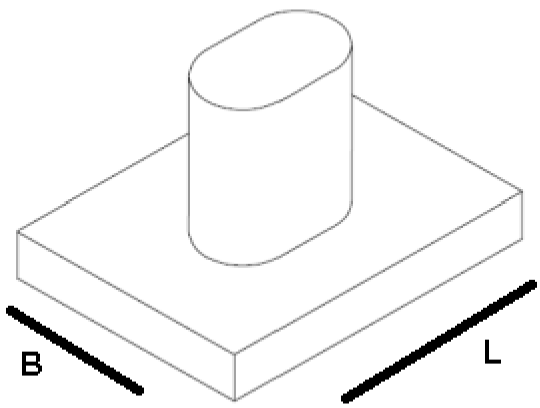


Fig. 2 Isolated spread footing, length (L) to width (B) ratio, L/B<10, modified after *Samanti, (2006)*

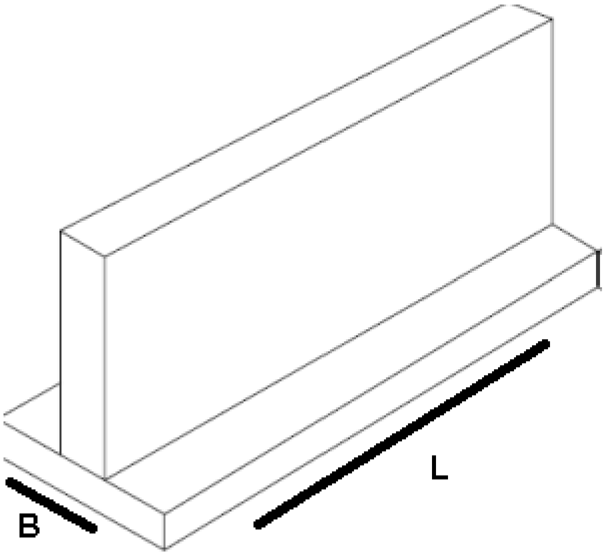


Fig. 3 Strip spread footing, length (L) to width (B) ratio, L/B>10, modified after *Samanti, (2006)*

Spread footings like in Fig. 2 and Fig. 3 are often used for bridges if the construction has a shallow foundation. Even when the bridge is set on deep foundations, shallow foundations are also needed for the supporting structure, visible in Fig. 4. The supporting structures and auxiliary foundations are needed to build up the main construction; afterwards the part of the bridge is adjusted to the right position. This technique to build bridges is called “roller lunch method”. There are several more ways to construct bridges; a good overview is present in *Kahn, (2015).*

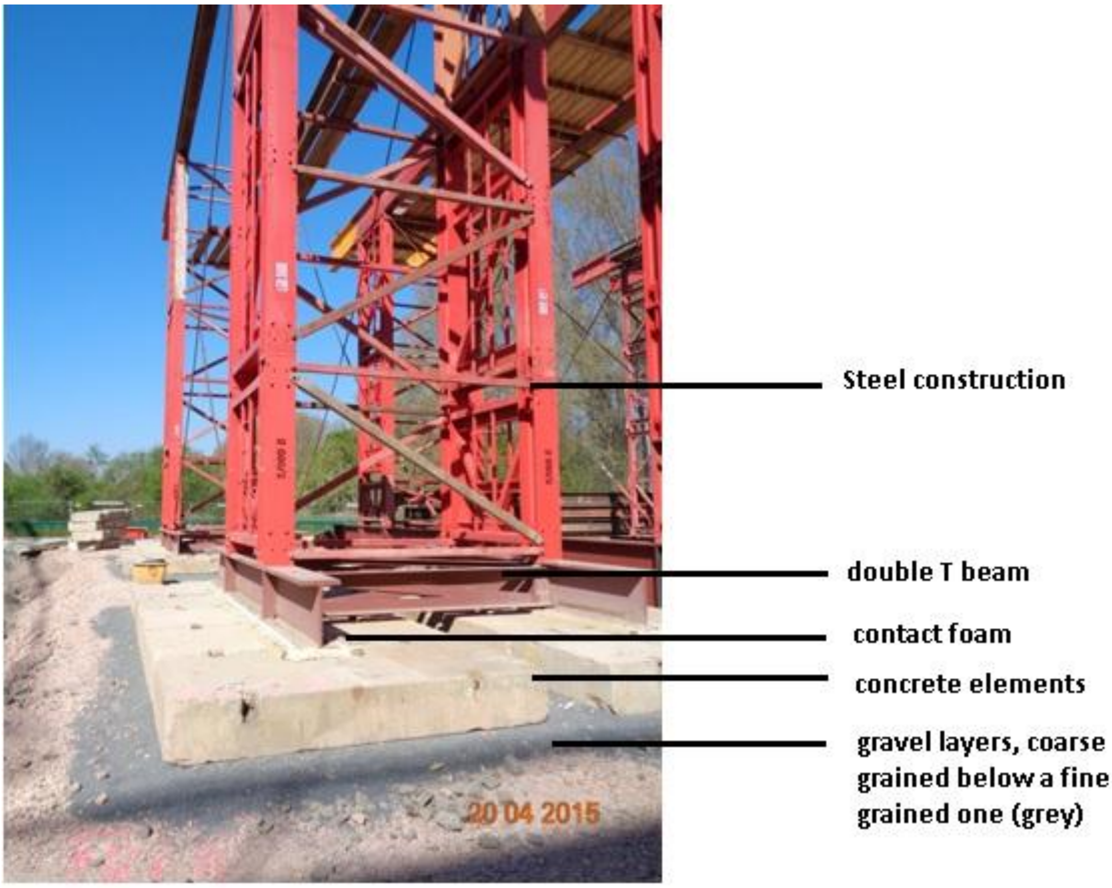


Fig. 4 Auxiliary foundation in detail, Schierstein bridge, Hessen, Germany

In Fig. 4 the basis of an auxiliary foundation can be seen with the supporting structure on top. The concrete foundations are constructed on gravel layers. On top of the concrete elements the steel construction begins. The contact surface between the steel parts and the concrete elements is initially spread with expanding foam to guarantee a homogenous load distribution.

4.3 Loads on foundations and settlement calculation

In Fig. 5 the loading behavior of foundations is presented. The behavior depends on the stiffness of two interacting positions, the soil body and the foundation construction.

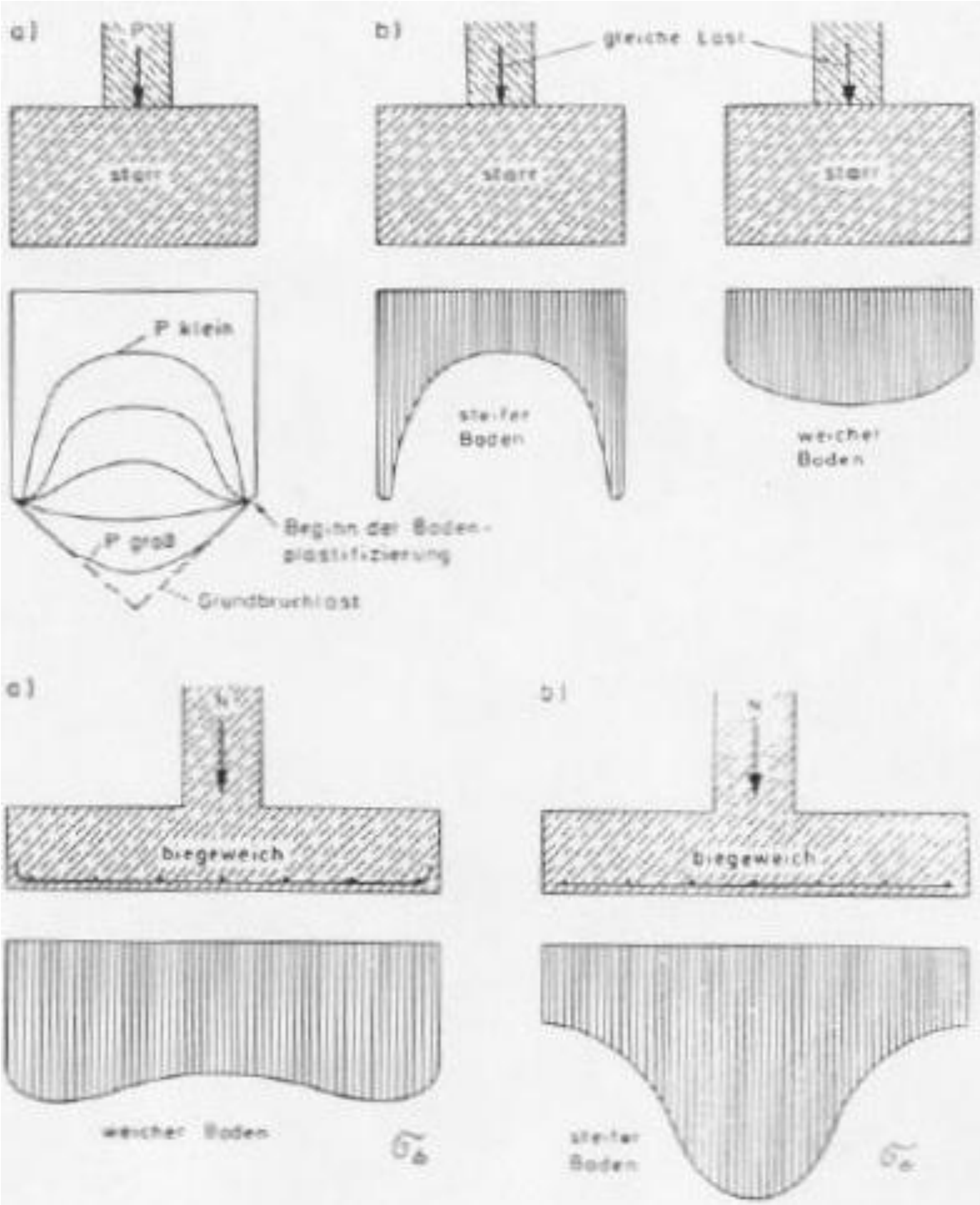


Fig. 5 Loading behavior of a foundation, in upper b) the foundation is stiff and the left one describes the behavior with a stiff soil whereas the right one with a soft soil. In c) the foundation is soft and the soil is soft as well. In lower b) the foundation is soft and the soil stiff. *(Baumgart, 2012)*

Following objections can be made which are described in more detail *in (Huder et al, 2011):*

- With stiff soil and a stiff foundation the result of the theoretical stress peaks at the edges and cannot be compensated by the soil. With rearrangement stress travels to the middle of the foundation. The calculation is made according to *Boussinesq*.
- With soft soil and a stiff foundation the stress rearrangement effect occurs earlier and the stress peak in the middle of the foundation is normally higher than on the edges
- With soft soil and a soft foundation the result is a relatively homogenous distribution of the stress
- The harder the soil the higher the concentration of stress in the middle of the soft foundation.

The different calculations for a soft or a stiff foundation can be evaluated by following equation.

$$K = \frac{stiffness\ of\ the\ foundation}{stiffness\ of\ the\ soil}$$

$$K = \frac{1}{12}\frac{E_b}{E_s}\left(\frac{h}{l}\right)^3$$

E_b= constrained modulus of the foundation

E_s= constrained modulus

h = height of the foundation

l = length of the foundation

K ≥ 0.1	stiff foundation
0.001 ≤ K ≤ 0.1	boarder area
K ≤ 0.001	soft foundation

Tab. 1 K-value which describes the behavior between soil and foundation, *(Bowles, 1982)*

This is the basic calculation for the decision of how the foundation and the soil interact.

Auxiliary foundations are evaluated as stiff foundations which is important to the later settlement calculation. The part of the settlement which results from the load of the foundation is calculable after *DIN 4019*. The soil settlement represents the decisive intermediate value for the calculation of the foundation parameters. For the stress distribution due to an additional load, is here idealized the soil as a homogenous half space with a linear elastic, isotropic material. (*Boussinesq* model)

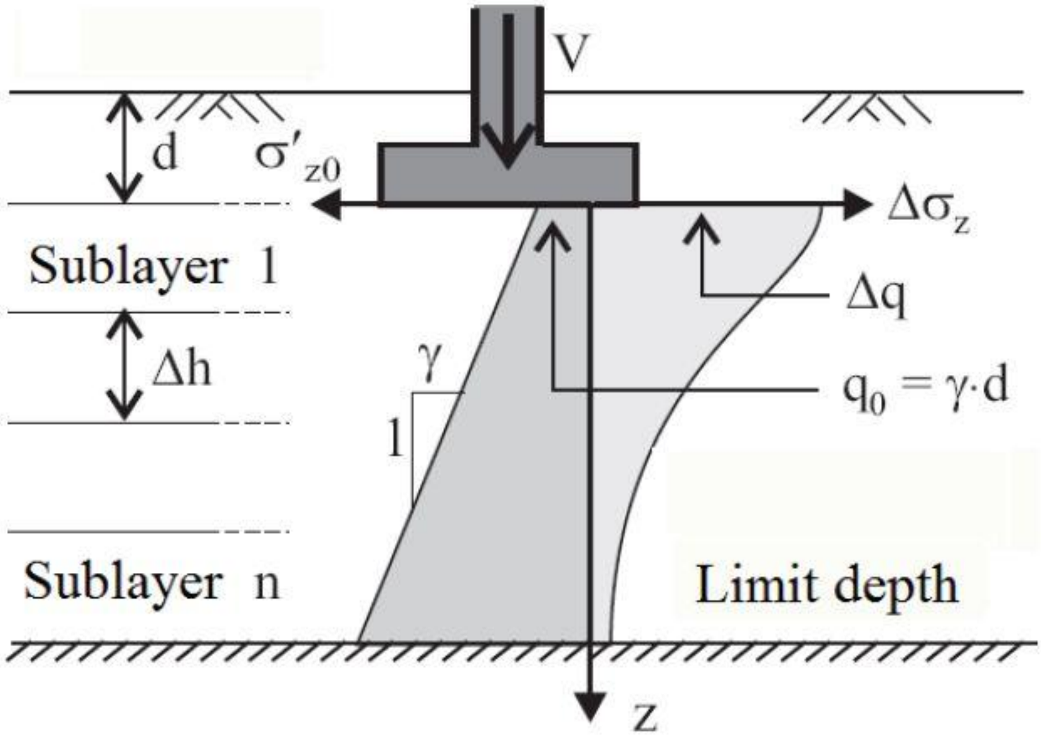


Fig. 6 Stress distribution in the elastic half-space down to the limit depth, modified after *(Hintner, 2008)*

The left hand sided grey area illustrates the stress which is initiated by the weight of the soil. Under the groundwater table has to be calculated with the submerged unit weight.

The stress is integrated by sub layers and together with the corresponding constrained modulus, the soil settlement are calculated.

In Fig 6 the symbols have following meanings:

Z = Soil depth

Δd = depth of the foundation

Δq = settlement generating load

In the Literature are given different symbols for the stress, a more detailed description of the loading behavior is in *(Kempfert & Raithel, 2012)*

$q = \sigma_0 = p$

q_0 = ($\sigma_0 = \gamma * d$)

$\Delta\sigma_z$= additional stress, stress from the structure load

In Germany is σ_z often expressed as $\sigma_{ü}$.

The additional stress results from the distribution into the depth which is illustrated in Fig.6 with the right grey function.

$$\Delta\sigma_z = i * \Delta q$$

i = characteristic point of a quadratic load area, visible in the tables of *Steinbrenner* (soft foundation) or *Kany* (stiff foundation) *(Kempfert & Raithel, 2012).* Due to that is it possible to calculate the settlement for stiff foundation with the *Steinbrenner* tables, but therefore is a correction needed.

Z = down to this depth σ_z forces settlement

$$\Delta\sigma_z = 0.2 * \sigma'_{Zo}$$

σ'_{z0}= additional stress at this depth

By 20% of the effective stress compared to the total stress are deformations negligibly small. In cohesive soft soils is to evaluate if it wouldn't be better to take a 10% boarder. This 20% criterion is used when no incompressible layer is delimiting the stresses in the subsurface.

The settlement can calculated by using closed or open formulas.

The open formula is defined as follows:

$$s = \frac{\sigma_z * d}{E_m}$$

s = settlement for this layer

d = thickness of layer

E_m = compression modulus ($E_m \approx E_s$ according to *DIN 4019*)

σ_z = additional stress

The closed formula is defined as follows:

$$s = \frac{\sigma_0 * b}{E_m} * f$$

σ_0 = average load distribution

b = smallest side length of the foundation

f = Settlement value according to *(Kany)*

The settlement is calculated for every layer and afterwards the different layers are added up. The different subdivisions build up the total settlement.

5 Soil

Soil particles are irregularly shaped solids that are in contact with adjacent soil particles. The weight and volume of a soil sample depends on the specific gravity of the soil particles, the size of the space between soil particles and the amount of the void space filled with water which is known as pore water (*Hillel, 2004)*.

Soil is usually a three-phase material: solid, water and gas

The proportion of the various phases, in terms of both weight and volume, can be represented schematically with the aid of the block diagram shown in Fig. 7.

Fig. 7 Description of the 3 Phase model soil. ‘S’ stands for solid, ‘w’ for water, ‘a’ for air and ‘v’ for voids. No subscripts are used relative to weight W and volume V of the entire soil mass after *(Schmidt, 2001)*

Relating to the block diagram there are a number of weight-volume or phase relationships that are useful in geotechnical engineering. These are essential values or parameters used in laboratory testing, shown in Fig.8.

• Kornwichte:	$\gamma_s = \frac{Trockengewicht}{Kornvolumen}$	$= \frac{G_d}{V_k} = \frac{\gamma_d}{1-n} = \frac{\gamma}{(1-n)\cdot(1+w)} = \gamma_w + \frac{\gamma'}{1-n}$
• Trockenwichte:	$\gamma_d = \frac{Trockengewicht}{Gesamtvolumen}$	$= \frac{G_d}{V} = (1-n)\cdot\gamma_s = \frac{\gamma}{1+w} = \gamma_r - n\cdot\gamma_w$ $= \gamma_s \cdot \frac{\gamma_r-\gamma_w}{\gamma_s-\gamma_w} = \frac{S_r\cdot\gamma_w\cdot\gamma_s}{w\cdot\gamma_s+S_r\cdot\gamma_w}$
• Feuchtwichte: Sättigung $S_r < 1$	$\gamma = \frac{Gesamtgewicht}{Gesamtvolumen}$	$= \frac{G}{V} = (1+w)\cdot\gamma_d = (1+w)\cdot(1-n)\cdot\gamma_s$ $= (1+w)\cdot\frac{S_r\cdot\gamma_w\cdot\gamma_s}{w\cdot\gamma_s+S_r\cdot\gamma_w} = (1+w)\cdot(\gamma_r - n\cdot\gamma_w)$
• Sättigungswichte Sättigung $S_r = 1$	$\gamma_r = \gamma(S_r = 1)$	$= \frac{G_r}{V} = \gamma_d + n\cdot\gamma_w = (1-n)\cdot\gamma_s + n\cdot\gamma_w = \gamma' + \gamma_w$ $= \left(1-\frac{\gamma_w}{\gamma_s}\right)\cdot\gamma_d + \gamma_w = \frac{\gamma_s-\gamma_w}{1+w}\cdot\frac{\gamma}{\gamma_s} + \gamma_w$
• Auftriebswichte:	$\gamma' = (1-n)\cdot(\gamma_s-\gamma_w)$	
• Porenzahl: **void ratio**	$e = \frac{\text{Hohlraumvolumen}}{\text{Feststoffvolumen}}$	$= \frac{V_0}{V_k} = \frac{n}{1-n} = \frac{\gamma_s}{\gamma_d} - 1 = (1+w)\frac{\gamma_s}{\gamma} - 1$ $= \frac{w\cdot\gamma_s}{S_r\cdot\gamma_w} = \frac{\gamma_s-\gamma_r}{\gamma_r-\gamma_w}$
• Porenanteil:	$n = \frac{\text{Hohlraumvolumen}}{\text{Gesamtvolumen}}$	$= \frac{V_0}{V} = \frac{e}{1+e} = 1 - \frac{\gamma_d}{\gamma_s} = 1 - \frac{\gamma}{(1+w)\cdot\gamma_s}$ $= \frac{w\cdot\gamma_s}{w\cdot\gamma_s+S_r\cdot\gamma_w} = \frac{\gamma_s-\gamma_r}{\gamma_s-\gamma_w}$
• Wassergehalt: Teilsättigung	$w = \frac{\text{Wassergewicht}}{\text{Feststoffgewicht}}$	$= \frac{G_w}{G_d} = \frac{\gamma}{\gamma_d} - 1 = \frac{\gamma}{(1-n)\cdot\gamma_s} - 1$ $= \frac{(\gamma_s-\gamma)\cdot S_r\cdot\gamma_w}{(\gamma-S_r\cdot\gamma_w)\cdot\gamma_s} = S_r\cdot\left(\frac{\gamma_w}{\gamma_d}-\frac{\gamma_w}{\gamma_s}\right)$
• Maximaler Wassergehalt: gesättigt ! $S_r = 1$	$w_{max} = \frac{n}{1-n}\cdot\frac{\gamma_w}{\gamma_s} = n\cdot\frac{\gamma_w}{\gamma_d} = e\cdot\frac{\gamma_w}{\gamma_s} = \frac{\gamma_r}{\gamma_d} - 1 = \frac{(\gamma_s-\gamma_r)\cdot\gamma_w}{(\gamma_r-\gamma_w)\cdot\gamma_s} = \frac{\gamma_w}{\gamma_d} - \frac{\gamma_w}{\gamma_s}$	
• Sättigungszahl:	$S_r = \frac{w}{w_{max}} = \frac{\gamma-\gamma_d}{n\cdot\gamma_w} = \frac{(1+e)\cdot\gamma-\gamma_s}{e\cdot\gamma_w} = \frac{w\cdot\gamma\cdot\gamma_s}{\gamma_w[(1+w)\cdot\gamma_s-\gamma]} = \frac{w\cdot\gamma_d\cdot\gamma_s}{\gamma_w\cdot(\gamma_s-\gamma_d)}$	

Fig. 8 Most important formulas for soil, red marked is the relationship of the void ratio, modified after *Triantafyllidis, (2013)*

Of particular note is the void ratio e, which correlates in general with the relative strength and compressibility of a soil sample, when the soil type is well known.

Lower void ratios generally indicate stronger, less compressible soils, and high void ratios may indicate weaker, more compressible soils which is susceptible for a settlement.

When stress is applied to a soil sample, the deformation that occurs depends on the forces between the soil particles that are in contact with each other (intergranular forces) in the void space and the water content. Dry soils with no pore water in the void space will deform due to a combination of sliding between the soil particles and deformation or crushing of the particles themselves *(Studer, 2007)*. Intergranular sliding accounts for most of the deformations that occur as the particles move to increase the contact area between the particles to support the increase in applied weight. Fully saturated soils under additional stress show a time dependent settlement behavior, especially for fine grained soil.

5.1 Settlement

Settlement is a vertical movement underneath a foundation element. This can occur through following points, as described in *Dachroth (1992)*:

- Compaction of the ground under a static load
- Compaction of the ground through a dynamic force
- Horizontal movement of the ground at the edge of the foundation
- Shrinking of fine grained soil
- Soften of fine grained soil
- Lowering of groundwater level
- Frost of the ground
- Collapse of underground cavities
- Elastic and plastic deformation of the ground above new founded cavities (Mining, Tunneling)

The compaction of the ground under a static load is a plastic and a partially elastic deformation which depends on:

- Maximum Load
- Size of the jacking point
- Form of the foundation
- Constrained modulus
- Thickness of the compressible layer

Differential settlement can occur when one load-bearing member of a structure experiences total settlement of a different magnitude than an adjacent load-bearing member. Because differential settlement introduces load and stress in the structure above the foundations in general, limiting differential settlement may frequently be of more interest in the design of a structure than total settlements. If total settlements are limited, of course, differential settlements will be limited to an even greater extent. Infrastructural structures, especially bridges, are not exceptionally tolerant of differential settlement. Deformation limitations will therefore frequently form the upper limit of permitted soil bearing capacities used to design shallow foundations *(Coduto, 1994).*

The two primary considerations that affect the selection and design of shallow foundations are the bearing capacity and the settlement potential of the ground within the zone of influence below the foundation. The total settlement of a shallow foundation results from a combination of the following (*Huder et al, 2011)*:

- Immediate Compression
  This settlement may result from compression of the material supporting the foundation or from reduction in the pore space in non-saturated soils, due to expulsion of air from the void space. In cohesionless soils, nearly all the settlement that results from an increase in stress is associated with immediate or compression.

- Primary Consolidation
  Consolidation settlement occurs when saturated, fine-grained soils experience an increase in stress. The water in the pore spaces initially carries the load. Then, as the water is expelled from the pore space, the soil experiences a reduction in volume and a decrease in the pore space between soil particles. The process can be slow to rapid and is a function of the permeability of the soil in the direction of drainage *(Verruiyt, 2010)*. The process slows and eventually stops once all the excess pore water pressure, induced by the stress increase, is dissipated. Primary consolidation is principally of concern for fine-grained, or cohesive soils, since coarser grained soils are typically permeable enough that any volume change, which occurs as a result of expulsion of water from void space occurs as rapidly as the loads are applied.

- Secondary Compression
  Some soils, after first experiencing primary consolidation settlement, continue to strain after excess pore-water pressures are dissipated. This process is termed secondary compression or “creep”. Organic soils and some inorganic fine grained soils are soil types that can exhibit both primary consolidation and secondary compression, or “creep”, settlement *(Souyama, 2005).*

5.2 Permeability

Water is essentially incompressible. The water contained in the pore space of soils which are saturated (pore spaces completely filled with water) is also a function of the ability for water to be expelled from the pore space. The permeability of soil is a measure of the speed with which water can pass through the pore space. The lower the permeability of a soil sample, the longer it takes for water to be expelled from the pore space.

In geotechnical engineering the permeability k quantifies the permeability of soil and rock for liquids or gases. For example ground water, oil or natural gas. The permeability is closely connected to the hydraulic conductivity coefficient as described in *Verruiyt (2010)*.

The permeability of soils depends primarily on their porosity, while the permeability of rock depends on the porosity and fissuring. The porosity of soils in turn depends on the particle size, its distribution and thus the pore volume of the soil (*Hillel, 2004)* as well visible in Tab.2.

Soiltype		clay		silt			sand			gravel	
			fine	middle	coarse	fine	middle	coarse	fine	middle	coarse
Grainsize	from	< 0.002	0.002	0.005	0.02	0.05	0.2	0.5	2	6	20
in mm	to		0.005	0.02	0.05	0.2	0.5	2	6	20	60
Hydraulic	cm/s	10^(-6)	10^(-5)	10^(-4)	10^(-3)	10^(-2)	10^(-1)	1	>1	>1	>1
		to 10^(-4)									
coefficent k	m/s	10^(-10)	10^(-7)	10^(-6)	10^(-5)	10^(-4)	10^(-3)	10^(-2)	10^(-1)		
		to 10^(-8)									

Tab. 2 Soil type with the permeability parameters, modified after *Floss, (2006)*

Permeability and hydraulic conductivity can be quantified in a similar manner. The flow rate Q through a permeable medium is a function of the pressure difference Δp, can be expressed in these different units:

- Transmission , area (m²)
- The coefficient of permeability, velocity (m/s).

In addition both variables are constant to the flow rate Q, provided the following conditions are met *(Verruiyt, 2010):*

- laminar flow

- no interaction between rock surface and flowing medium
- only one phase in the pore space in saturated conditions

5.3 Consolidation states

“If a load is applied quickly to a soil sample with low permeability, the pore water will initially carry the load until the water drains from the pore space and the soil particles begin to slide and accept the load. While the load is still carried by the pore water, the water will experience increased or excess pore water pressure. As the excess pore water pressure dissipates, the soil sample will deform into a smaller volume and a denser configuration. This process is termed consolidation.” *(Kempfert & Raithel, 2012)*

The mechanical compaction takes place essentially by compression of the pores. If the pores are filled with water, a compaction can be achieved only by an extraction of pore water due to the incompressibility of water.
If the water movement is hindered due to low permeability and long drainage paths, the load increase is first only by the pore water and therefore the pressure rises. Because the drainage of porewater pressure is gradually transferred to the soil skeleton, it condenses so far that it can absorb the load increase directly *(Studer, 2007).*

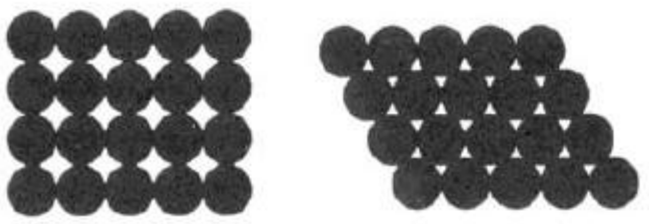


Fig. 9 Scheme for lose packing and dense packing, (*Kolymbas, 1998)*

The reverse process applies to load reduction. When the pore water gets under reduced pressure, the surrounding water is sucked into the sample. As a result, the negative pressure gradually decreases; the grain structure is relieved and loosens. This process is also referred to as negative consolidation *(Verruiyt, 2010).*
The duration of the pore water pressure equalization is theoretically infinite, since its rate asymptotically approaches zero. Practically the consolidation period is completed after just 98 % of the pore water pressure is decreased. The consolidation coefficient

result from the square of drainage length based on the consolidation period and is a constant. According to these definitions *(Terzaghi et al., 1996)*, the height of the pore water pressure has no effect on the consolidation period.

Only in fine-grained soils a remarkable flow obstruction of the pore water occurs. The notion of consolidation is applicable only to cohesive soils *(Kolymbas, 2007)*:

- In case of the under consolidated (UC) case the granular structure has not compacted enough to accommodate the increase of the required load. It still needs more time to drain more pore water out of the system.
- In case of the normal consolidated (NC) case, the granular structure is just condensed enough to accommodate the increase of the required load.
- In case of the over consolidated (OC) case, the granular structure is denser by a formerly acting compaction than necessary for the actually required load. In the latter two cases, in NC and OC no water movement occurs. In the transition from OC to NC a negative consolidation takes place.

In normal and consolidated soils the shear strength is proportional to the grain distribution and density, which means the friction angle is constant and the bottom cohesion is lower, shown in Tab.3.
“Consolidated soils with the same soil composition have initially a higher shear strength, which is based on the tendency for loosening tight grain stands in shear dilatancy”. *(Verruiyt, 2010)*
Still present pore water delays this process, due to the loosening associated with the increase in pore volume under reduced pressure, which stabilizes the granular structure with respect to shear and tensile stresses.
With increasing shear strength the soil tends to loosen up and loosens the OC- shear strength back to the value at NC (*Hillel, 2004).*

In a normally consolidated ground the maximum stress is equal to the current stress. An over consolidated soil has experienced stresses greater than momentary ones, such as the ballast of ice shields.
The Over Consolidation Ratio (OCR) represents the extent of the consolidation. It is the ratio between the maximum vertical stress in the past and the current vertical stress.
Often the maximum stress of the past is unknown; there are laboratory tests like the Casa Grande Trial, which takes the normal and the compression behavior comparing it to the consolidated soils.

The over consolidation has great influence on the shear strength of cohesive soils and on the constrained modulus. Due to the bias non-cohesive soils receive cohesion in addition to the frictional force. In nature, caused by the over consolidation a hard crust often forms, in which the shear strength is greater than in normal consolidated soil beneath the crust.

Consolidated soils are also stiffer which has far-reaching consequences for the earth pressure and the design of structures like *Verruiyt, (2010)* describes it.

Interface Materials	Coefficient of Friction, tan δ	Friction Angle, δ (degrees)
Mass concrete on the following materials:		
Clean sound rock	0.70	35
Clean gravel, gravel sand mixtures, coarse sand	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45	19 to 24
Fine sandy silt, nonplastic silt	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay	0.40 to 0.50	22 to 26
Medium stiff and stiff clay and silty clay (masonry on foundation materials has same friction factor)	0.30 to 0.35	17 to 19

Tab. 3 Overview over the ultimate friction factors for dissimilar materials according to *NAVFAC, (1986)*

In Tab. 3 it is shown that preconsolidated clay has a coefficient of friction from 0.40 to 0.50 and a friction angle from 22 to 26. Therefore unconsolidated clay has a coefficient of friction from 0.30 to 0.35 and a friction angle 17 to 19.

This shows the magnitude of impact on the soils state and abilities, which is a major point of how a soil behaves when it’s stressed. That’s why the soils consolidation state is important in a settlement analysis.

5.4 Constrained modulus

The constrained modulus is a characteristic parameter for calculating the settlement of a soil. It is determined by compression tests, like the Oedometer trial, with restricted lateral expansion and has a comparable meaning as the elastic modulus for solid-state materials. The constrained modulus depends on the load range; it is used among others in the constrained modulus method and also for illustration of the soil in a ground model and for the design of foundations.

- In Switzerland it is called "Zusammendrückungsmodul M_e" and is determined by results of the Oedometertrail or the Plateload test
- In Germany it is called "Verformungsmodul E_v" from the Plate load test and "Steifemodul E_s" from the Oedometertrail

  (Huder et al, 2011)

		1	2	3
		Elastizitätsmodul E	**Steifemodul E_s**	**Verformungsmodul E_v**
1	E		$E = \frac{1-\nu-2\nu^2}{1-\nu} \cdot E_S$	$E = (1-\nu^2) \cdot E_V$
2	E_S	$E_S = \frac{1-\nu}{1-\nu-2\nu^2} \cdot E$		$E_S = \frac{(1-\nu) \cdot (1-\nu^2)}{1-\nu-2\nu^2} \cdot E_V$
3	E_v	$E_V = \frac{1}{1-\nu^2} \cdot E$	$E_V = \frac{1-\nu-2\nu^2}{(1-\nu) \cdot (1-\nu^2)} \cdot E_S$	

Fig. 10 Relationships between the different modulus's for a half isotropic medium with the Poissonratio $0 \leq \nu \leq 0.5$ *(Prinz & Strauß, 2010)*

To determine the constrained modulus, the pressure and the settlement curve of a soil must be known. The constrained modulus is the slope of the tangent to the pressure and the settlement curve and therefore the ratio of the pressure difference and from the settlement differents between two load levels. The pressurization curve obtained in the Oedometer test represents the measured deformations as a function of the associated effective stress.

The constrained modulus depends on the stress range; therefore, the pressure-reduction curve is not linear. That means an Oedometer test must be performed in a plurality of load stages. Between the load levels, the pressurization line is assumed to be linear. The compressive stresses increase with depth due to the dead weight of the soil. *(Studer, 2007)*

The constrained modulus E_s expressed as the secant modulus.

$$E_s = \frac{\Delta\sigma'}{\Delta\varepsilon}$$

E_s is depending on the load, ε is not absolute and depends on the reference height h_0

$$\varepsilon = \frac{s}{h_0}$$

.

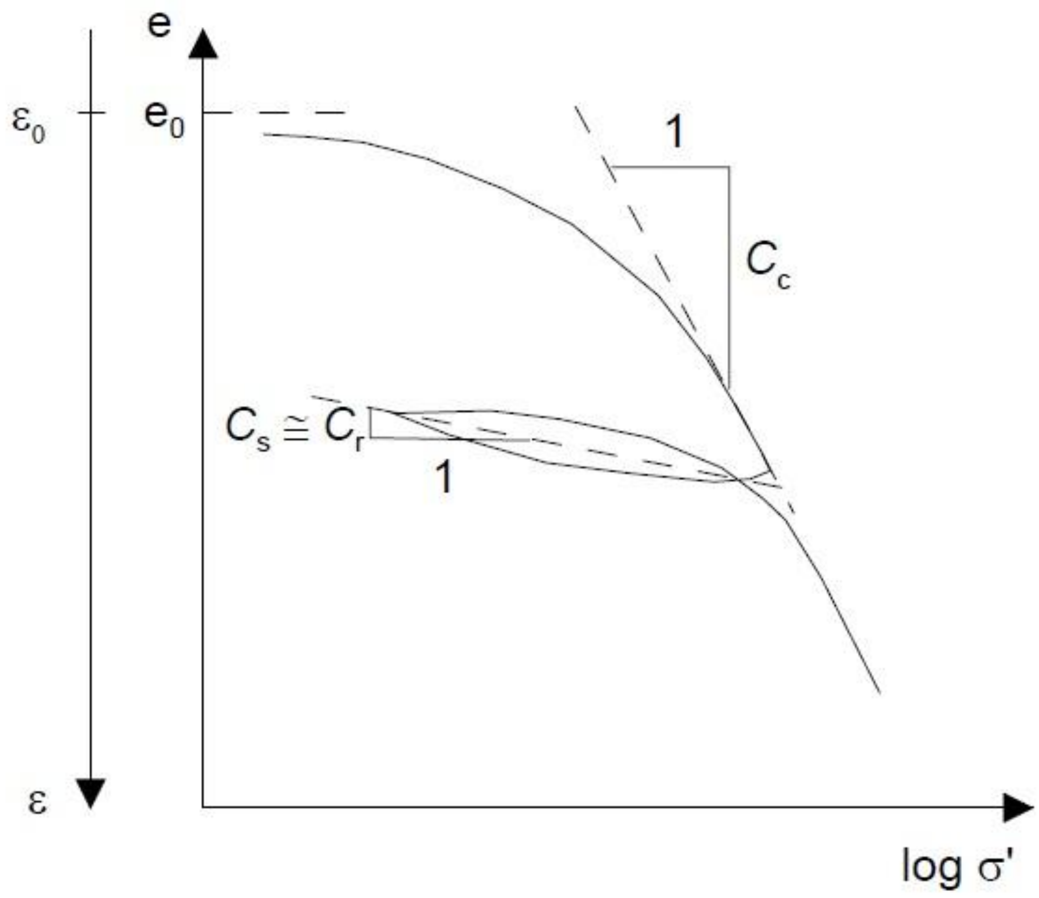


Fig. 11 Estimation of deformation indexes the compression index C_c, the swell index C_s and the recompression index C_r in the ($\log \sigma' - e$) diagram, modified after *Souyama, (2005)*

The C_c value defines the inclination of the linear area of the compression line in the pressure -void ratio –diagram.

$$C_c = -\frac{\Delta e}{\Delta \log \sigma'}$$

The C_r value and the C_s value are mostly shown by the inclination of the pressure reduction line, visible in Fig.11.

For normal consolidated fine grained soils can C_c be estimated after *Terzaghi & Peck (1967)*

$$C_c = 0.009 * (w_L - 10)$$

The compression index depends on the water content.

Compressibility of soil	Compression index C_c
low	$C_c < 0.2$
middle	$C_c = 0.2$ to 0.4
high	$C_c > 0.4$

Tab. 4 Experience values of the Compression index C_c after *Mitchell (1993)*

The stress dependent constrained modulus for primary loading and reloading can be calculated with the following equations.

Primary loading

$$E_{s,p} = \frac{\sigma' * (1 + e)}{C_c}$$

Reloading

$$E_{s,r} = \frac{\sigma' * (1 + e)}{C_r}$$

For simplification is the constrained modulus often expressed as the average constrained modulus, or there are given two values in a geotechnical report, the primary loading constrained modulus and the reloading constrained modulus. The first time loading the constrained modulus it is going to be low, because the soil is first partially irreversibly compressed during its consolidation period (Primary loading). Therefore the storage density decreases. If the soil is released and then loaded again, the constrained modulus is much greater (reloading).

6 In-Situ Tests for determining the constrained modulus

One of the main advantages of in-situ tests is the ability to assess the underground conditions in its natural environment, with the goals to:

- estimate the geotechnical parameters
- determine sub-surface stratigraphy and identify the soil
- provide results for direct geotechnical design

This chapter gives a short overview of the common in-situ methods, which are used at Max Bögl for evaluating the constrained modulus out of field data.

6.1 Dynamic Probing

The Dynamic Probing Test is described in *DIN 4094*.

The penetrometer consists basically of three elements: the bar, the anvil and the drop weight. The bar is driven into the ground with constant impact energy. The required blow rate which is needed for 10cm is named N_{10}. The impact rate N_{10} is a measure of the compactness of non cohesive or cohesive soils.

The differences between the different DP devices is listed in Tab. 5. The weight and the drop height are the influence factors for the penetration depth. The possible depth which can be investigated with the Dynamic Probing, is 8 meters for the DPL, 20-25 meters for the DPM, up to 25 meters for the DPH and the DPSH is also used for depth deeper than 25 meters.

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

The result of the dynamic probing can only be interpreted, if you know in which kind of soil you are. The dynamic probing should only be used as a supplementary examination to a test pit or a borehole. It is not an independent investigation tool.

DP device	Symbol	Units	DPL	DPM	DPH	DPSH	
						DPSH-A	DPSH-B
Drive block weight	m	kg	10 ± 0,1	30 ±0,3	50 ± 0,5	63,5 ± 20	63,5±0,5
Drop height	h	mm	500±10	50 ± 10	500 ± 10	500 ± 100	750 ± 20
Anvil							
Diameter	d	mm	50<d<0,5Dh	50<d<0,5Dh	50<d<0,5Dh	50<d<0,5Dh	50<d<0,5Dh
Weight (max)	m	kg	6	18	18	18	30
90° Cone							
Nominal area of base	A	cm^2	10	15	15	16	20
Diameter of base, new	D	mm	35,7±0,3	43,7±0,3	43,7±0,3	45,0±0,3	50,5±0,5
Diameter of base (min)		mm	34	42	42	43	49
Surface length (mm)	L	mm	35,7 ± 1	43,7±1	43,7±1	45,0±0,3	51 ±2
Cone tip length		mm	17,9±0,1	21,9±0,1	21,9±0,1	22,5±0,1	25,3 ±0,4
Max permitted tip wear		mm	3	4	4	5	5
Driving rods							
Mass (max)	m	kg/mm	3	6	6	6	8
OD diameter (max)	dr	mm	22	32	32	32	35
Rod deviation							
Lowest 5 m		%	0,1	0,1	0,1	0,1	0,1
Rest		%	0,2	0,2	0,2	0,2	0,2
Specific work per stroke	mgh/a EN	kJ/m^2	50	100	167	194	238

Tab. 5 Table shows the technical parameter of the Dynamic Probing testing devices, modified after *EN ISO 22476*

Dynamic Probing is mainly used in cohesionless soils. In soft cohesive and organic soils, the sleeve friction can have substantial effects on the penetration resistance. The penetration resistance increases with the depth at the same consistency of the material.

Close to boarderlines of a given soil layer the penetration resistance will be influenced by the type of soil below and above. The compressibility and inclination of the layer below the penetration location will be influenced.*(Hashmat, 2000)*

More technichal details are described in *DIN 4094*, all correlations of geotechnical parameters and results are conservatively estimated correlations, which means that these values have to be interpreated carefully.

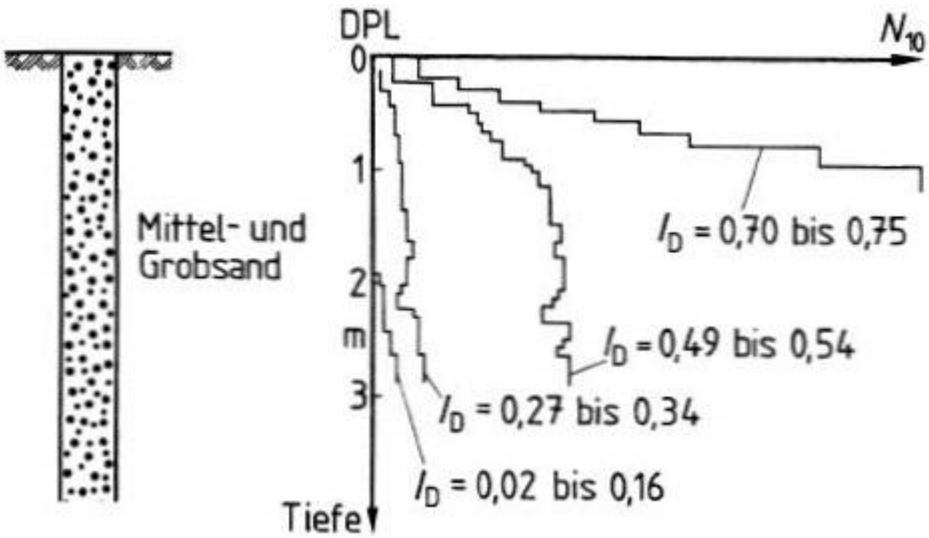


Fig. 12 Example of a DPL probing with different compaction indexes I_D in middle and coarse grained sand after DIN 4049

Fig.12 shows a profile of a borehole which has the strata of middle and coarse grained sand. With the knowledge that the DPL was done in sand, it is possible to evaluate the density of the sand.

Compactness	DPL N_{10}	DPM N_{10}	DPH N_{10}
very loose	0-6	0-4	0-1
loose	6-10	4-11	1-4
middle dense	10-50	11-26	4-13
dense	50-64	26-44	13-24
very dense	>64	>44	>24
Consistency			
very soft	0-3	0-3	0-2
soft	3-10	3-8	2-5 (4)
stiff	10-17	8-14	(4) 5-9 (8)
very stiff	17-37	14-28	(8) 9-17
hard	>37	>28	>17

Tab. 6 Density values according to the blow counts N_{10}, modified after *Prinz & Strauß, (2010)*

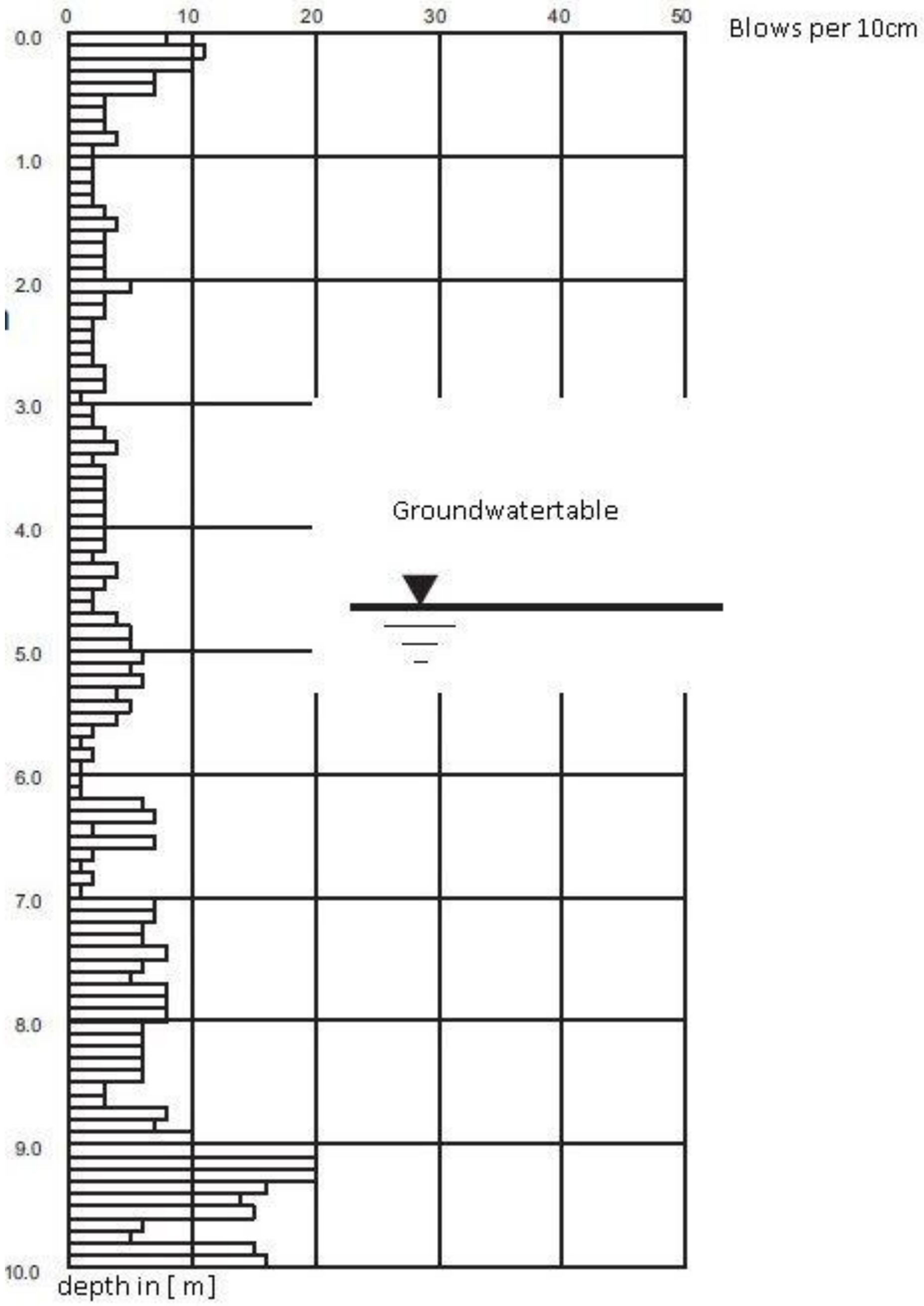


Fig. 13 DPH profile example, with the blow number on the x-axis and on the y-axis the depth

Dynamic Probing is not anindependent investigation tool as described before. So in general without the information about the stratigraphy and the soil it is not possible to reach satisfying conclusions, if the higher resistance is a change of the strata or a change in the density of the same soil type. The *DIN 4094* provides as well a correlation for penetrating soil in the groundwater which is an important point due to the fact that the water reduces the blow counts.

6.2 Standard Penetration Test

The Standard Penetration Test is described in the *DIN 4094*. The STP is very common in the US and the UK. In Germany the test has some modification to the classical SPT and is named “Bohrlochrammsondierung” with the abbreviation BDP. But generally BDP is also called SPT in Germany and differs in spite of the mentioned technical parameters. The classical SPT is a probing on the bottom of a borehole where the penetrometer takes a sample 45cm under the borehole surface. The number of blows for the last 30cm is named N_{30} and is the result value. This is shown in Fig. 14 the classical (international) SPT gets rammed of the top. Compared to that has the BDP a closed tip and the hammer is situated in a waterproof casing directly above the tip in the borehole.

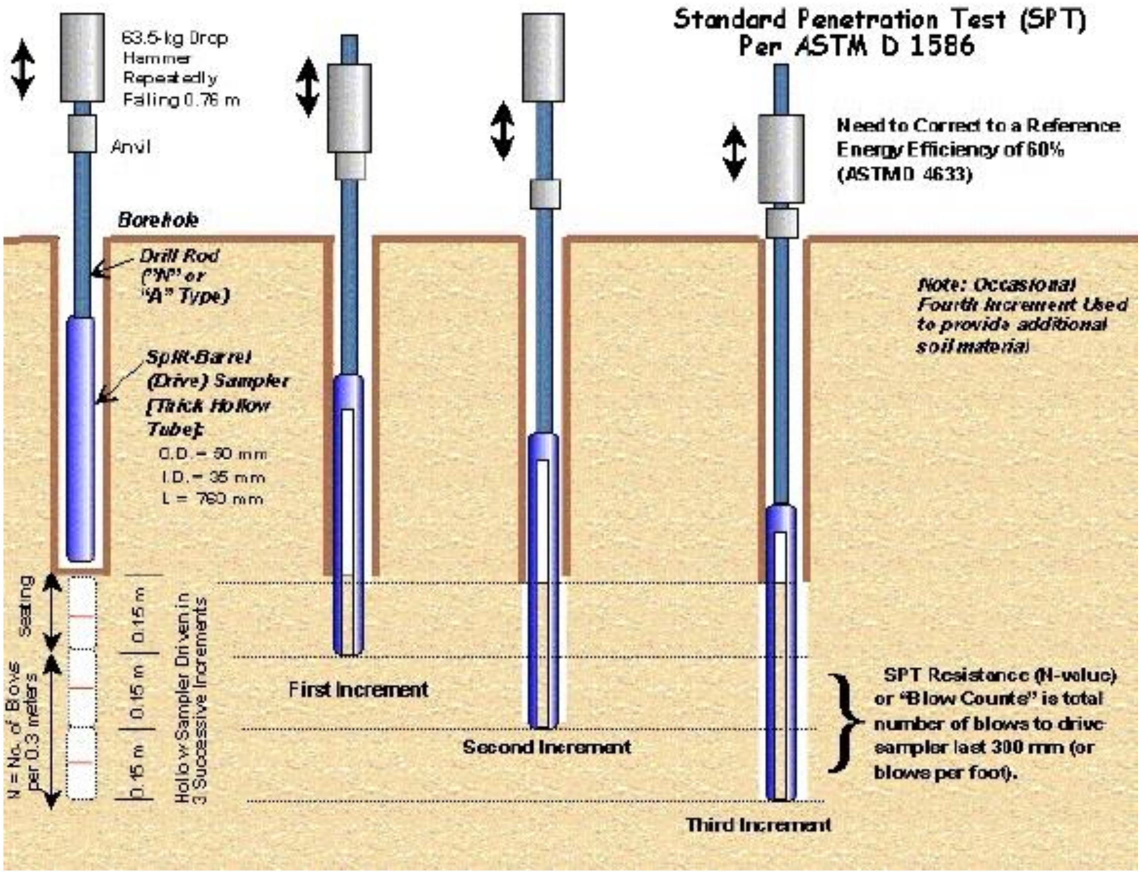


Fig. 14 SPT procedure after ASTM after *Zhang, (2001)*

The advantages of the SPT Test:

- quick

- relatively cheap
- widely available
- independent results on different depths
- no extra equipment needed

Disadvantages of the SPT Test:

- many error sources (which are clearly reduced for the BDP)
- experienced drilling crew needed

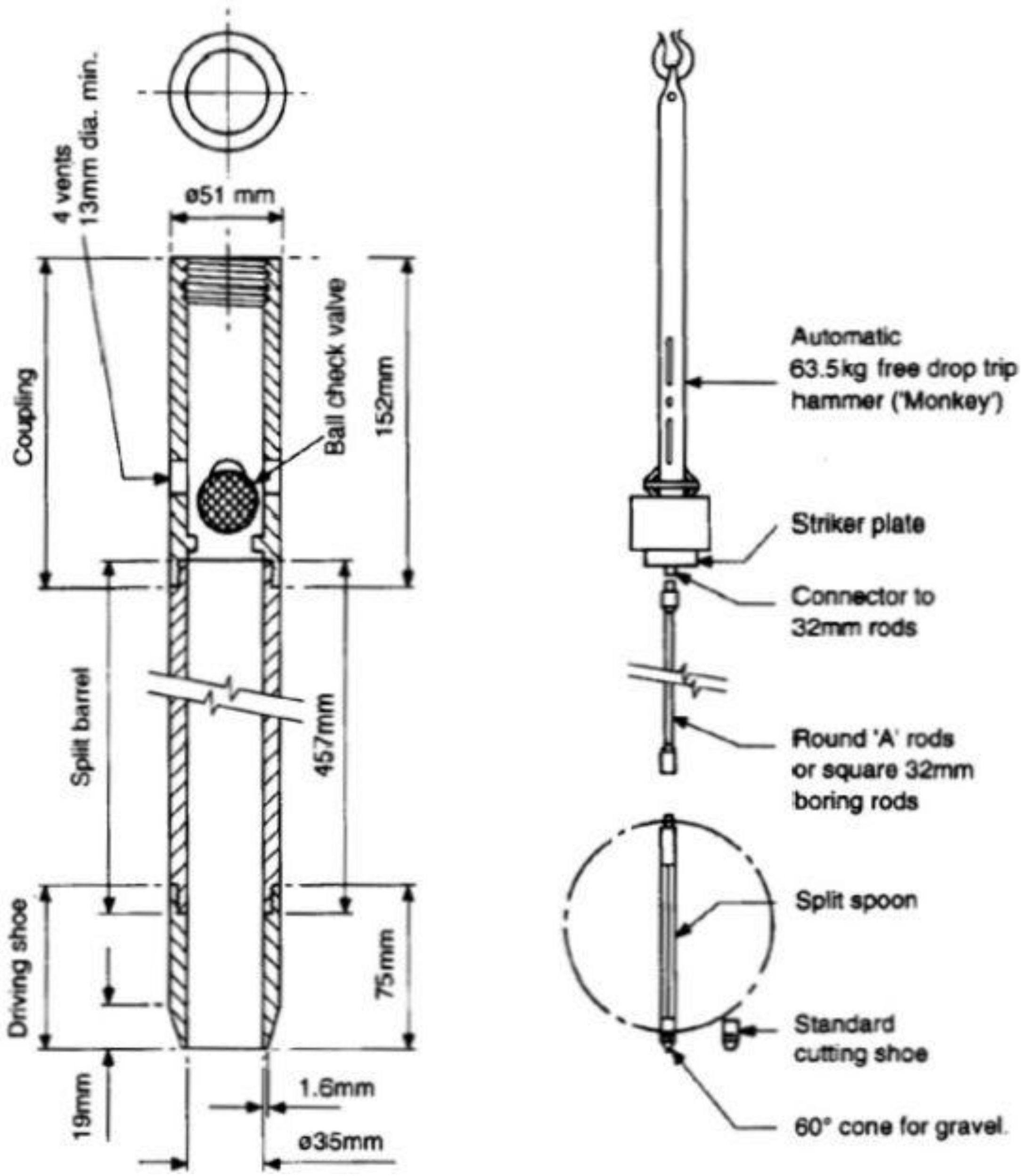


Fig. 15 Classical SPT device named according to *Clayton et al., (1995)*

The classical (international) SPT consists of driving a standard 50mm outside diameter thick, walled sampler into the soil at the bottom of the borehole, using repeated blows of a 63.5 kg hammer falling through 760mm. A more accurate description of SPT and BDP is shown in the *DIN 4094* or in the *ASTM*.

Relative Density	SPT N_{meas} (blows/300 mm or blows/ft)
Very loose	0–4
Loose	5–10
Medium Dense	11–30
Dense	31–50
Very Dense	>51

Tab. 7 Cohesionless soil density description based on SPT values *(Terzaghi et al., 1996)*

In Tab.7 the basic SPT values for the relative density of sand are shown and in Tab.8 the consistency of fine grained soils in relation to SPT results is illustrated.

Consistency	SPT N_{meas} (blows/300 mm or blows/ft)
Very Soft	0–1
Soft	2–4
Medium Stiff	5–8
Stiff	9–15
Very Stiff	16–30
Hard	31–60
Very Hard	>61

Tab. 8 Fine grained soil consistency description based on SPT values *(Terzaghi et al.,1996)*

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

Tab. 9 SPT-values with the matching constrained modulus recommended from the German institution for hydraulics

In the lecture notes of Professor Vogel of the Technical University of Munich is a table which directly indicates a constrained modulus value to the blow counts. This table can give an overview, but for a detailed analysis the ranges are too wide and it is not clear if it is meant the classical SPT or the BDP. *Liao & Whitman, (1986)* published the following equation which is now used in many regions of the world to correct the official (international) SPT data. In Tab.10 the different correction factor values are shown. The $N_{1,60}$ value is the value of the classical SPT test corrected to 60% of the theoretical free-fall hammer energy and the overburden pressure.

$$N_1 = C_n * N$$

$$N_{1,60} = N * C_N * C_E * C_B * C_R * C_S$$

Factor	Term	Equipment Variable	Correction
Overburden Pressure	C_N	-	$(P_a/\sigma'_v)^{0.5}$ $C_N \leq 2$
Energy Ratio	C_E	Safety Hammer Donut Hammer	0.60-1.17 0.45-1.00
Borehole Diameter	C_B	65-115 mm 150 mm 200 mm	1.00 1.05 1.15
Rod Length	C_R	3-4 m 4-6 m 6-10 m 10-30 m > 30 m	0.75 0.85 0.95 1.0 < 1.0
Sampling Method	C_S	Standard Sampler Sampler without liners	1.0 1.15-1.30

Tab. 10 Summary of the correction factors for the official SPT measurements for formula, after *NCEER Workshop, (1997)*

There are several more correction equations for the SPT test, which are published in *Schnaid, (2009).*

6.3 Cone Penetration Test

The Cone Penetration Test is described in the *DIN 4094.*

A cone penetrometer is pushed into the soil by a constant static force at a constant rate (20 mm/s). The total resistance and the tip resistance are measured separately. The tip of the bar has a diameter of 3.56 cm and a peak cross section of 10 cm².

According to *DIN 4094* the values for the peak pressures, bulk density, angle of friction and the constrained modulus can be determined.

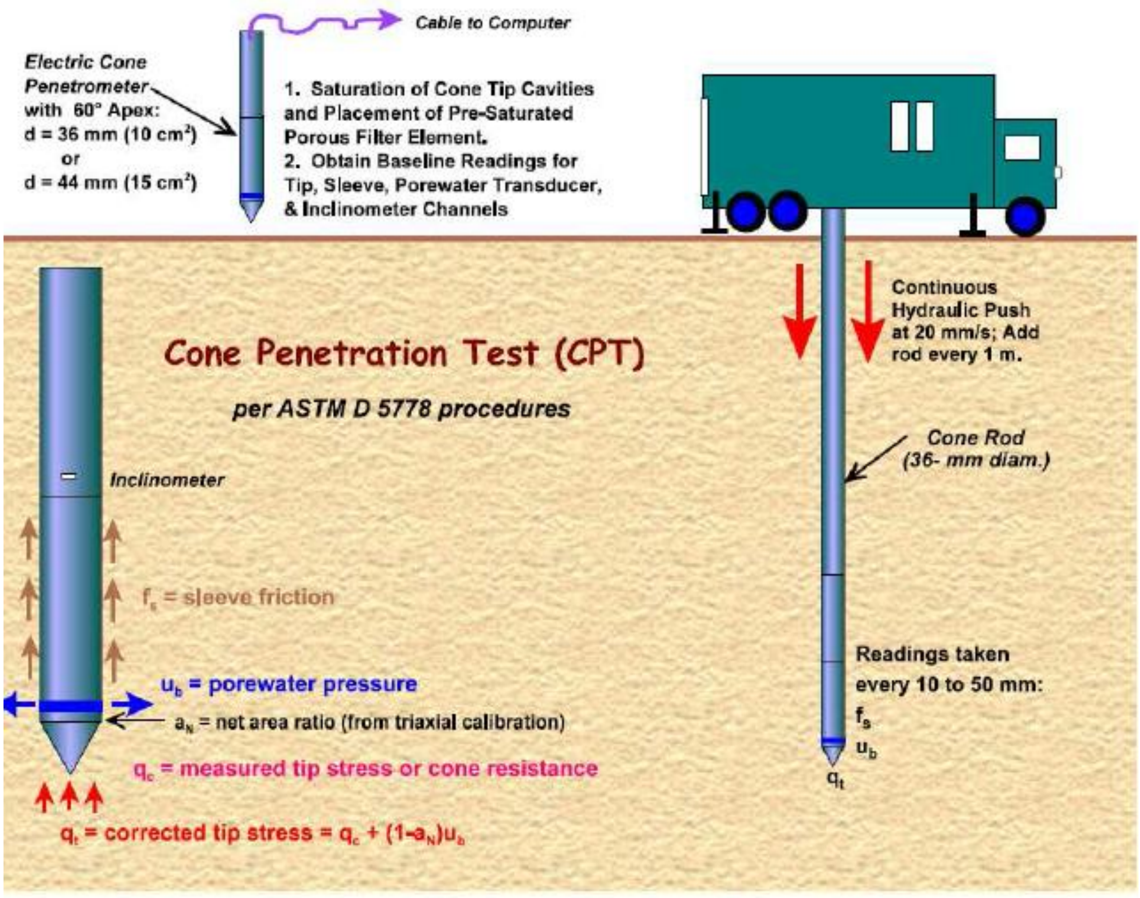


Fig. 16 Components and procedure of the CPT after *Mayne et al, (2001)*

There are two major modifications on the standard CPT: the CPTu, which measures the pore water pressure and the SCPTu, which is collects seismic data.

CPT disadvantages

- Does not give a sample
- Will not work in soil with gravel
- Need to mobilize a special rig

Most 10 cm² commercial penetrometers have an area ratio between 0.75 and 0.82, many 15 cm² cones show a range of 0.65 to 0.8 and yet several older models indicate values as low as a ~0.35. The value of the area ratio should be provided by the manufacturer *(Sachsenhofer, 2012).*

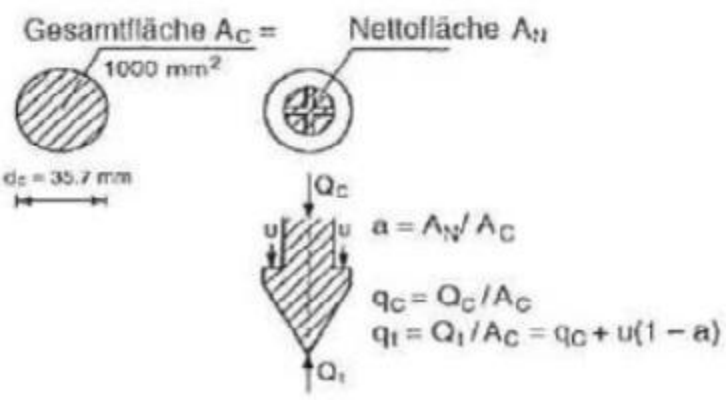


Fig. 17 Definition of the area ratio of a CPT device *(Witt, 2008)*

Corrected tip resistance

$$q_t = q_c + (1 - a)u$$

q_t = corrected tip resistance
q_c = cone tip resistance
u = pore water pressure measured behind the cone
a = area ratio, visible in Fig.17

Normalized cone resistance

Wroth, (1984) published this equation for normalizing the cone resistance.

$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma_{vo}'}$$

Robertson, (2009) developed a correction equation where the atmospheric pressure is a correction value as well.

$$Q_{tn} = \frac{q_t - \sigma_{vo}}{\sigma_{atm}} * \left(\frac{\sigma_{atm}}{\sigma_{vo}'}\right)^n$$

$Q_{t\,(n)}$ = normalized cone resistance

σ_{vo}' = vertical total stress

σ_{vo} = vertical effective stress

σ_{atm} = atmospheric pressure (0.1 MPa)

n = stress exponent that varies with the soil type behavior index

$$n = 0.381(\, I_c) + \; 0.05\,\left(\frac{\sigma_{vo}'}{p_a}\right) - 0.15$$

I_c = Soil behavior type index

p_a = reference stress

The Friction ratio R_f can be defined as:

With q_t, the corrected tip cone resistance:

$$R_f = \frac{f_s}{q_t}$$

With q_c, the tip cone resistance:

$$R_f = \frac{f_s}{q_c}$$

Normalized friction ratio

$$F = 100 * \frac{f_s}{(q_t - \sigma_{vo})}$$

CPTU correction

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{vo}'}$$

u_2 = pore water behind the cone

u_0 = in-situ (equilibrium pore water pressure)

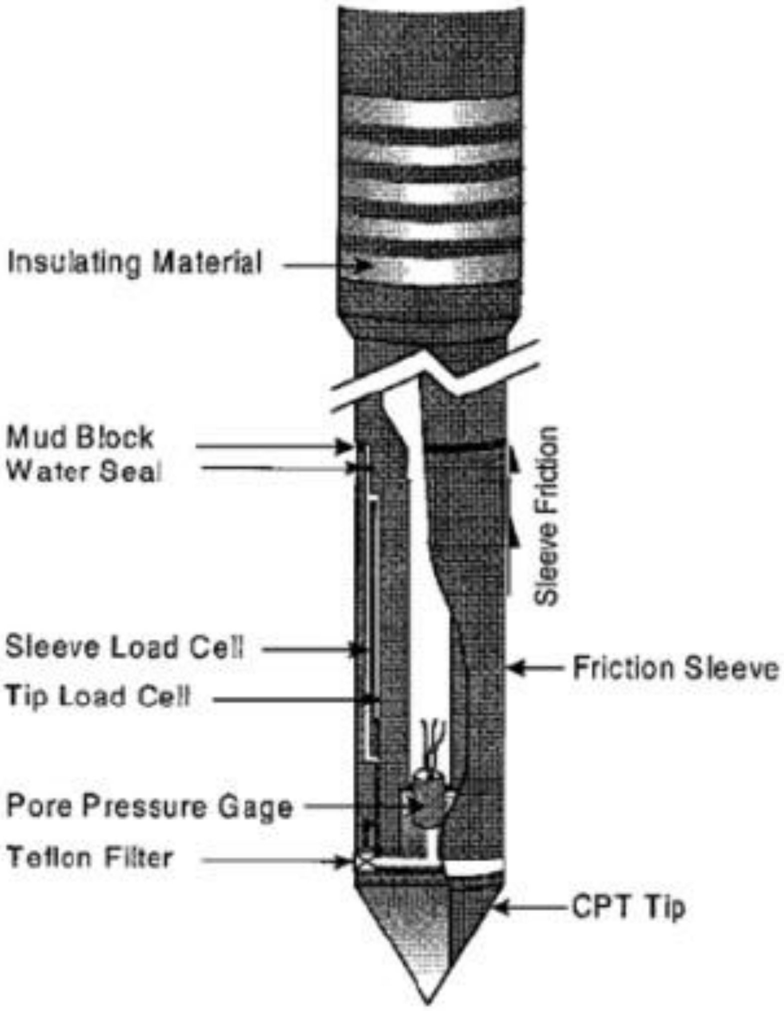


Fig. 18 Schematic detail view of a cone penetrometer device, after *Ozan, (2003)*

Fig.18 shows the detailed image of a CPT device with the two load measurement cells. The tip load cell is signal output for the cone tip resistance q_c and the sleeve load cell for the sleeve friction f_s.

Following points influence the measured results:

- grain size, grain geometry, grain roughness
- compressibility, unit weight, shear friction
- cementation
- OCR (over consolidation ratio)
- layer thickness
- calibration errors

- pore water pressure
- inclination of the CPT device

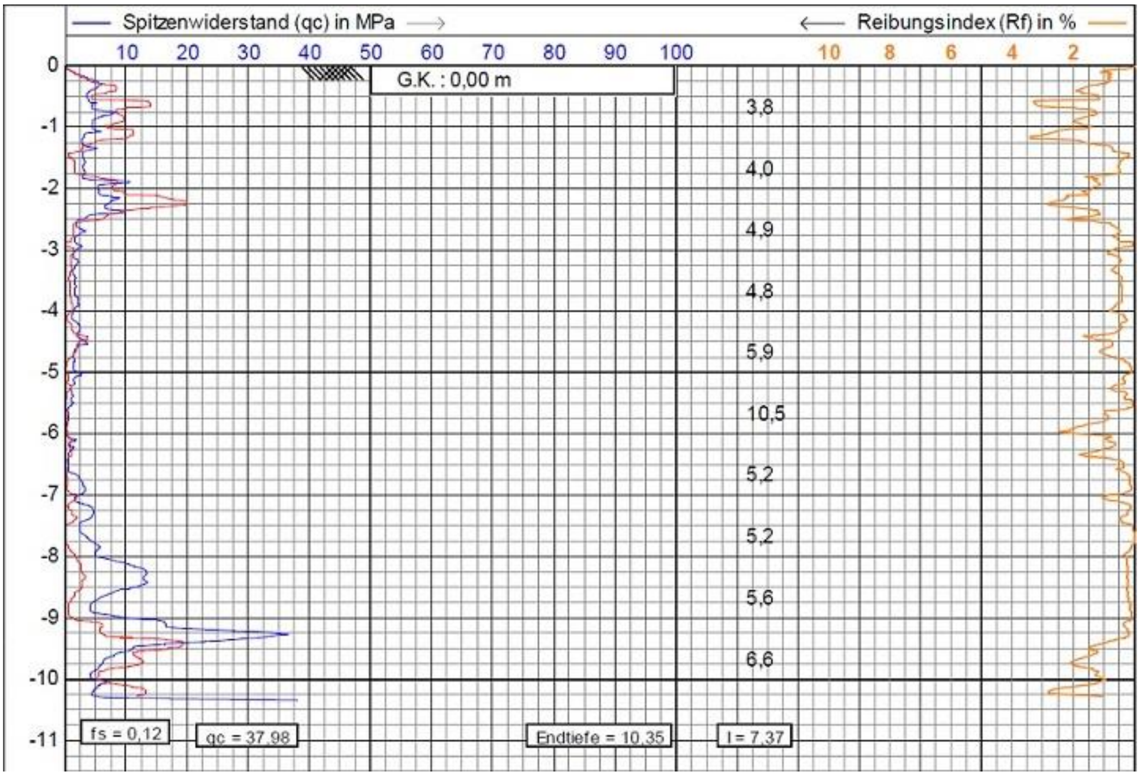


Fig. 19 CPT-data of a project of Max Bögl; $q_c$in blue, f_s in red in the middle the I-Index which is the soil behavior type Index after *Robertson* and described in the chapter soil type charts and on the right side the R_f in orange.

Fig. 19 shows how CPT data is provided, the description is in German. There are several different designs of providing the data. Often the data of one parameter is plotted in their own diagram. In a later chapter this profile will be discussed.

6.4 Correlation of the individual tests

All correlations require an engineering judgment. Lots of authors are very careful concerning the application of such correlations and as a result, awareness of these results should be ensured for the user. This chapter gives a short overview and to get a first impression of different correlations in between CPTu- data and soil parameters.

"These correlations are approximate and their use requires the experience of engineering judgment regarding the inevitable uncertainties in estimated property values". (*McGregor et al., 1998)*

Compactness	Id	SPT N30 (Terzaghi, 1967)	SPT N30 (Kolymbas, 2007)	SPT N30 (Prinz, 2010)	SPT N30 (Floss, 2006)	CPT qc [MN/m²] (Prinz, 2010)	CPT qc [MN/m²] (Floss, 1979)	DPL N10 (Floss, 1979)	DPL N10 (Prinz, 2010)	DPM N10 (Prinz, 2010)	DPH N10 (Prinz, 2010)	DPH N10 (ICP, Presentation)
very loose	0.15-0.15	0-4	0-4	0-3	0-4		<5	<10	0-6	0-4	0-1	0-1
loose	0.15-0.35	4-10	4-10	3-8	4-10	<5/7.5	5-10	10-20	6-10	4-11	1-4	1-4
middle dense	0.3-0.65	10-30	10-30	8-25	10-30	5/7.5-10/15	10-15	20-30	10-50	11-26	4-13	4-13
dense	0.65-0.85	30-50	30-50	25-42	30-50	10/15-20/25	15-20	30-40	50-64	26-44	13-24	13-24
very dense	>0.85	>50	>50	42-58	>50	>20/25	>20	>40	>64	>44	>24	>24
Consistency	Ic											
very soft	0-0.5		0-2	0-2	0-2				0-3	0-3	0-2	0-2
soft	0.5-0.75		2-4	2-4	2-4	1.0-1.5			3-10	3-8	2-5 (4)	2-5
middle	0.75-1.00		4-8	4-8	4-8							
stiff			8-15	8-15	8-15	1.5-2.5			10-17	8-14	(4) 5-9 (8)	5-9
very stiff	>1.00		15-30	15-30	15-30	2.5-5.0			17-37	14-28	(8) 9-17	9-17
hard	>1.00		30	>30	>30	>5.0			>37	>28	>17	>17

Tab. 11 Different literature values of different tests

All tests have already been standardized but in different literature sources are still some differences which may lead to a wrong interpretation. Especially if some values are directly at the border of two consistencies or densities, that can lead to wrong interpretations of the data and therefore to a wrong geological model. Therefore are the result influenced by the shape of grains, boulders, blocks, waste, etc. The occurrence of these materials often leads to higher values and to a miss interpretation of the compactness or consistency.

Lagerung	Spitzendruck q_c [MN/m²]	DPH N_{10H}	DPM N_{10M}	DPL N_{10L}	SPT N_{30}
locker	< 5/7,5	1-4	4-11	6-10	3-8
mitteldicht	5/7,5-10/15	4-13	11-26	10-50	8-25
dicht	10/15-20/25	13-24	26-44	50-64	25-42
sehr dicht	> 20/25	> 24	> 44	> 64	42-58
Konsistenz	**Spitzendruck q_c [MN/m²]**	**DPH N_{10}**	**DPM N_{10}**	**DPL N_{10}**	**SPT N_{30}**
weich	1,0-1,5	2-5 (4)	3-8	3-10	2-8
steif	1,5-2,5	(4) 5-9 (8)	8-14	10-17	8-15
halbfest	2,5-5,0	(8) 9-17	14-28	17-37	15-30
fest	> 5,0	> 17	> 28	> 37	>30

Fig. 20 Literature values of different test after *(Prinz & Strauß, 2010)*

In Fig. 20 *Prinz & Strauß, (2010)* show an easy way of correlation of the different tests. They use the density and the consistency to specify the test result values to their subdivisions.

Correlation between CPT and SPT

From the late 1960s to the 1990s, there was a large amount of research being conducted in order to establish a connection between the SPT blow counts, and results from CPT. This aspect is caused by the fact that the CPT has some advantages over the SPT:

- Provides much better resolution, reliability
- Versatility; pore water pressure, dynamic soil properties

	In-Situ Test Results	Relative Density	ϕ' (degrees)	
			(a) [3]	(b) [4]
SPT N-Value [1] (blows/300 mm or blows/ft)	0 to 4	Very Loose	< 28	< 30
	4 to 10	Loose	28 to 30	30 to 35
	10 to 30	Medium	30 to 36	35 to 40
	30 to 50	Dense	36 to 41	40 to 45
	> 50	Very Dense	> 41	> 45
Normalized CPT cone bearing resistance (q_c/P_a) [2], [4]	< 20	Very Loose	< 30	
	20 to 40	Loose	30 to 35	
	40 to 120	Medium	35 to 40	
	120 to 200	Dense	40 to 45	
	> 200	Very Dense	> 45	

Notes:
(1) SPT N-values are field, uncorrected values.
(2) P_a is the normal atmospheric pressure = 1 atm ~ 100 kN/m^2 ~ 1 tsf.
(3) Range in column (a) from Peck, Hanson, and Thornburn (1974).
(4) Ranges in column (b) and for CPT are from Meyerhof (1956).

Tab. 12 Correlations between SPT and CPT results and friction angle of cohesionless soils after *Kulhway and Maine, (1990)*

There are many different correlations between all tests published. For example shows Tab.12 the values of SPT and CPT tests and relates the relative density and friction angle to the test results. Tab.13 shows a correlation of the undrained shear strength.

Equation	Soil Condition	Associated Test
$S_u = 0.29\, P_a\, N^{0.72}$	Normally consolidated to lightly overconsolidated	SPT
$S_u = \dfrac{q_c - \sigma_{vo}}{N_k}$	Various	CPT

Symbols: P_a = atmospheric pressure, N = uncorrected SPT blow count, q_c = cone tip resistance, σ_{vo} = total overburden stress (same units as q_c), N_K = cone factor; typical value is 15.

Tab. 13 Correlations between SPT and CPT results and undrained strength of fine-grained soils, (*Kulhway and Maine,1990)*

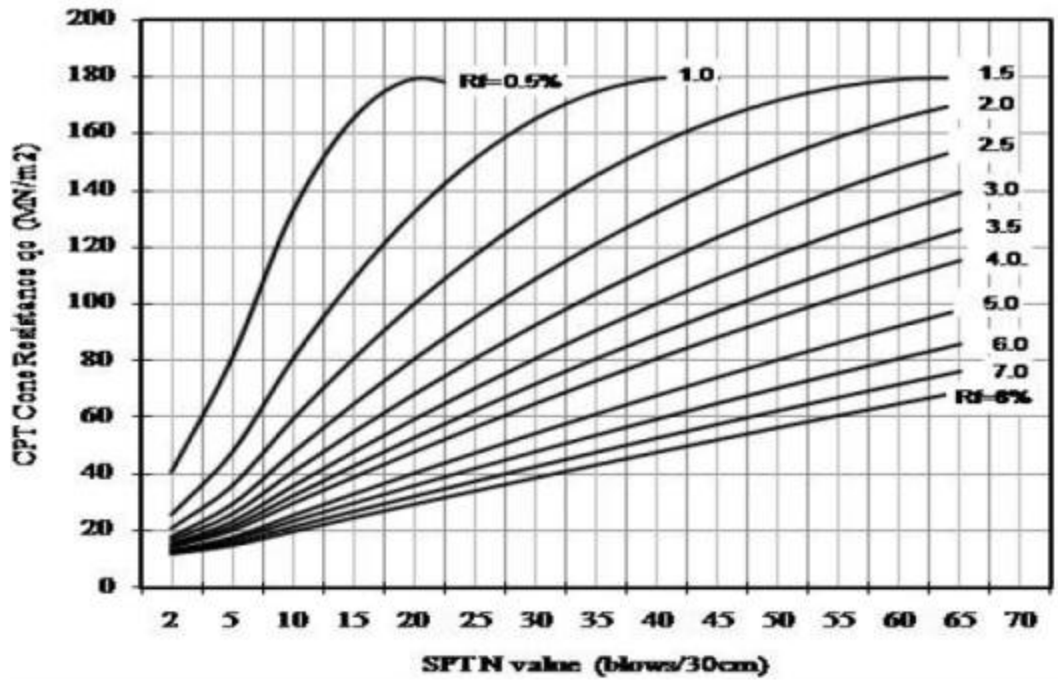


Fig. 21 Combined $q_c - R_f - N_{30}$ correlation chart for soils *(Zein, 2002)*

This diagram from *Zein, (2002)* shows correlation results from his research work. It uses not only q_c and N_{30}, but also the R_f value. This leads to more precise results according to the published paper *(Zein, 2002).*

Bodenart	q_s/n_{30} (MN/m^2)
Fein- bis Mittelsand oder leicht schluffiger Sand	0,3 bis 0,4
Sand oder Sand mit etwas Kies	0,5 bis 0,6
weitgestufter Sand	0,5 bis 1,0
sandiger Kies oder Kies	0,8 bis 1,0

Tab. 14 Relationship between tip cone resistance q_c and the N_{30} value after *DIN 4014*, $q_c = q_s$

On the left side of Tab. 14 is the soil type described. On the first line is fine grained to middle grained sand or low silty sand mentioned. The second one is sand or sand with less gravel and the third is a wide graded gravel, the last one is gravel or sandy gravel. The tendency is that the coarse grained materials have higher correlation values than the fine grained soils. The grain size distribution is the problem, the variability of the behavior of the different soil types lead to different relationships. The *DIN 4094* provides following correlations.

For uniformed graded sands:

$$q_c = 0.5 * N_{30}$$

For well graded sand gravel mixtures:

$$q_c = 0.7 * N_{30}$$

For uniform graded gravels:

$$q_c = 1.1 * N_{30}$$

Elkateb et al, (2010) published the following correlation. He implements the parameter I_c, which is the soil behavior type index (*Jefferies and Davies, 1993).*The soil behavior type index is described in more detail in chapter 7.1.

$$N_{60} = \frac{q_c * 10^3}{8.5 * p_a * (1 - \frac{I_c}{4.6})}$$

There are more correlations, a good overview is given in a table from the work of *Kara & Gündüz, (2010).*

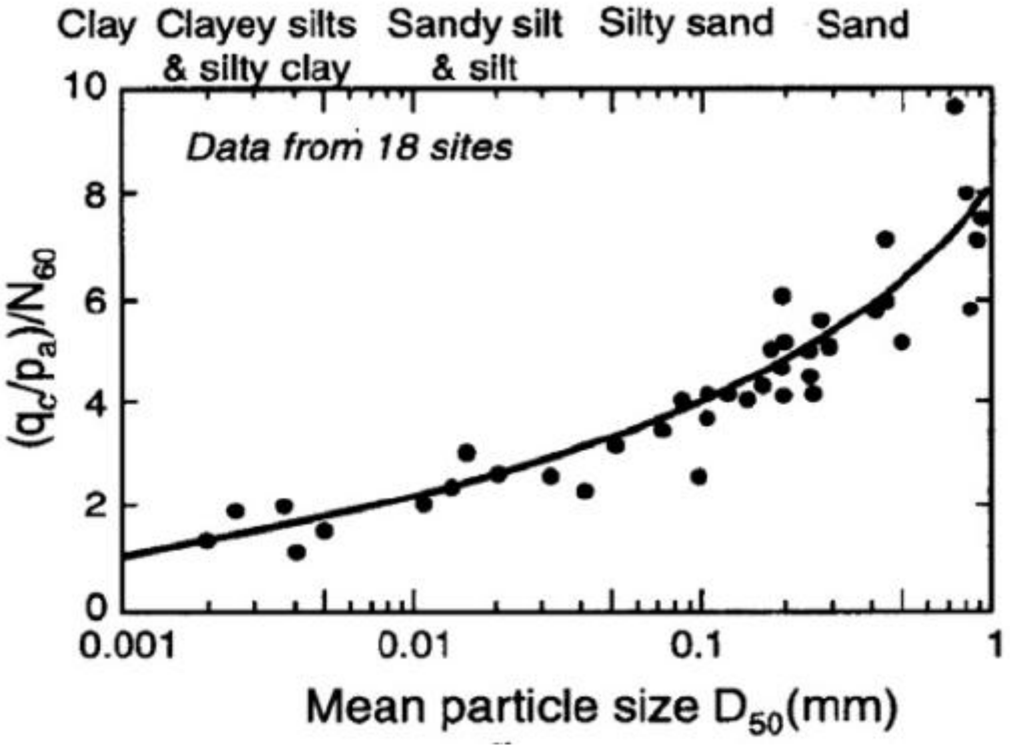
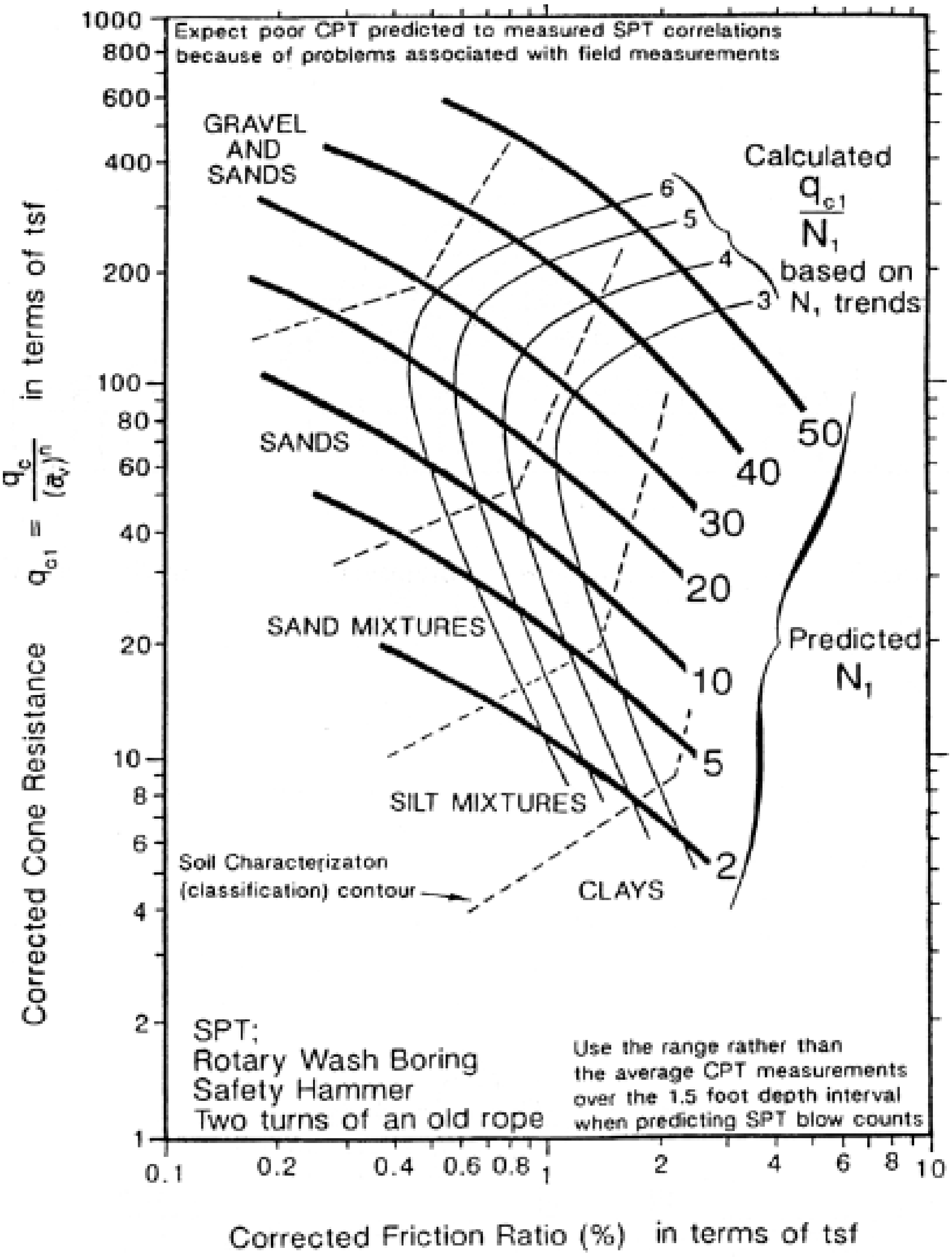


Fig. 22 CPT-SPT correlation with grain size, after *Robertson et al., (1983)*

Robertson et al, (1983) developed a correlation with the data from 18 different sites which are illustrated in Fig 22. *Robertson* uses the mean particle size to evaluate the relationship of q_c divided by the atmospheric pressure to the N_{60} value. The curve fits through most of the data points or at least quite close, but there are some outliners as well. "These values provide a reasonable estimate of SPT N_{60} values from CPT data. For simplicity the above correlations are given in terms of q_c. For fine grained soft soils the correlations should be applied to the total cone resistance q_t", *(Practical Applications of the Cone Penetration Test, 2007). Jefferies and Davies, (1993)* suggested that this approach can provide better results than the actual SPT test values caused by the poor repeatability of the SPT.



$$FR_1 = \frac{f_{s1}}{q_{c1}} 100 = \frac{f_s / \bar{\sigma}_v}{q_c / (\bar{\sigma}_v)^n} 100 = \frac{f_s}{q_c} \frac{1}{(\bar{\sigma}_v)^{(1-n)}} 100$$

Fig. 23 SPT- CPT correlation from *Olsen, (1988)*

This diagram from *Olsen (1988)* was developed in the United States which is visible due to the fact that the unit of the y-axis is [tsf], which are tons per square foot. Professor Rogers of the University of Missouri preferred this diagram in his online lectures, applicable for the usage in engineering practice. The x-axis is the corrected friction ratio and the SPT values can be read in the middle where the different N lines are plotted.

Correlation between DP and CPT

Soil classification*	SPT q_c/N_{30}	DPH q_c/N_{30}	DPL q_c/N_{30}
SE	0.5	0.7	0.25
SW, SI	0.7	1.0	0.35
GE, GW, GI	1.1	1.5	—

* S — sand, G — sand and gravel, E — poorly graded (even-graded), W — well graded (multi--graded), I — poorly graded with some grain diameter missing (gap-graded)

Tab. 15 Correlation ratios for cohesionless soils according to *Biedermann, (1978) (in Smoltczyk, 2001)*

Tab.15 was published by *Biedermann, (1978)* and shows correlations for SPT to CPT, DPH to CPT and for DPL to CPT. *Biedermann* uses different values for every soil classification and for the DPH and for DPL he calculates with the N_{30} value which is not standard for Dynamic Probing.

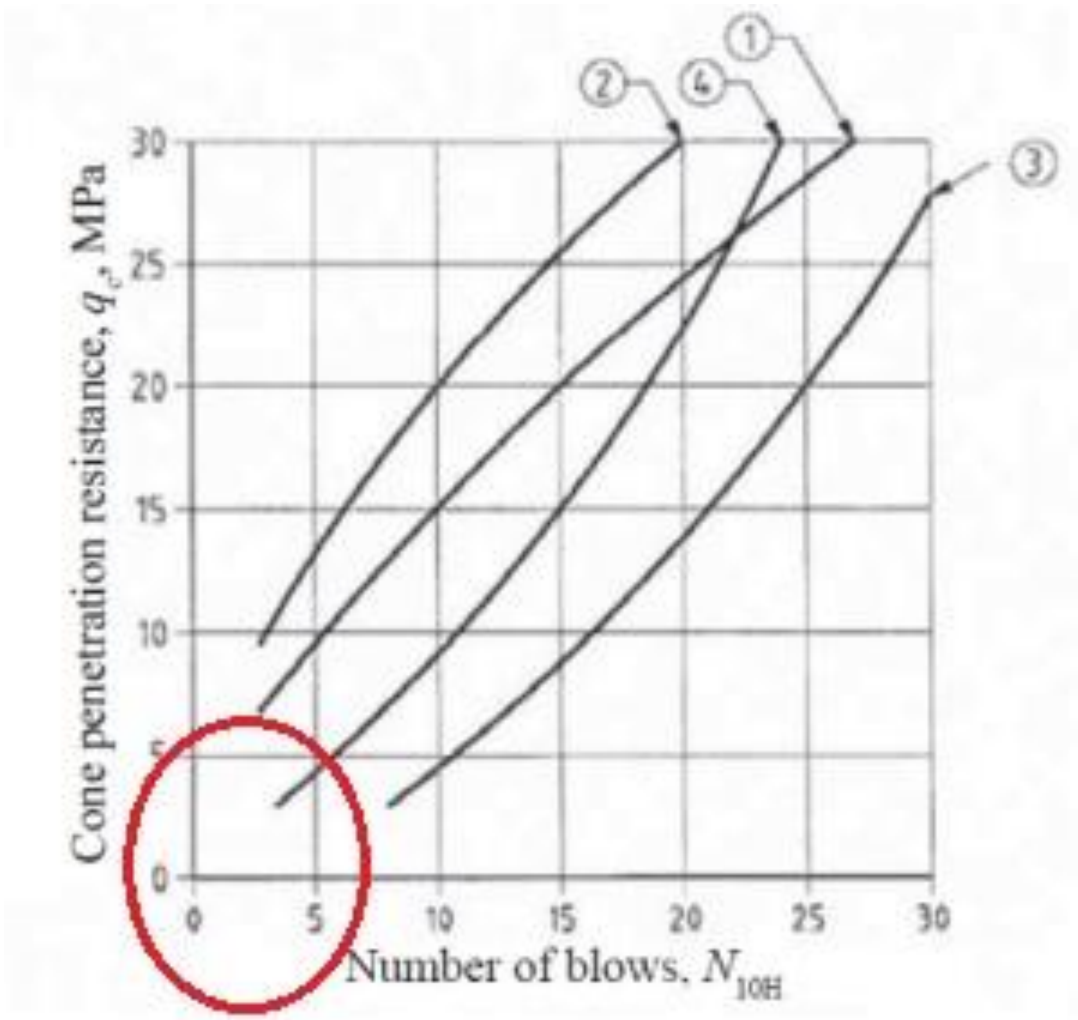


Fig. 24 Correlations between CPT and DPH, 1- poorly graded sand above groundwater, 2- poorly graded sand below groundwater, 3- well graded sand gravel above groundwater, 4- well graded sand and gravel below groundwater according to *DIN 4094*

DIN 4094 provides Fig.24, that diagram is not very useful for loose densities or soft consistencies because of the area from0 to 5 [MPa], which is marked with the red cycle. The area is not touched by 3 of the 4 correlation lines. So it is not possible to make correlations for all materials which have a lower q_c than 5 [MPa].

Butcher et al, (1995) published these correlations for clays:

For soft clay:

$$q_t = 0.24 * q_d + 0.14$$

For stiff clay:

$$q_t = q_d$$

Correlation between SPT and DP

DIN 4094 provides the following correlations. Additional correlations are found for SPT to DP in *Sachsenhofer, (2012).*

Coarse grained soils above the groundwater

DPH: $3 \leq N_{10} \leq 50$

$$N_{30} = 1.4 * N_{10}$$

Clay with low and medium plasticity, above groundwater

DPH: $2 \leq N_{10} \leq 13$

$$N_{30} = 1.0 * N_{10} + 3$$

DPL: $2 \leq N_{10} \leq 30$

$$N_{30} = 0.6 * N_{10}$$

DPH:$3 \leq N_{30} \leq 15$

$$N_{10} = 0.5 * N_{30} + 2$$

DPH: $3 \leq N_{30} \leq 17$

$$N_{10} = 1.5 * N_{30}$$

7 Determination of the constrained modulus

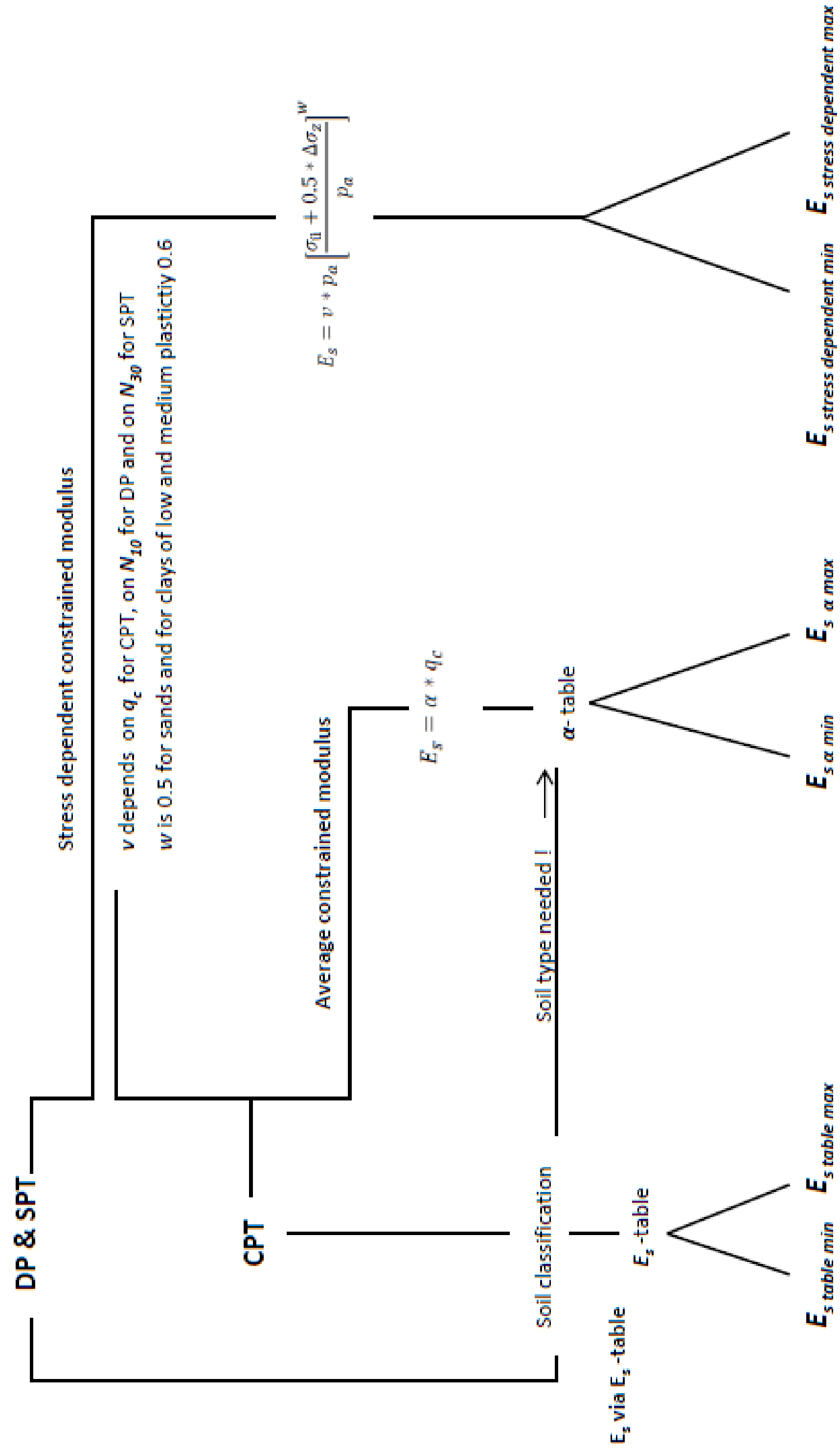


Fig. 25 Three possibilities to determine the constrained modulus for a soil with the data of CPT and SPT, for all DP devices work the way via the constrained modulus table, stress dependent only for DPH

7.1 The constrained modulus via constrained modulus table (E_s via E_s-table)

The determination of the constrained modulus by the constrained modulus table is the most common way in practice. This is the easiest one but is not very precise. But obtaining first results is possible after a short time and especially when it comes to practical work the first approach to data of a project is to get an idea which values can be problematic. For a first rough estimation a conservative approach is necessary and this method is often used in practical work.

Overview soil type classification charts

“It is often important to realize that the classification charts are generalized global charts that provide a guide to the soil behavior type. The charts cannot be expected to provide accurate predictions of the soil type for all soil conditions. However, in specific geological areas the charts can be adjusted for local experience to provide excellent local correlations”, citied from the manual *Practical Applications of the Cone Penetration Test*. This chapter shows a short overview for some well known charts which are used in the German construction sector. A greater overview for more charts is the work of *Sachsenhofer, (2012)* and *Van T Veen, (2015)* where several more examples of soil type classification charts are examined.

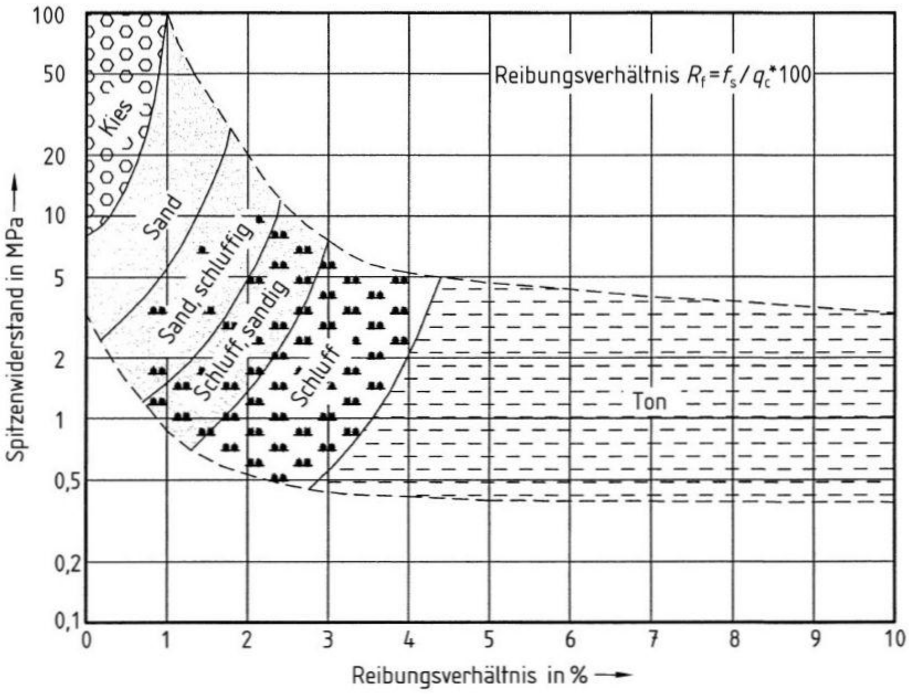


Fig. 26 Soil type classification chart which is recommended by *DIN 4094*

The soil type classification chart, which is shown in Fig. 26 is recommended by the *DIN 4094* and there usage is widely spread in Germany. This Soil type chart has no field for peat and organic soils.

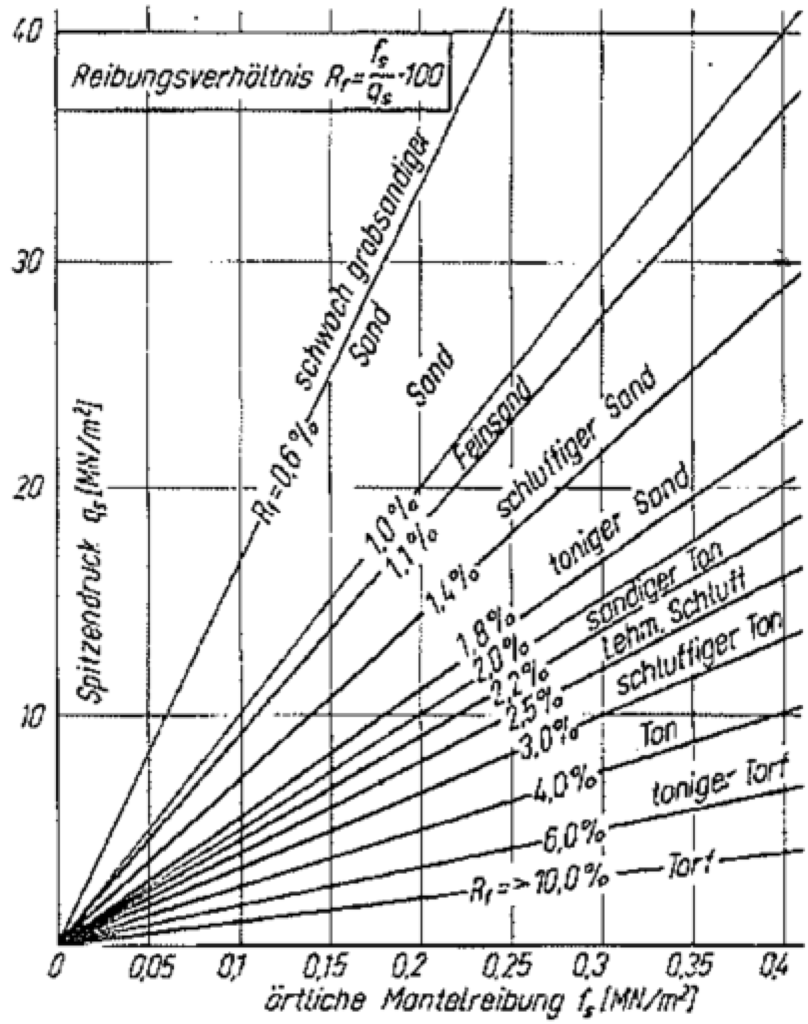


Fig. 27 Soil type classification chart of the *Fugro Company, (Jacobs,1996)*

The *Furgo Company* is well known in the CPT sector. They have published these two charts an older one shown in Fig 27. and a newer one displayed in Fig. 28. In a report of the *Austrian Society for Geomachanics, (2013)* they got quite good references for their charts, but in the report there is no description about which one is meant. Generally the references of the company are well, so this can lead to the assumption that their results are reliable.

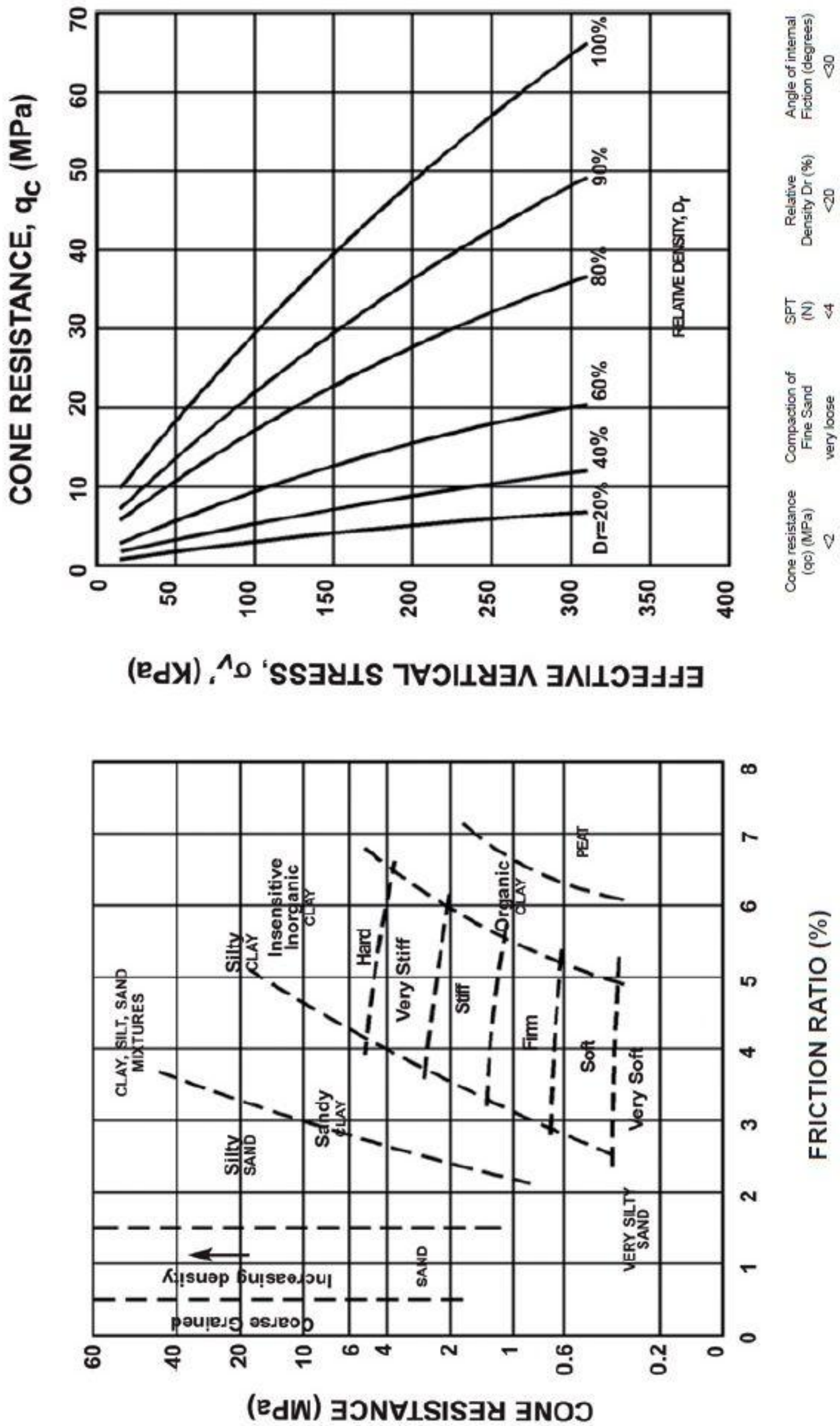


Fig. 28 New soil type classification chart of the *Fugro Company*

With the usage of the newer *Fugro* soil type classification chart, it is also possible to get a value for the relative density. The basis for this diagram was developed by *Robertson and Campanella, (1983)* for unaged and uncemented quartz sands. On the right hand diagram in Fig. 28 a value for the relative density of a soil can be created. The parameters of the effective vertical stress and the cone resistance are needed to classify the density. The left hand diagram shows a soil type classification chart which has areas for every material plus areas for every consistency state of clay, except over consolidated clay.

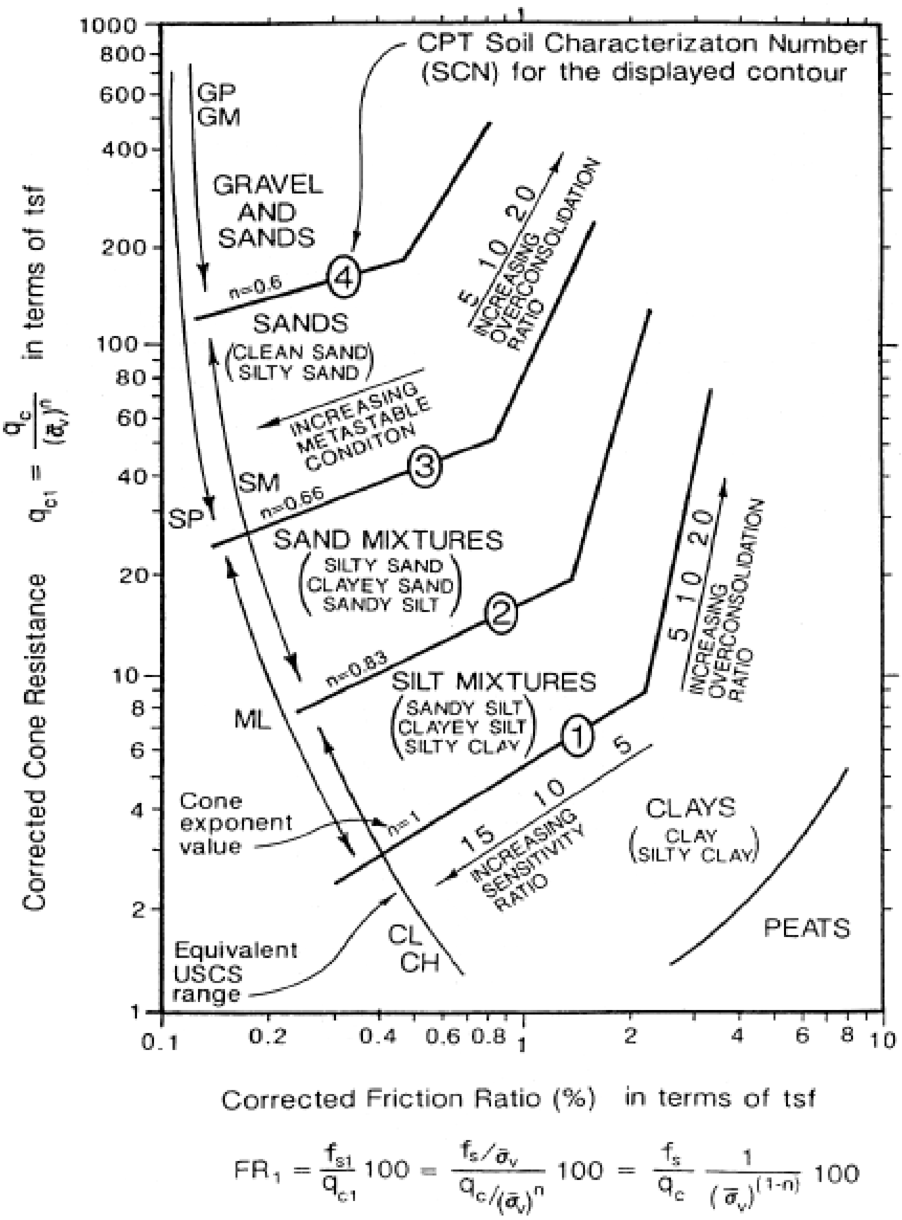


Fig. 29 This is a soil type classification chart which is often used in the US and UK, the unit for a force is [tsf] which means tons per square foot. 1[tsf] = 0.096 [MPa], after *Olsen, (1988)*

This chart is not well known in Europe, maybe this is become of the units [tsf] which are not standard in Europe. Professor Rogers of the University of Missouri prefers this chart in his online lectures. His lectures are watchable on YouTube. He also experienced that the Olsen chart for the conversion from SPT data to CPT shows quite reliable results.

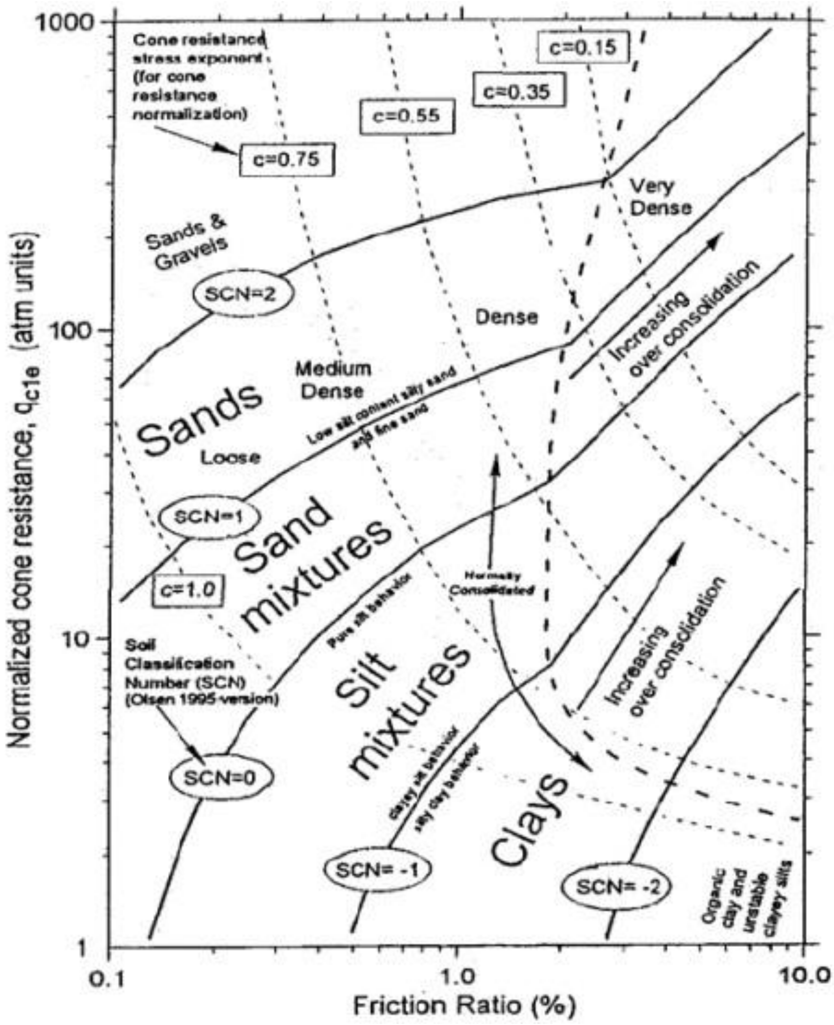


Fig. 30 This is another soil type chart from *Olsen, (1994),* with usage of the unit [atm], 1 [atm] = 0.101325 [MPa]

In Fig. 30 uses Olsen the soil classification number (SCN) to divide the different areas, these field are defined by SCN which is defined as follws:

- SCN = 0 represents pure silt
- SCN = 1 represents a fine sand or low silt content silty sand
- SCN = -1 represents the boundary between silty clay and clayey silt
- SCN ≥ 1 represents sand
- SCN ≤ -1 represents clay

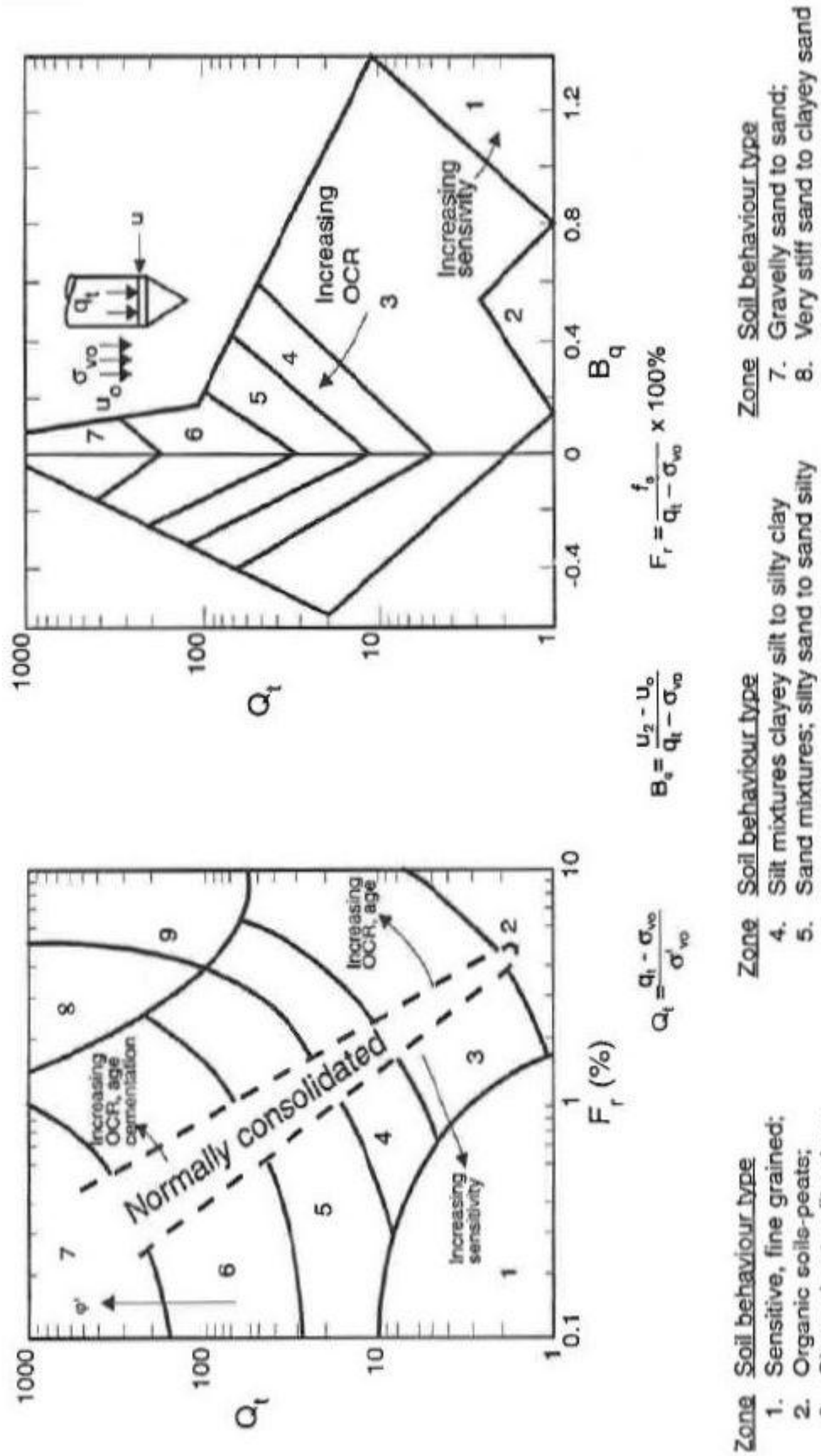


Fig. 31 Soil type classification chart based on normalized CPT/CPTu data after *Robertson, (1990)*

In Fig.31 a popular soil classification chart based on normalized CPT data is shown. This diagram from *Robertson, (1990)* indentifies general trends in the ground parameters, such as the OCR, the friction angle, the cementation for sandy soils and the soil sensitivity for cohesive soils. The chart is global in nature and provides only a guide to the soil behavior type. Overlap in some zones should be expected and the zones should be adjusted somewhat based on local experience *(Practical Applications of the Cone Penetration Test, 2007).*

Roberston & Wride, (1998) introduced the following equation to simplify the application of the soil type classification chart in Fig. 31. They combined the parameters of the normalized cone penetration resistance and the friction ratio into one soil behavior type index.

$$I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5}$$

$$Q = \frac{(q_c - \sigma_{vo})}{p_a} * \left(\frac{p_a}{\sigma'_{vo}}\right)^n$$

$$F = \left[\frac{f_s}{(q_c - \sigma_{vo})}\right] * 100\%$$

Soil Behavior Type Index, I_C	Zone	Soil Behavior Type
I_C<1.31	7	Gravelly sand to dense sand
1.31<I_C<2.05	6	Sands: clean sand to silty sand
2.05<I_C<2.60	5	Sand Mixtures: silty sand to sandy silt
2.60<I_C<2.95	4	Silt Mixtures: clayey silt to silty clay
2.95<I_C<3.60	3	Clays: silty clay to clay
3.60<I_C	2	Organic soils: peats

Tab. 16 Boundaries of soil behavior type index *Robertson and Wride (1998)*

The soil behavior type index I_c, developed by *Robertson and Wride (1998),* is a useful tool to define a soil's behavior type, which is defined according to a set of numerical boundaries, shown in Tab.16. *Robertson and Wride (1998)* followed the same principles set by *Jefferies and Davies (1993)*, in which the soil behavior type index I_c, was determined by the radius of each concentric circle acting as the boundaries between the soil behavior type zones.

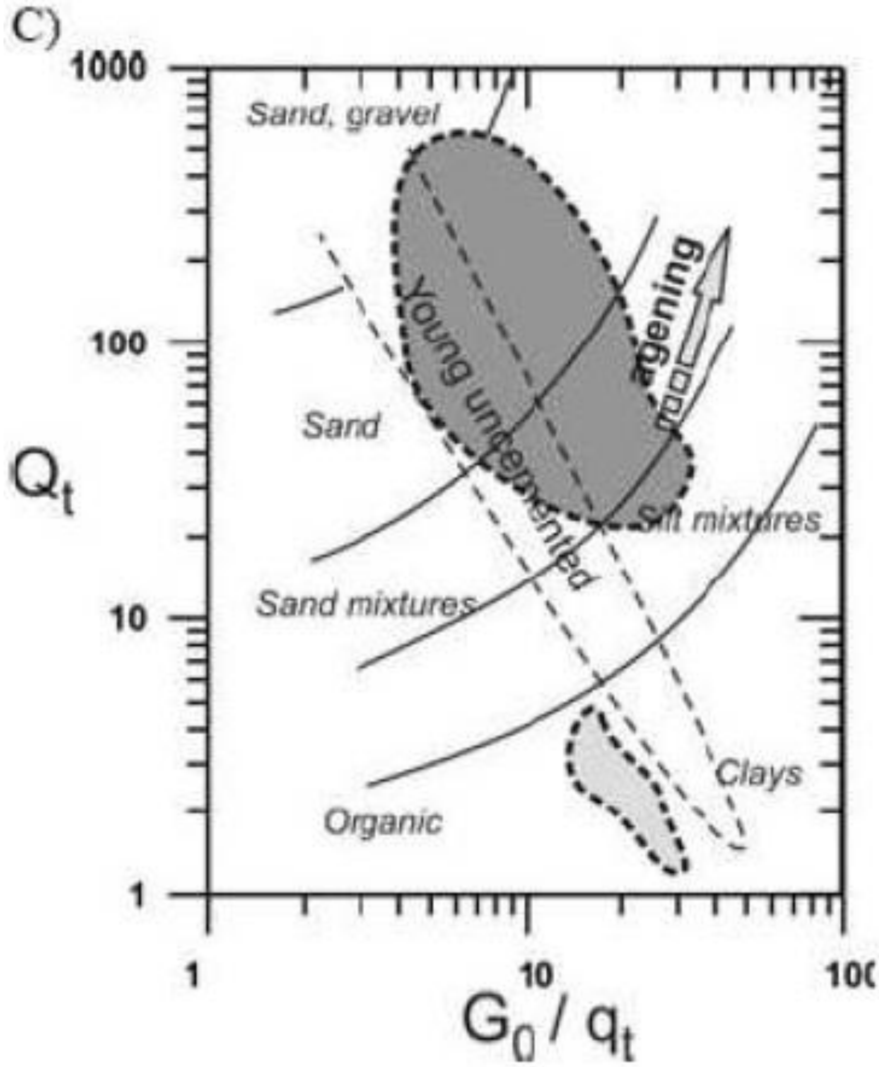


Fig. 32 Soil type classification chart with usage of the seismic value G_0 *(Robertson et al. 1995)*

This diagram has the best references for organic soils and peat according to *Mlynarek et al, (2010).* He published a study in organic soils where he first used the standard diagram from *Robertson, (1990)* due to the erroneous interpretation he recommends the use of seismic measurements to provide the usage of the diagram in Fig. 32.

E_s-table values

The values in Tab. 17 are collected from different authors and show the main experiences of them. There are constrained modulus values which are reliably and as an example for sand it is already tried to differ between rounded and angular particles. But the main point, which is clearly shown in this table, is that the range of the constrained modulus is still high for one soil type even when the density or the consistency is known. It was tried to get as much information as possible in the table, such as the shape of grains, the compactness and the consistency. Some data had no information about these attributes so they are plotted in the right column, which is named no info (no information).

Type of Soil	shape of grains		Compactness, Es [MN/m²]		
		loose	**middle dense**	**dense**	no info
gravel		29-77	77-96	96-191	
					100-200
sandy gravel		30-80	80-100	100-200	
sand		9.5-29	29-48	48-77	
	rounded	20-50	50-100		
	angular	40-80	80-150		
			30-50	50-80	
fine sand		8-12	12-20	20-30	
		7-11	11-19	19-28	
			Consistency, Es [MN/m²]		
		soft	**stiff**	**very stiff**	no info
coarse silt		5-8	10-15	20-40	
silt					3-10
		3-6	6-10	15-30	
					2-19
organic silt			0.5-5		
clay		1-2.5	2.5-5	5-10	
		0.4-4	3-8.5	7-17	
clay (skinny)		2-5	5-8	12-20	
clay (fat)		1.5-4	4-7	12-30	
organic clay			0.5-4		
sandy clay					28-42
till					30-100
peat					0.1-2
loam		4-8		5-20	
loess					14-57

Tab. 17 Constrained modulus values, E_s [MN/m²], according to *Kêzdi* in *Floss, (1979); Richter, (1989); EAU, (1990) and Aashto, (2004) & (2006)*

7.2 The constrained modulus via average constrained modulus according to *DIN 4094* (E_s via α-table)

This way of determining the constrained modulus via the average constrained modulus is more precise. The cone tip resistance q_c is corrected with the regional factor α. To choose an appropriate α value, the soil type has to be considered (for example sand, clay, etc…). Knowing this, it is now the task of the editor to evaluate and to choose the most meaningful value for α. This requires experience; the result here will be a two solutions result, one minimum and one maximum. The minimum value is often used which leads to an "error", caused by the conservative assumption in general.

$$E_s = \alpha * q_c$$

q_c= cone tip resistance

α = regional factor

Soil	Cone resistance q_c [Mpa]	Regional factor α
Clay, low plasticity	$q_c < 0.7$ MPa	$3 < \alpha < 8$
	0.7 MPa $< q_c <$ 2 MPa	$3 < \alpha < 8$
	$q_c > 2$ MPa	$1 < \alpha < 2.5$
Clay, high plasticity	$q_c < 2$ MPa	$2 < \alpha < 6$
Peat & Clay, highly organic	$q_c < 0.7$ Mpa	
depends on the water content	50 % $< W_n <$ 100 %	$1.5 < \alpha < 4$
	100 % $< W_n <$ 200 %	$1 < \alpha < 1.5$
	$W_n >$ 500 %	$\alpha < 0.4$
Silt,low plasticity	$q_c < 2$ MPa	$3 < \alpha < 6$
	$q_c > 2$ MPa	$1 < \alpha < 2$
Silt, highly compressible	$q_c < 2$ MPa	$1 < \alpha < 2$
Silt, highly organic	$q_c < 1.2$ MPa	$2 < \alpha < 8$
silty Sand		$\alpha = 2$
fine and middle grained Sand		$\alpha = 3.5$
coarse grained Sand, gravelly Sands		$\alpha = 5$
sandy Gravels and Gravel		$\alpha = 6$

Fig. 33 α-values translated into English and according to *DIN 4094*

$M=\frac{1}{m_v}=\alpha q_t$		
$q_t < 7$ bar $7 < q_t < 20$ bar $q_t > 20$ bar	$3 < \alpha < 8$ $2 < \alpha < 5$ $1 < \alpha < 2.5$	Clay of low plasticity (CL)
$q_t > 20$ bar $q_t < 20$ bar	$3 < \alpha < 6$ $1 < \alpha < 3$	Silts of low plasticity (ML)
$q_t < 20$ bar	$2 < \alpha < 6$	Highly plastic silts & clays (MH, CH)
$q_t < 12$ bar	$2 < \alpha < 8$	Organic silts (OL)
$q_t < 7$ bar: $50 < w < 100$ $100 < w < 200$ $w > 200$	$1.5 < \alpha < 4$ $1 < \alpha < 1.5$ $0.4 < \alpha < 1$	Peat & organic clay (P_t, OH)

Fig. 34 Estimation of the constrained modulus, M, for clays after *Mitchell and Gardner, (1975)*

The results of *Mitchell and Gardner, (1975)* are nearly the same as the values for α in the *DIN 4094*. A small difference is found for the values of silts of low plasticity, according to *Sachsenhofer, (2012)* probably a mistake in the *DIN*.

Other Authors describe the usage of the same α-values for every soil type.

Author	α value
Mayne (2001)	8
Kuhlway and Mayne (1990)	8.25
Senneset et al.(1989)	4-8
Meigh (1987)	2-8

Tab. 18 Single α-values for every soil type from different authors

Robertson (2009) published another α correlated equation which is related to the soil behavior type index.

$$E_s = \alpha * (q_c - \sigma_{vo})$$

For $I_c > 2.2$

$$\alpha = Q_t \, , when \; Q_t > 14$$

$$\alpha = 14, when\ Q_t > 14$$

For $I_c < 2.2$

$$\alpha = 0.03 * [10^{0.55*I_c+1.68}] * (q_t - \sigma_{vo})$$

Lunne and Christopherson, (1983) published the following α-values.

NC, uncemented sand:

For $q_c < 10\ [MPa]$

$$E_s = 4 * q_c$$

For $10\ [MPa] < q_c < 50\ [MPa]$

$$E_s = 2 * q_c + 20\ [MPa]$$

For $q_c > 50\ [MPa]$

$$E_s = 120[MPa]$$

OC sands:

For $q_c < 50[MPa]$

$$E_s = 5 * q_c$$

For $q_c > 50[MPa]$

$$E_s = 250[MPa]$$

7.3 Constrained modulus via stress dependent constrained modulus according to *DIN 4094*

CPT

$$E_s = v * p_a \left[\frac{\sigma_{\ddot{u}} + 0.5 * \Delta\sigma_z}{p_a}\right]^w$$

E_s = constrained modulus

v = stiffness factor

p_a = atmospheric pressure (p_a = 0.1 [MPa])

$\sigma_{\ddot{u}}$ = vertical stress at the foundation level, after *DIN 4019* $[\sigma_{\ddot{u}} = \gamma * z]$

$\Delta\sigma_z$= increase of the vertical stresses at the foundation level due to loading, after *DIN 4019* $[\Delta\sigma_z = i * \sigma_1]$, i is used in the tables of *Steinbrenner*

w =stiffness exponent (w=0.5 for sands, and w=0.6 for clays of low and medium plasticity)

Values for v:

For clays with w=0.6; ($0.6\ [MPa] \leq q_c \leq 3.5\ [MPa]$)

$$v = 15.2 * log(q_c) + 50$$

For sands with w=0.5; ($5\ [MPa] \leq q_c \leq 30\ [MPa]$)

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

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

The cone resistance is required as well as the knowledge of the Coefficient of Uniformity when q_c is higher or equal to 5 [MPa]. The range between the values 3.5 [MPa] and 5 [MPa] for q_c is not discussed in the *DIN 4094* and neither is the case that clay has a higher cone tip resistance than 3.5 [MPa]. On the other side it is not defined when sand has lower cone tip resistance than 5 [MPa]. Due to the fact that not all [MPa] values are defined the area of application of this formula is limited.

DP

The formula is the same, but to evaluate the parameter v, it is necessary to examine the N_{10} value, the *DIN 4094* defines:

For sands with w= 0.5, above the groundwater table:

DPH: $(3 \leq N_{10} \leq 10)$

$$v = 249 * log(N_{10}) + 161$$

DPL: $(4 \leq N_{10} \leq 50)$

$$v = 214 * log(N_{10}) + 71$$

For clays of low and medium plasticity w=0.6, above the groundwater table:

DPH: $(6 \leq N_{10} \leq 13)$

$$v = 6 * N_{10} + 50$$

DPL: $(6 \leq N_{10} \leq 19)$

$$v = 4 * N_{10} + 30$$

SPT

In the German *DIN 4094* is the modified SPT called BDP:

For sands with w=0.5, above the groundwater table:

BDP: $(3 \leq N_{30} \leq 25)$

$$v = 217 * log(N_{30}) + 146$$

For clays of low and medium plasticity w=0.6, above the groundwater table

BDP: $(3 \leq N_{30} \leq 23)$

$$v = 4 * N_{30} + 50$$

8 Application for an example of a construction

For my master thesis I was provided by Max Bögl with data from their construction sites. Fig.35 shows the works steps which were required to get to from field data to settlement estimations for auxiliary foundations.

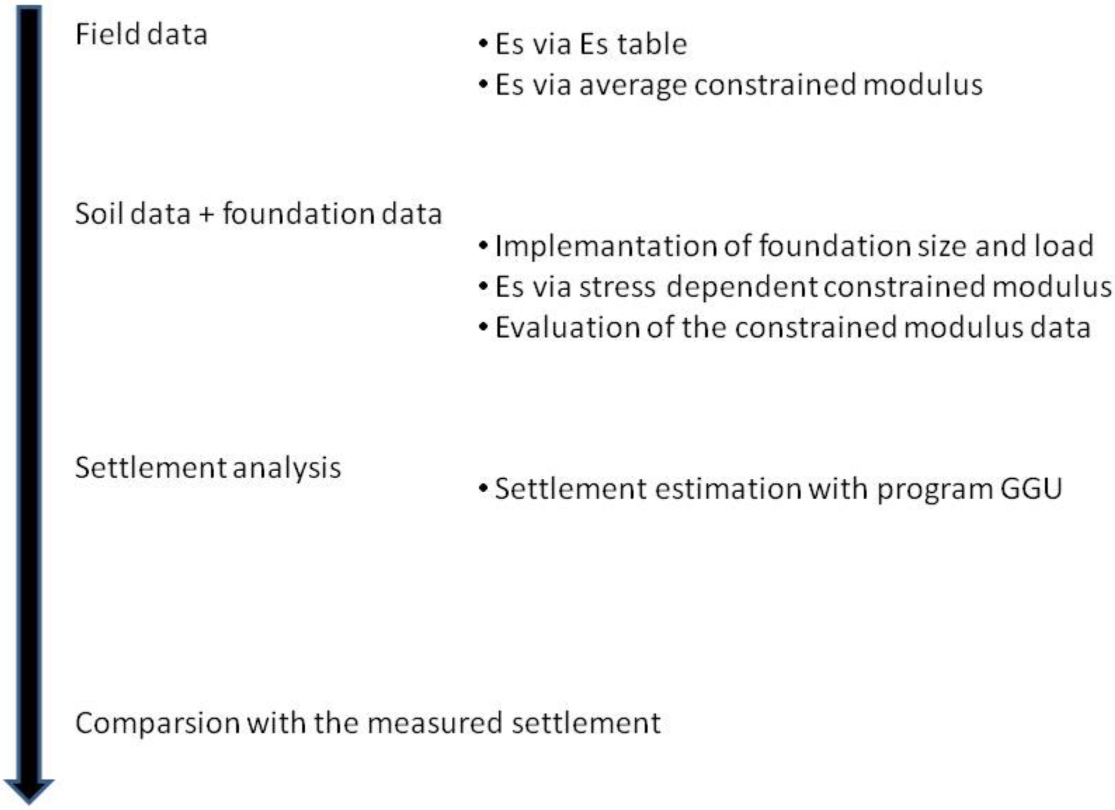


Fig. 35 Work steps of how the field data is handled

The raw field data is interpreted after a scheduled visit at the construction site and after reading and analyzing the geotechnical report. The result of the CPT and the DPH data is used to calculate the constrained modulus values, via all different ways. With the information of the foundation size and its load it is possible to calculate the stress dependent constrained modulus. After that, all different calculation ways were analyzed and according to these results a representing value is established for every stratigraphic layer. A settlement analysis with the program GGU footing using the most reliable parameters and a comparison of the resulting data and the measured settlement data is done.

8.1 Project Schierstein Bridge

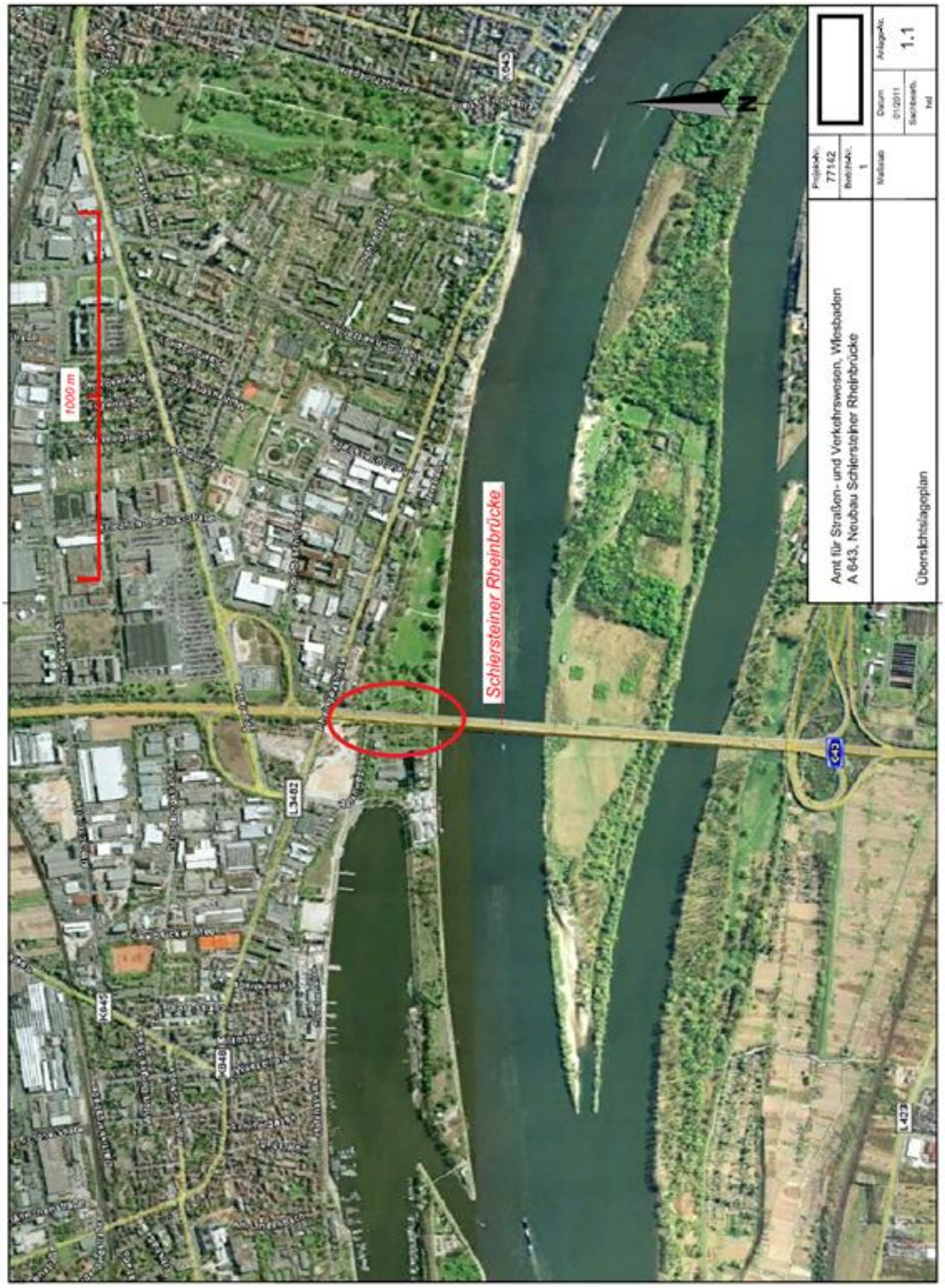


Fig. 36 Overview of the “Schiersteiner Rheinbrücke” from the geotechnical report

The project is a bridge construction in Germany which is located between Mainz and Wiesbaden. Parallel to the old bridge a new one will be constructed on the west side. The bridge is a fast connection between two cities and produces a higher traffic capacity. The red circle shows the area where the data was taken.

The geology in this area is mostly linked to the river Rhine, which is a big accumulator for fluviatil sediments shown in Fig. 37 in form of silts, sands and gravels.

At this construction site a lot of In-situ testing in form of CPT and DPH Tests has been done. This data are used and examined.

Schicht		Wichte		Scherfestigkeit			Steifemodul	Wasser-durchläs-sigkeit
		γ	γ'	φ'	c'	c_u	E_{se} / E_{sw}	k_f
		[kN/m³]	[kN/m³]	[°]	[kN/m²]	[kN/m²]	[MN/m²]	[m/s]
1	Auffüllungen: grob-/gemischtkörnig feinkörnig	 20 19	 10 9	 32,5 27,5	 0 2 ÷ 5	 0 20 ÷ 50	 -- --	 10^{-2} ÷ 10^{-6} 10^{-6} ÷ 10^{-8}
2a	Schluffe	19 ÷ 20	9 ÷ 10	25 ÷ 27,5	2 ÷ 5	20 ÷ 50	5÷10 / --	10^{-5} ÷ 10^{-9}
2b	Sande, locker bis mitteldicht	18 ÷ 19	9 ÷ 10	30 ÷ 32,5	0	0	20÷50 / --	10^{-4} ÷ 10^{-6}
3	Sande und Kiese, mitteldicht bis dicht	20 ÷ 21	11 ÷ 12	32,5 ÷ 37,5	0	0	50÷80 / --	2x10^{-3} ÷ 10^{-5}
4	Hydrobien-Schichten							
4a	Tone und Schluffe	19 ÷ 20	9 ÷ 10	17,5 ÷ 22,5	10 ÷ 30	75 ÷ 300	15÷25/30÷50	10^{-7} ÷ 10^{-9}
4b	Sande	18 ÷ 19	9 ÷ 10	27,5 ÷ 30	0	0	20÷50 / --	10^{-4} ÷ 10^{-7}
4c	Kalkstein, Algenkalk	22 ÷ 24	12 ÷ 14	30 ÷ 45 [1)]	--	2 ÷ 200 [2)]	50÷250 / --	10^{-1} ÷ 10^{-6}
5	Corbicula-Schichten	20	10	20	20	100 ÷ 200	15÷30/30÷60	10^{-7} ÷ 10^{-9}

[1)] Ersatzreibungswinkel   [2)] Gesteinsfestigkeit des Kalksteins in MN/m²

Fig. 37 Soil parameters of the geotechnical report

In the geotechnical report the stratigraphy is defined as shown below in English, Fig. 37:

1 Anthropogenic sediments: coarse/ mixed sizes of graines
fine grained

2a Silt

2b Sand: loose to middle dense

3 Sand and gravel: middle dense to dense

4 Hydrobien layers

4a Clay and Silt

4b Sand
4c Limestone
5 Corbicula layers

The thicknesses of the layers verify due to the geotechnical report. The other technical parameters should have the same signs like in a “Geotechnical report”. The data in the report shows no values for the constrained modulus in the anthropogenic sediments which is a necessary value for the settlement calculation of the auxiliary foundations. That is a major point why the geotechnical report cannot be the only basis for the design of the auxiliary support construction. So some data like the drillings are from the geotechnical report and the CPT data are taken by a company which was contracted by Max Bögl.

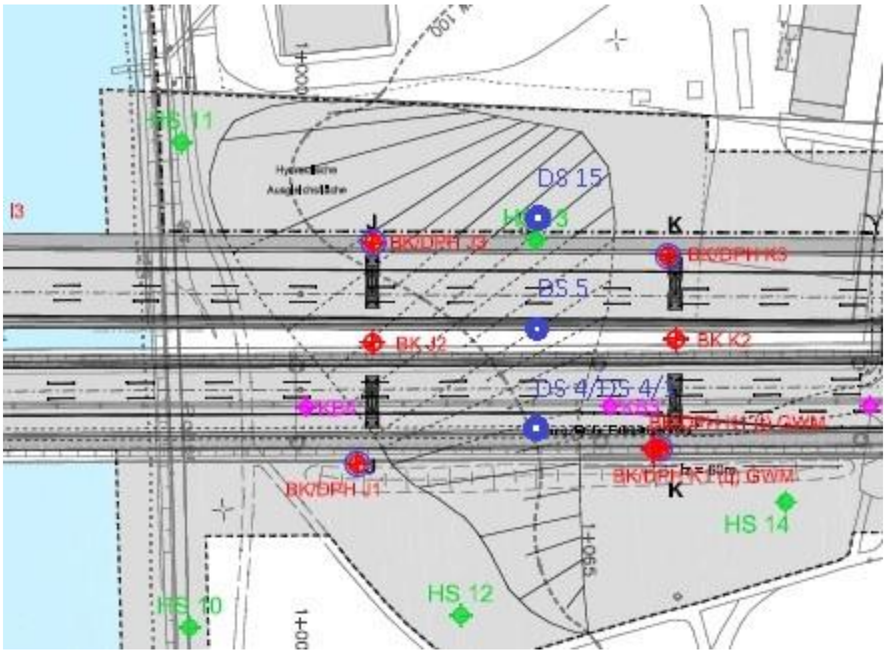


Fig. 38 Overview of the northern bridge part from the Axis J to K with the plotted data points, up is north, no scale because this is a copied detail map out of a larger one

Fig.38 shows the northern part of the bridge construction. This area is examined. DS is the German abbreviation for “Drucksondierung” and means CPT, BK means borehole and DPH is the Dynamic Probing Heavy. The drillings are set up along the J axis and the CPT points are in the middle between the axis J and K. In this area the first supporting structure is set up.

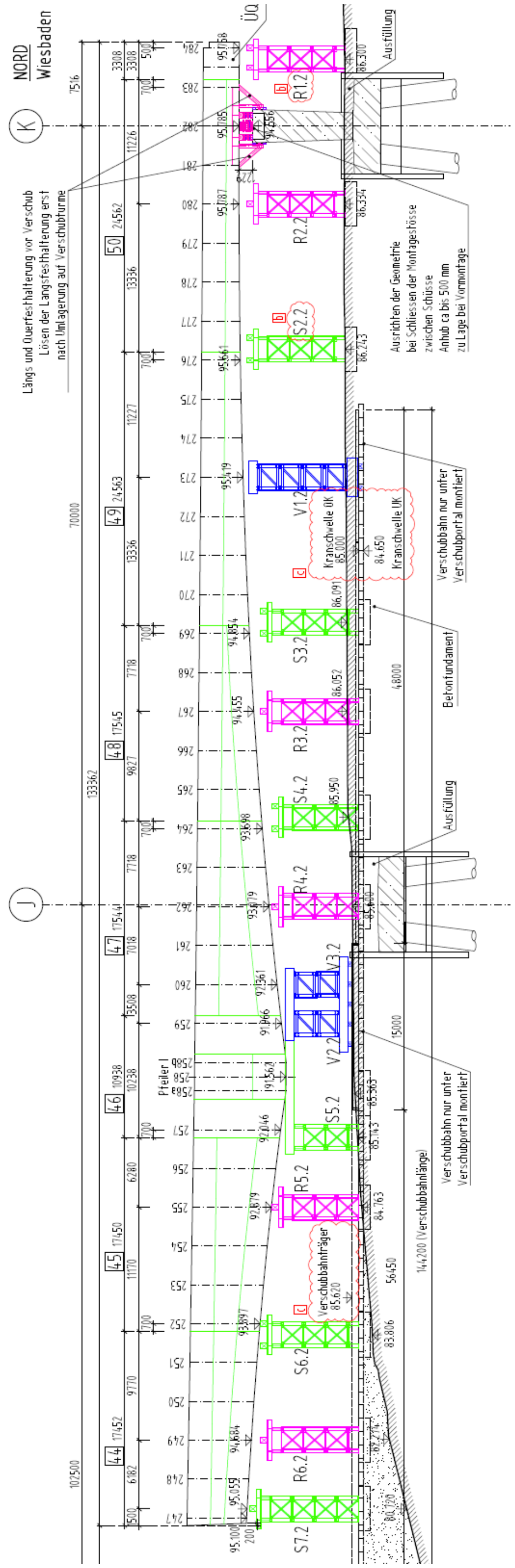


Fig. 39 Overview of the auxiliary foundation structure of the northern part of the Schierstein Bridge, no scale

In Fig.39 the auxiliary foundation structure between the axis J and K is shown. The distances between the objects are given in centimeters. The different auxiliary foundations are named with a letter and a number, for example next to the K axis R1.2 and R2.2. The total length of this supporting structure is 133.36 meters. It is an enormous steel construction which is lying on the supporting structure. In Fig. 40 an ending of the construction is shown.

The DPH data and the drilling BK J 1 and BK J 3 have been used by Max Bögl to evaluate the ground at the left side from the auxiliary structure S4.2. The CPT data was used to evaluate the area between S4.2 and the axis K.

Fig. 40 Construction of bridge with the supporting structure and the auxiliary foundations, middle part of the Schierstein bridge

8.2 Soil type classification charts application with a CPT profile

To show how the soil type classification charts work and how they can be used is shown in this chapter. Due to the small scale heterogeneity of the soil in this area, just plotted data points of different areas and not a whole profile is examined.

The soil type classification charts can be used to identify the soil type which is necessary for the determining ways of the constrained modulus via the constrained modulus chart and via the α-chart (Fig.25).

In Fig. 41 four colored lines are set. These are the points which are examined to test the soil type classification charts. The data of these lines are shown in Tab. 19

Point	q_c	f_s	R_f	colour	I_c-Index
1	5	10	2		3.8 to 4.0
2	7.5	15	2.5		4.0 to 4.9
3	1	1	2		10.5 to 5.2
4	7.5	2.5	1.5		5.2 to 5.6

Tab. 19 Data of the 4 Points which are marked in Fig. 41, q_c and f_s in [MPa]

This data are now plotted in the different soil type classification charts which are shown on the following pages. There are just the soil type charts of the *DIN 4094* used as well as the ones' of the *Fugro Company* and the CPT data, which already has the classification of the soil behavior type index value.

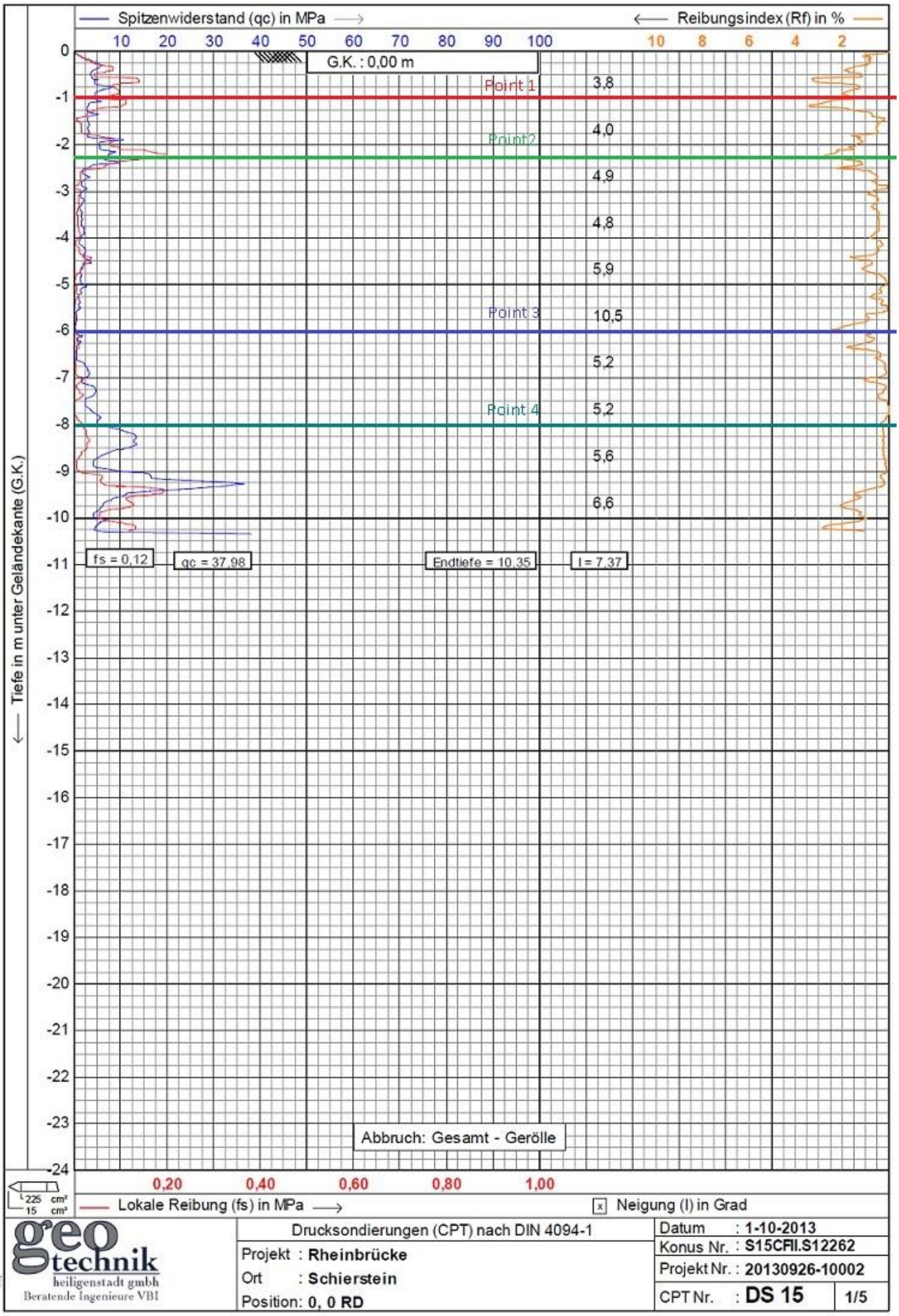


Fig. 41 CPT profile of DS 15 with 4 colored lines which provide the data for the soil type classification charts test and are interpreted and described in the next chapter

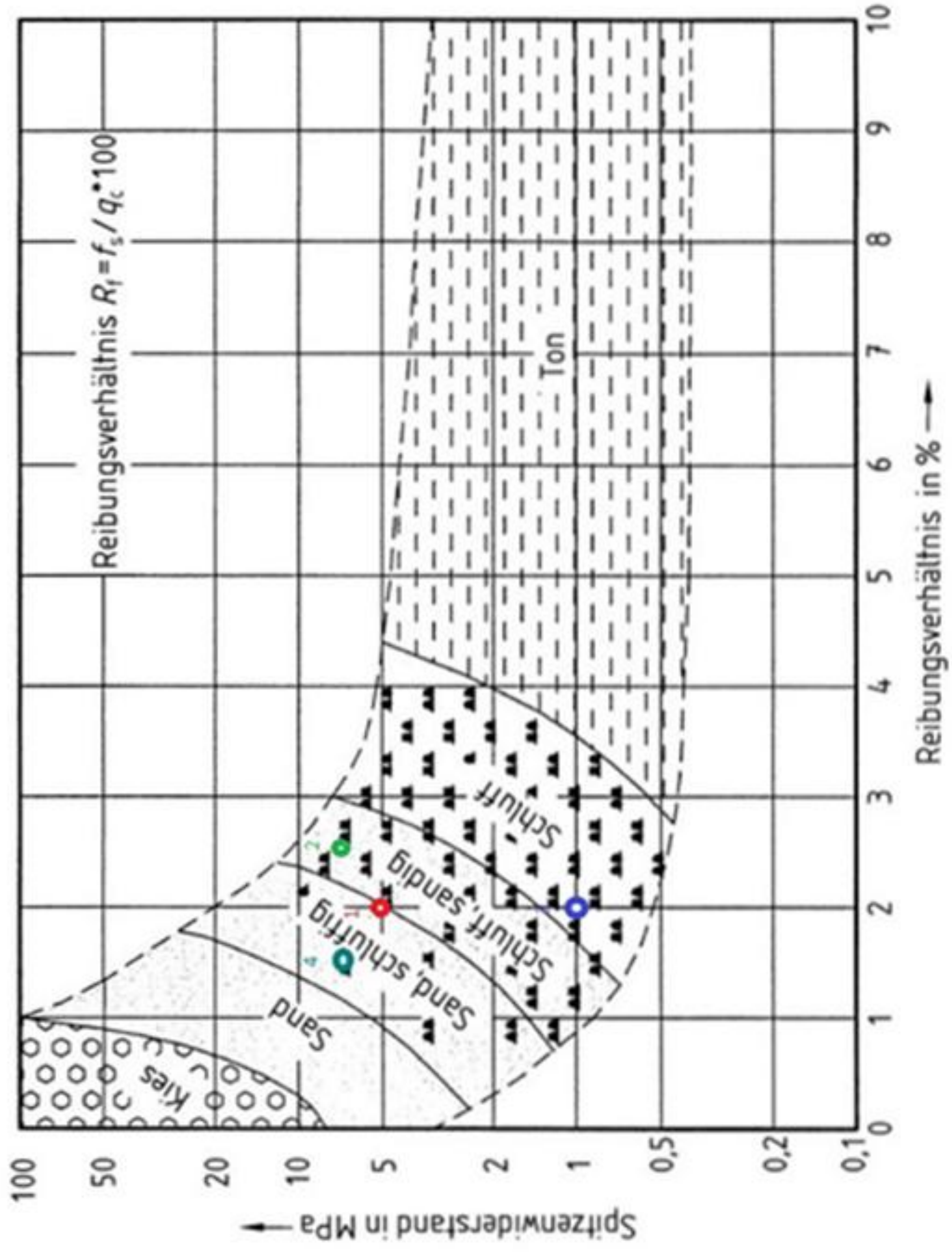


Fig. 42 Soil type classification chart after *DIN 4094* with the plotted example points of Tab.19

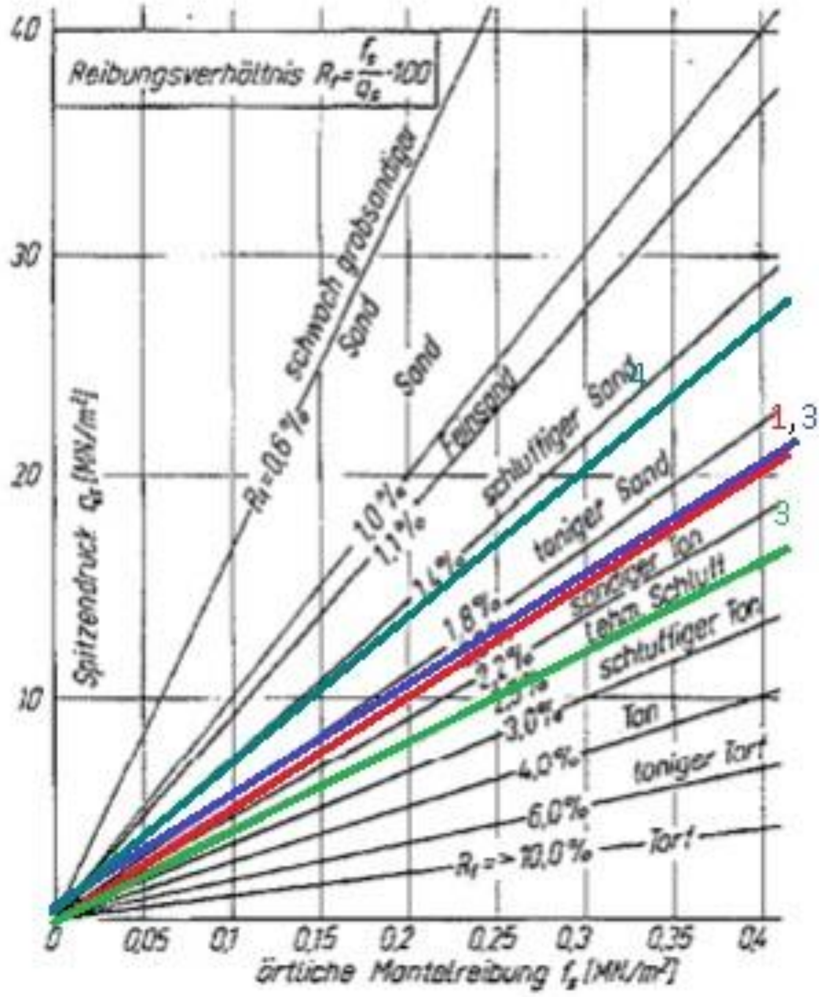


Fig. 43 *Fugro* soil type classification chart with the plotted example points of Tab.19

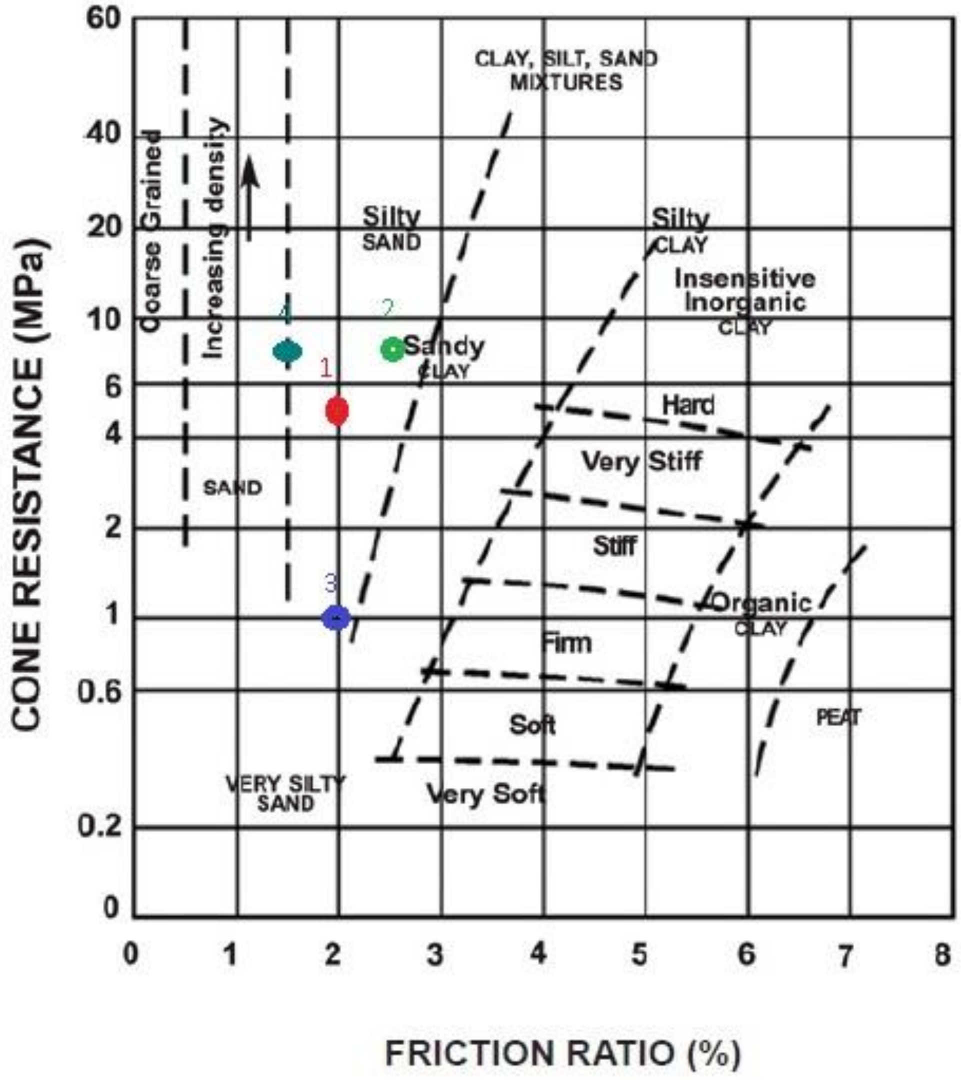


Fig. 44 Newer soil type classification chart of the *Fugro Company* with the plotted example points of Tab.19

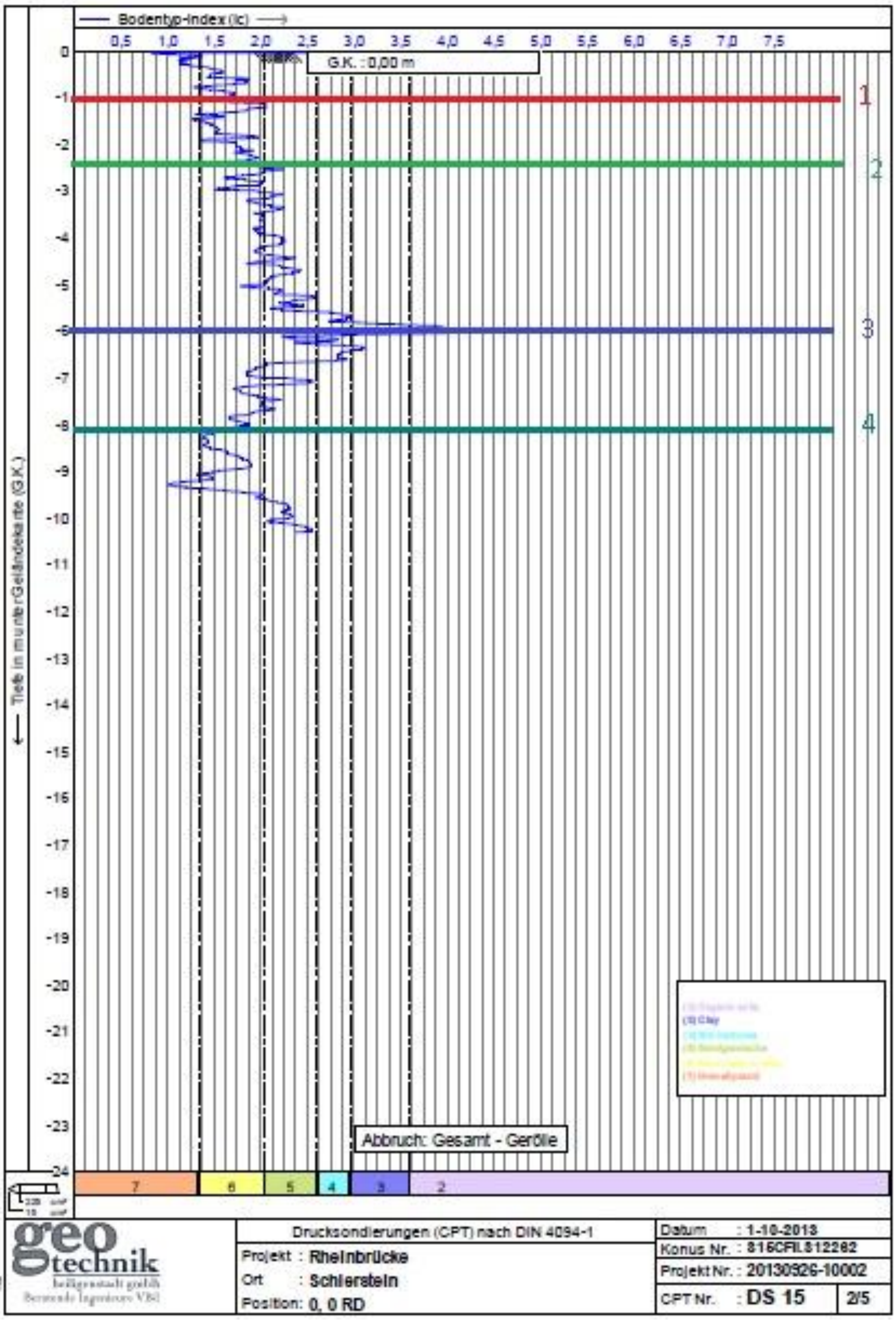


Fig. 45 Soil type classification with soil type behavior index (*Robertson & Wride, 1998),* with the plotted example points of Tab. 19

The plots show different results which are displayed in Tab. 20. The tendency of the results is pointing in the same direction. However, these results leave some room for interpretation and the constrained modulus is estimated via Tab. 17.

Point	q_c	Compactness / Consistency	DIN 4094 chart	Fugro old	Fugro new	Soil behavior type I_c
1	5	loose / hard	Sand, silty	sandy Clay	silty Sand	Sand clean to silty
2	7.5	middle dense / hard	Silt, sandy	Loam, Silt	silty Sand	Sand clean to silty
3	1	very loose / soft	Silt	sandy Clay	silty Sand	Clay
4	7.5	middles dense	Sand, silty	silty Sand	Sand, silty Sand	Sand clean to silty

Point		Es chart values	E_s min	E_s max	E_s min	E_s max	E_s min	E_s max	E_s min	E_s max
1			9.5	29	12	20	9.5	29	9.5	29
2			15	30	15	30	29	48	29	48
3			3	9	2	5	9.5	29	0.4	4
4			29	48	29	48	29	48	29	48

Tab. 20 Results of the soil classifications for the four example Points with the estimated E_s-values after Tab. 17, E_s in [MN/m²]

The compactness or the consistency of the soil was evaluated by using the cone tip resistance q_c. Point 1 with a q_c value of 5 [MPa] is evaluated as loose for a non-cohesive soil and as hard for a cohesive soil. Point 2 has a q_c value of 7.5 [MPa] which is already a middle dense compactness and for a cohesive soil the consistency would be hard according to *Prinz & Strauß, (2002)*.

A q_c value of 1[MPa] like Point 3 shows, means a soft consistency or a very loose compactness for a non cohesive soil. Point 4 has a value of 7.5 [MPa] and is interpreted in compactness and consistency like Point 2.

The estimation of the E_s-values are interpreted as follows. If the result was named, for example silty sand it would be interpreted as a clean sand. Which can already lead to an error, but there are no literature values for soil mixture states. So the result still has the magnitude for sand which is well visible in Tab. 17. A more specified interpretation is not possible due to the fact that for the mixture states of such soils no literature values are available. In Tab. 20 it is visible that the variability of the constrained modulus is quite high. The magnitude for Point 1 is 9.5 to 29 [MN/m²], 15 to 48 [MN/m²] for Point 2, 0.4 to 29 [MN/m²] for Point 3 and 29 to 48 [MN/m²] for Point 4.

The soil type classification charts give an overview which kind of material is tested, even if silty sand or just clean sand is tested. By the usage of different charts one can be nearly sure to be in the right stratigraphic unit. When it comes to interpreting results where organic soils can possibly occur, the soil type classification chart of *Robertson et al., (1995)* with the seismic value G_0 *(Mlynarek et al, 2010)* should be used. The usage is limited for estimation due to the fact that the result still has a wide range. The problem is to interpret the constrained modulus from these results, because just from the soil type charts results it is not possible to give an accurate value, just a magnitude.

8.3 Constrained modulus from CPT data

For this application the data of the CPT profile DS 5 and DS 15 is chosen to estimate the constrained modulus. The soil classification after *Robertson, (1990),* the Soil behavior type index I_c and the interpretation of the editor is used. These two profiles are located between the axis J and K (Fig.38). The soil type classification according to *Robertson, (1990)* is used to interpret the stratigraphy of the CPT profiles. The classification is very precise and indicates layers on a very small scale. The scale is too precise for an interpretation. Therefore the soil type behavior index, the cone tip resistance curve and the sleeve friction curve for the interpretation of these profiles are used as well. Afterwards an idealized profile out of these data with the foundation parameters is built up. Then is the constrained modulus estimated over all three calculation ways for CPT data, which is the important parameter for the settlement analysis.

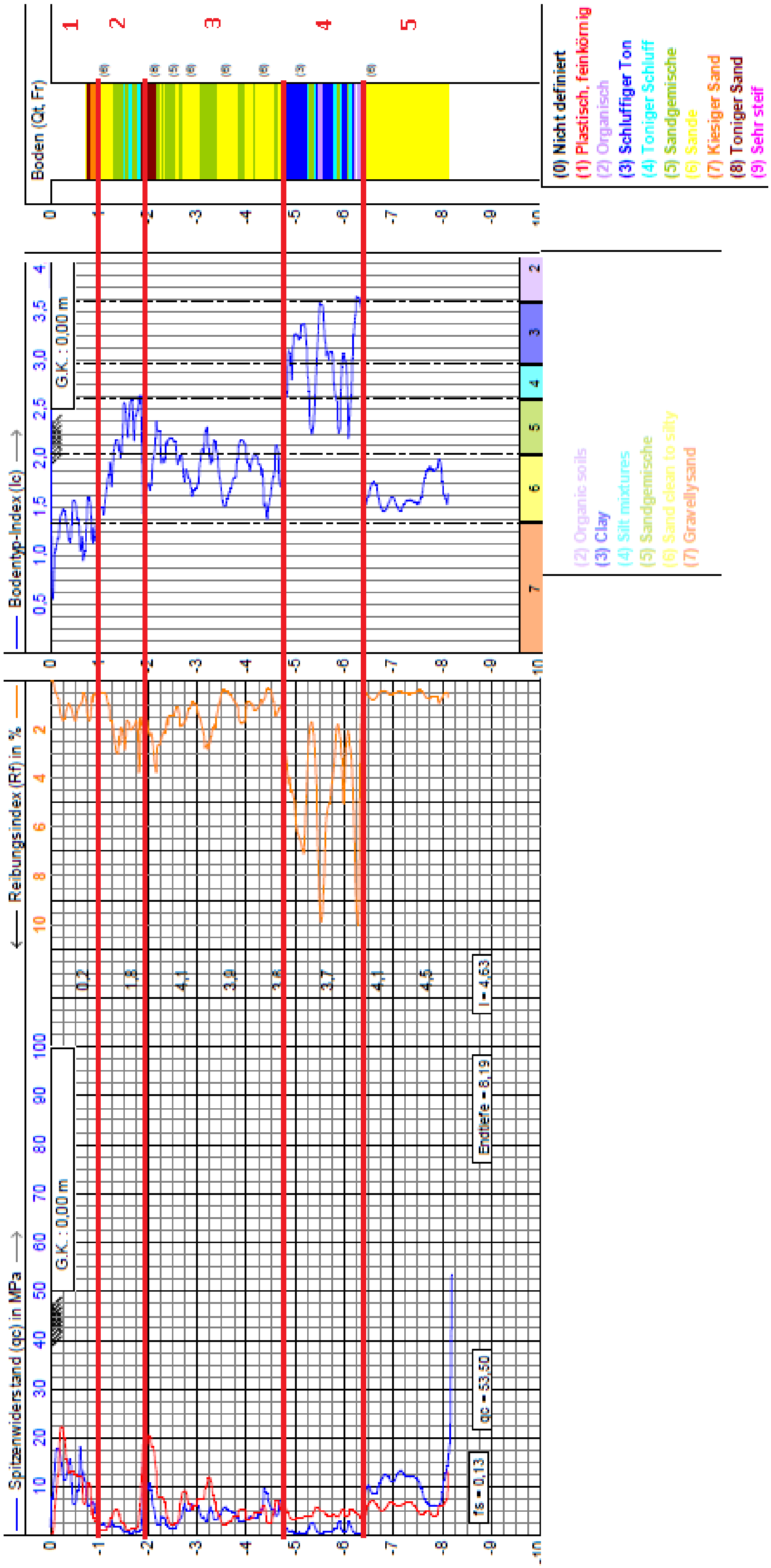


Fig. 46 CPT profile DS 5 and interpretation, the red lines symbolize the stratigrahic boarders

The CPT profile DS5 shows in the depth from 0 m to 1 m a cone tip resistance (blue) of higher values than 10 [MPa] and a sleeve friction (red) with nearly the same tendencies occurs which leads to a relatively low friction ratio. This indicates a middle dense gravelly sand.

From 1m to 2 m the cone tip resistance is low, with values between 1-2.5 [MPa]. The f_s value is a bit higher in average than the q_c value, considering the R_f value, leads to a material classification which indicates a sand mixture with a loose density.

Layer 3 from 2m to 4.8 meters depth is sand with a loose compactness. The cone tip resistance is in average about 4 [MPa]. The f_s value is a bit higher which leads to a relatively low friction ratio.

Between 4.8 m and 6.30 m the cone tip resistance is really low with an average value of 1 [MPa] and the f_s value is in average 4 [MPa]. The friction ratio is high and indicates a soil which is a mixture between clay and silt and has a soft consistency.

From 6.30 m to 8 m the f_s value is low and the q_c value is in average in the area about 10 [MPa], these values lead to a classification of a sand with a middle dense compaction.

Depth	Soil	qc average	Compactness / Consistency	E_s values via E_s chart		E_s via α-chart			
[m]		[MPa]		E_s min [MN/m²]	E_s max [MN/m²]	α min	α max	$E_{s\,\alpha}$ min [MN/m²]	$E_{s\,\alpha}$ max [MN/m²]
1,00	sand, grave	10	middle dense	80,0	100,0	3,5	5,0	35,0	50,0
2,00	sand mixtur	1	loose	9,5	29,0	2,0	5,0	2,0	5,0
4,80	sand	4	loose	9,5	29,0	2,0	5,0	8,0	20,0
6,30	clay,silty	1	soft	0,4	4,0	4,0	7,0	4,0	7,0
8,00	sand	10	middle dense	29,0	48,0	2,0	5,0	20,0	50,0

Tab. 21 Result of the CPT DS 5, the E_s-values with the minima and maxima via the E_s-chart and via the α chart

The constrained modulus results of the CPT DS 5 are calculated by using the E_s-chart and the average constrained modulus with the α-chart. With the average cone tip resistance q_c it is possible to estimate the density of the soil, which makes the result more precise, but the literature values for the way via the E_s-chart shows still an enormous range, visible at Tab. 17.

The values which are calculated with the regional factor α have a bit smaller range in average, especially when the cone resistance is low. This depends on the calculation formula.

$$E_s = \alpha * q_c$$

The maximum α range is 2 to 5, so the result of the corresponding constrained modulus depends on the multiplication with the cone tip resistance and their range is bigger. Due to that fact it should be easier to calculate a more precise range for lower cone tip resistance values; this depends indeed on the correctness of the regional factor α. Which leads to the next question; how α is estimated and is it possible to define this factor more precisely.

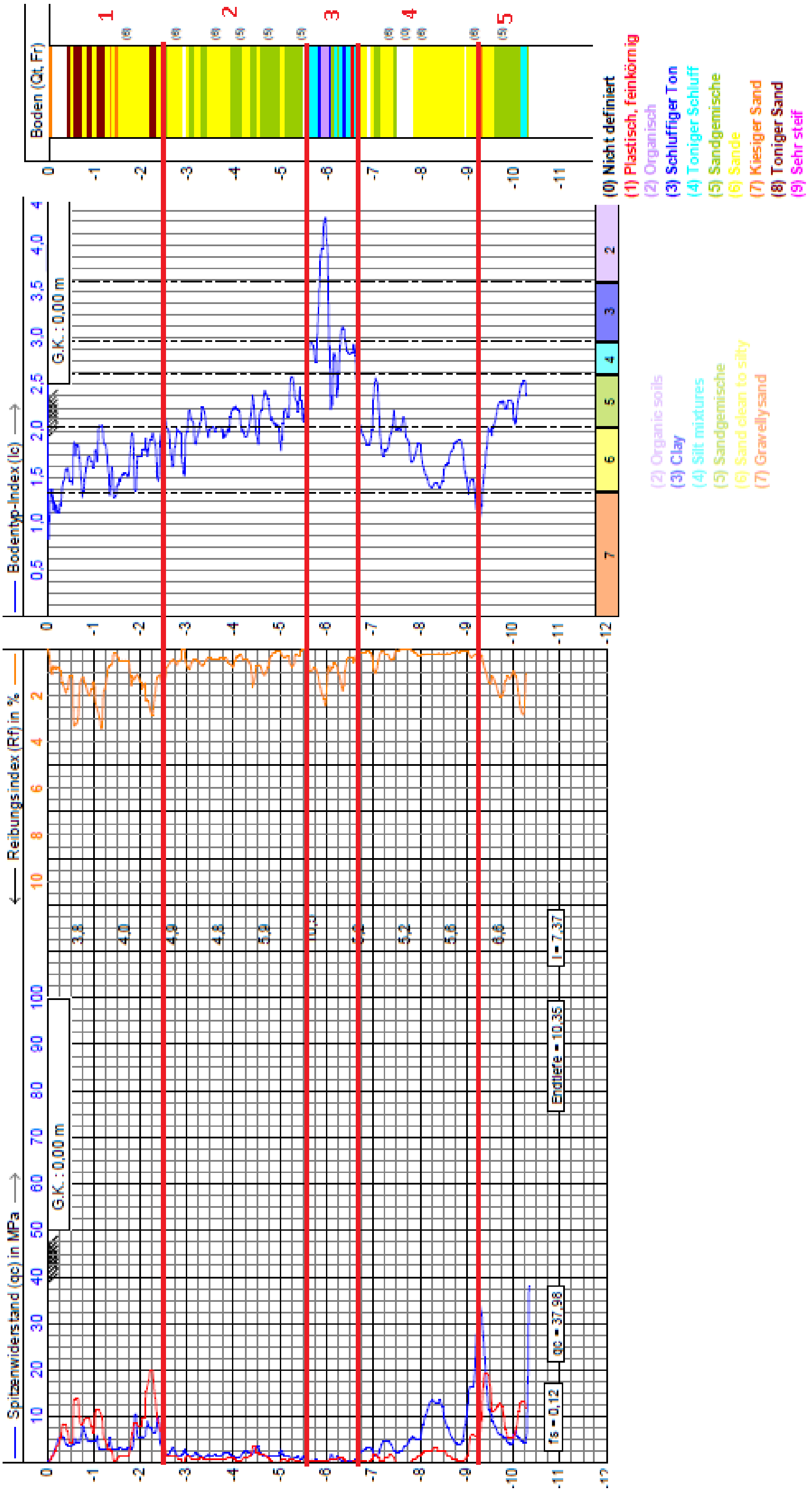


Fig. 47 CPT profile DS 15 and interpretation, the red lines symbolize the stratigrahic borders

From 0 m to 2.5 m the tip cone resistance q_c is often changing but always under 10 [MPa] which indicates a loose compactness. Due to that the "behavior" of the sleeve friction f_s is nearly the same compared to the q_c, but always a bit higher, which leads to an oscillation of the friction ratio R_f in the area about 2 to 4 and indicates a loose soil.

In the depth form 2.5 m to 5.5 m the cone tip resistance q_c and the sleeve friction f_s is low and always around 2 [MPa], which indicates a very loose or soft area. The friction ratio is low as well which can indicate loose clean sand.

The area between 5.5m and 6.75 meters has an average cone tip resistance of 1 [MPa]. The soil behavior type index and the soil type classification according to *Robertson, (1990)* indicates silty to clayey material in this area, which is soft due to the fact that the q_c is about 1 [MPa].

In the depth of 6.75 m to 9.25 m the cone tip resistance is rising again to an average value of 7 [MPa] which indicates sand with a middle dense compaction. The profile in figure 47 shows in this area a quite high cone resistance with a much lower sleeve friction which leads to a low friction ratio.

In the last examined depth between 9.25 m to 10 m a change is seen so that the sleeve friction is getting higher again. This indicates a soil mixture of sand which has a middles dense compactness.

Depth	Soil	qc average	Compactness / Consistency	E_s values via E_s chart		E_s via α-chart			
[m]		[MPa]		E_s min [MN/m²]	E_s max [MN/m²]	α min	α max	$E_{s\,\alpha}$ min [MN/m²]	$E_{s\,\alpha}$ max [MN/m²]
2,50	sand,silty	5,0	loose	9,5	29,0	2,0	3,5	10,0	17,5
5,50	sand mixture	2,0	loose	9,5	29,0	2,0	5,0	4,0	10,0
6,75	Silt,clayey	1,0	soft	3,0	6,0	4,0	5,0	4,0	5,0
9,25	sand	7,0	middle dense	29,0	48,0	2,0	5,0	14,0	35,0
10,00	sand mixture	10,0	middle dense	29,0	48,0	2,0	5,0	20,0	50,0

Tab. 22 Result of the CPT DS 15, the E_s-values with the minima and maxima via the E_s-chart and via the α chart for a low cone tip resistance

Tab.22 shows the same structure like Tab. 21, wide ranges for the E_S-chart values and smaller ranges for the α values, especially when the cone tip resistance is low.

Idealized CPT data

For the usage of these CPT profiles the ground is now idealized by the editor which is an interpretation of the data. Additionally included is the foundation and an additive soil as well as the mechanical compaction layer. Fig. 48 shows how the additives were built in before the construction started. To build up an idealized profile follows the work steps of the Max Bögl company

Fig. 48 Shows the surface before the Ryolith was added.

The surface was steam rolled after the soil was excavated by an excavator, to build up the basis for the additive soil. The first dumps of Ryolith are steam rolled into the surface to establish a basis for the additive soil layer above.

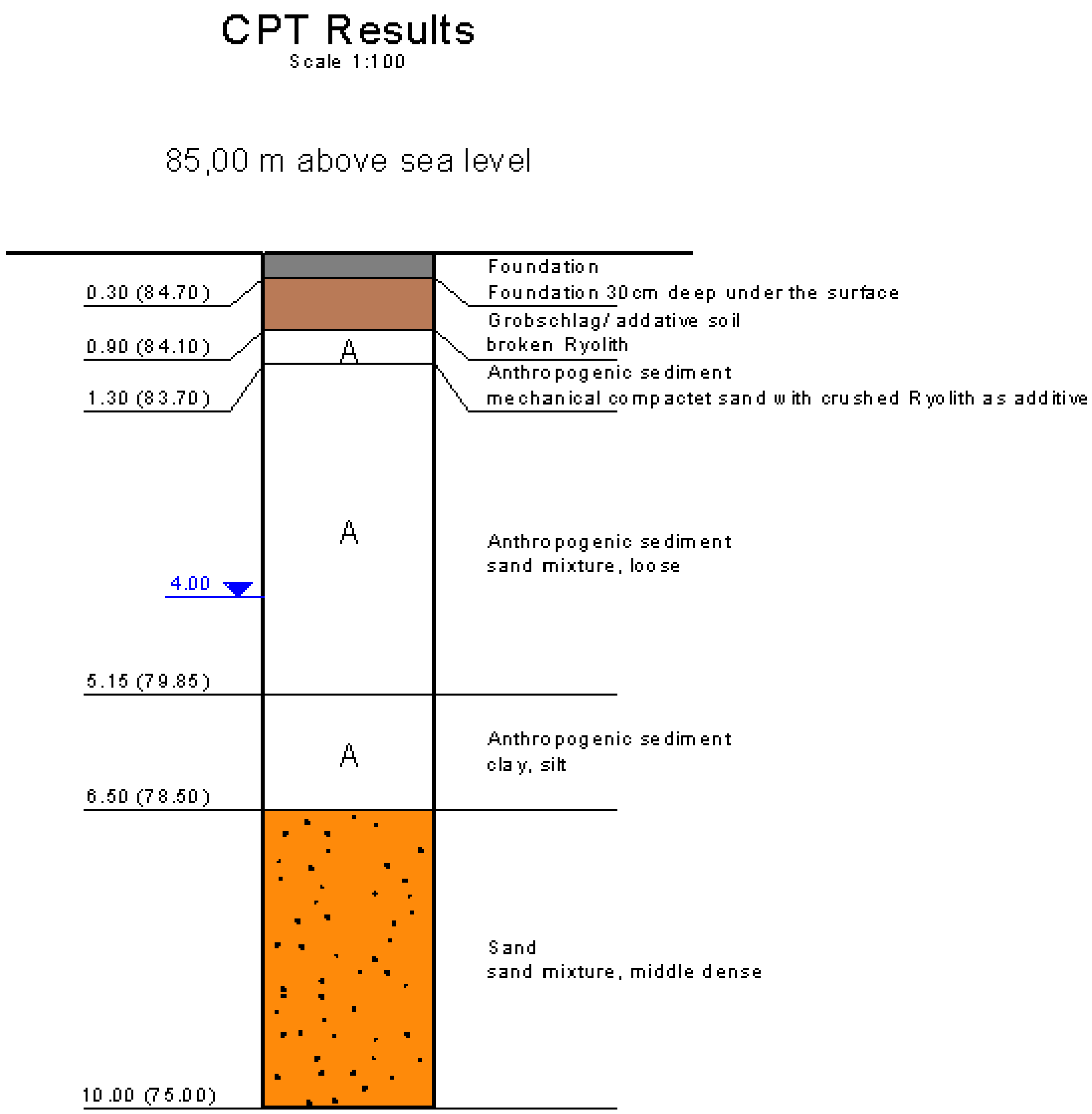


Fig. 49 Idealized subsurface from CPT data

Fig. 49 shows an idealized subsurface interpretation of the CPT Profiles DS 5 and DS 15, the stratigraphic layers are identified on both profiles but the depth is not completely the same, so the average between the two CPT profiles has been taken. For the clay layer this assumption was necessary, because of the different depth of the clay in DS 5 and DS 15.

From 0m to 0.3 meters the foundation is located. Below the foundation the additive soil is located up to a depth of 0.9 meters which consists of crushed Ryolith. From 0.9m to 1.3 meters there is sand which is mechanical compacted with a steam roller. On top of this layer Ryolith was put in as well and afterwards steam rolled. From 1.3m to 5.15 meters there is sand with a loose compactness and at 4 meters below the surface is the groundwater. In the depth from 5.15m to 6.5 meters a mixture between silt and clay with soft consistency is located. Below 6.5 meters there is sand which is middle dense and partly dense.

Depth	Soil	qc a.	density	γ	γ'	$\sigma_ü$	Z	Z/b	a/b	i	σ_z	v_{min}	v_{max}	w
[m]		[MPa]		[kN/m	[kN/m	[kN/m²		b=2,5m	10m/2,5m					
0,30	Foundation													
0,90	additive soil			21,00	11,00	12,60	0,60	0,24	4,00	0,78	195,00			
1,30	sand (m. compacted)		dense	20,00	11,00	20,60	0,90	0,36	4,00	0,70	175,00			
5,15	sand mixture	1 to 4	loose	20,00	10,00	86,10	4,85	1,94	4,00	0,23	57,50	50,00	213,54	0,6/0,5
6,50	clay	1,00	soft	19,00	9,00	98,25	6,20	2,48	4,00	0,15	37,50	50,00	50,00	0,60
10,00	sand mixture	7 to 10	middle dense	20,00	10,00	133,25	9,70	3,88	4,00	0,08	20,00	243,13	450,00	0,50

Depth	E_s via E_s chart [MN/m²]		E_s via α table [MN/m²]				E_s stress d. [MN/m²]		average E_s	Interpretated E_s
[m]	E_s min	E_s max	α min	$E_{s\,\alpha}$ min	α max	$E_{s\,\alpha}$ max	E_s stress min	E_s stress max		
0,30										
0,90										100,0
1,30										40,0
5,15	9,5	29,0	2	2,0	5	20,0	5,4	22,9	14,8	15,0
6,50	0,4	4,0	4	4,0	7	7,0	5,5	5,5	4,4	5,0
10,00	29,0	48,0	2	14,0	5	50,0	29,1	53,9	37,3	40,0

Tab. 23 Result of the CPT data analysis, the green values are given by Max Bögl

Tab. 23 shows all three determining ways on the example of the idealized profile. The technical parameters are used from the CPT data and from the geotechnical report.

In a next step it was necessary to calculate the effective stress. The constrained modulus has been calculated with the tables of Kany and the consistency via the stress dependent constrained modulus according to *DIN 4094*

The constrained modulus results of the idealized profile of each layer are evaluated by the interpretation of the editor. The constrained modulus values of the editor fit quite well to the average values of all different calculation methods. This profile data of the idealized profile will be implemented in a settlement analysis for an auxiliary foundation in a later chapter. Afterwards it will be compared with the occurred settlement data.

Difficulties have occurred while determining the stress dependent constrained modulus for the sand mixture layer. The sand mixture is not defined according to *DIN 4094*, because of the q_c value which is under 5 [MPa] and so for sand not defined. Nevertheless to calculate the stress dependent constrained modulus is determined the minimum with the formula for clay or a low plastic medium material, because for these materials are the [MPa] range defined. For the maximum is taken the standard formula for sand, knowing that this area is not defined according to *DIN 4094*. This leads to wide range for the stress dependent constrained modulus for the sand mixture.

8.4 DPH application in combination with borehole data

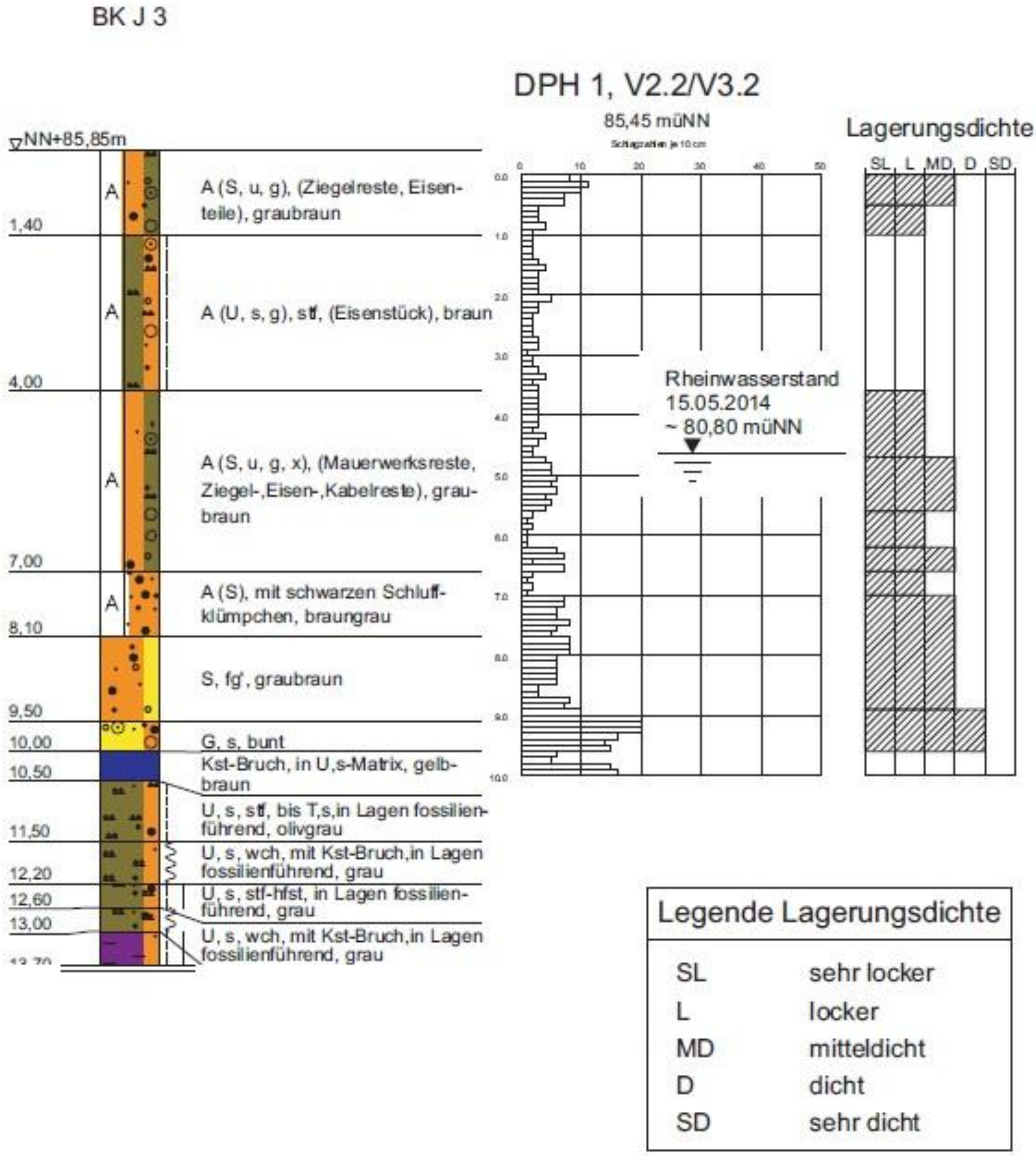


Fig. 50 Borehole data of BK J3 and the related DPH data of DPH 1,V2.2/V3.2 and the converted density.

The borehole data from BK J3 shows the heterogeneity of the soil. Until 8.10 m there is “Auffüllung” which is the German word for anthropogenic sediments and has the abbreviation A. These sediments are a mixture between silt and sand with a poor amount of gravel. Due to that, it includes some waste like parts of bricks or metal or plastic pieces.

In combination with DHP profile it is possible to characterize the density or the consistency of the soil. Care should be taken in areas with groundwater. Therefore the *DIN 4094* provides a correction.

From 0m to 1.4 m the subsoil consists of a silty, gravelly sand. It includes parts of broken bricks and metal. 1.4m to 4 meters is silt with soft consistency. It includes as well sandy and gravelly material. In the depth 4m to 7 meters it is sand, silty, gravelly and boulderness. The density is loose to middle dense and it includes some waste like brick parts, metal parts and wire parts. From 7m to 8.1meters there is sand which has small lumps of black silts inside which is probably some waste material.

From 8.10m to 9.5 meters start the fluviatil sediments which have a middle dense compaction. It is sand with a small amount of fine gravel and from 9.5m to 10m it is a dense sandy gravel.

Tab. 24 shows the results and the matching E_s values via Tab. 17 for this material with their compactness or consistency.

Depth	Soil	Compactness / Consistency	E_s value after E_s table	
[m]			E_s min	E_s max
0				
1.40	sand, silty, gravelly	loose to middle dense	9.5	48
4.00	silt, sandy, gravelly	soft	6	10
7.00	sand, silty, gravelly, blouderness	loose to middle dense	9.5	48
8.10	sand	middle dense	29	48
9.50	sand, poor fine gravelly	middle dense	29	48
10.00	gravel, sandy	dense	100	200

Tab. 24 Constrained modulus results of DPH 1, V2.2/V3.2

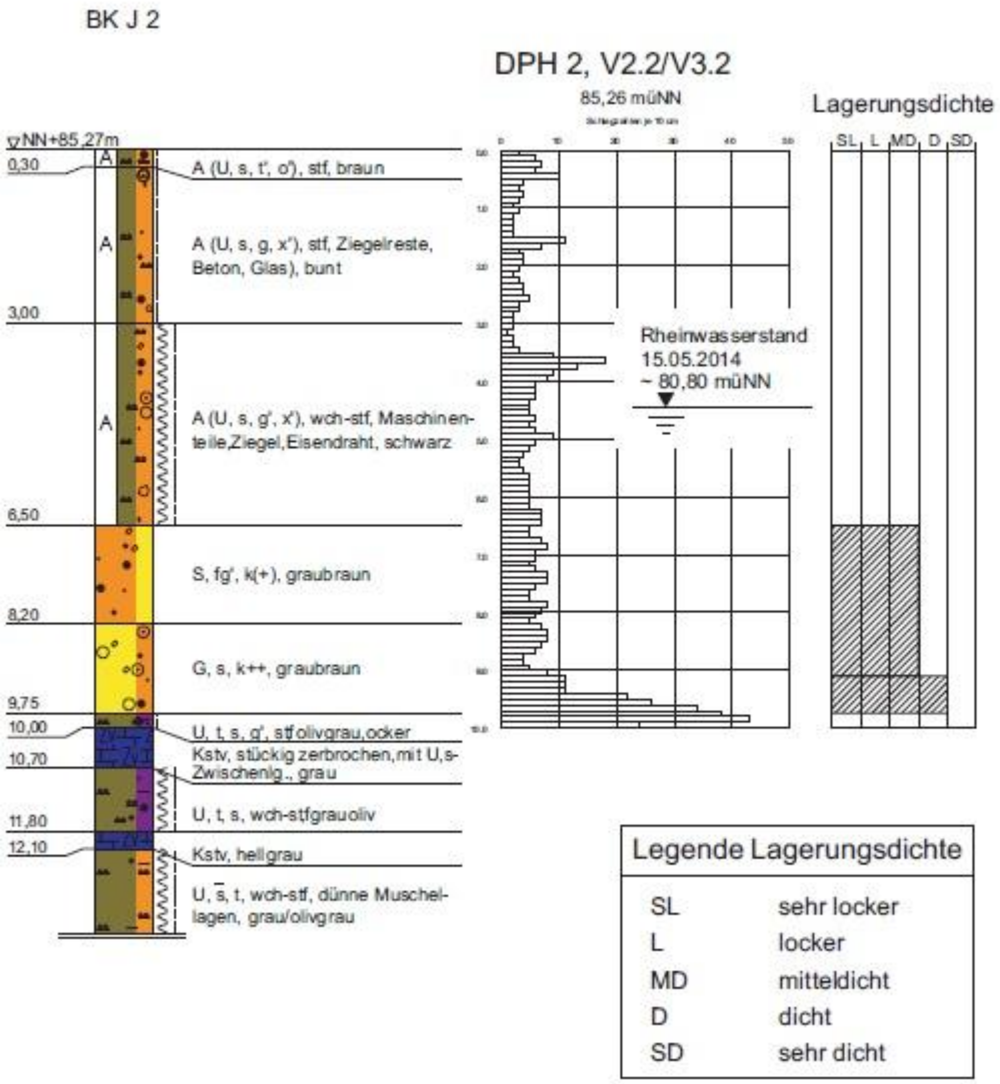


Fig. 51 Data of the borehole BK J2 and DPH 2, V2.2/V3.2

The profile of BK J2 shows the same tendency until 6.5 m where anthropogenic sediments are located. From 0 m to 0.3 m it is a cohesive (silt, sandy and poor clayey) soil which has a stiff consistency. From 0.3 to 3 meters it is soft silt, sandy, gravelly and poor boulderness, additionally some brick material was found. From 3 to 6 meters the soil is a sandy, poor gravelly and poor boulderness silt. In this area bricks, metal parts and wires were also found.

The fluvatile sediment starts in a depth of 6.5 m. From 6.5m to 8.2 meters sand with a small amount of fine grained gravel and a middle dense compactness was detected. From 8.2 m to 9.75 m the soil is sandy gravel with a middle dense compaction. The last part which is tested by the DPH is from 9.75 m to 10 m and it is silt which is clayey, sandy and poor gravely.

The result of the interpretation is shown in Tab.25

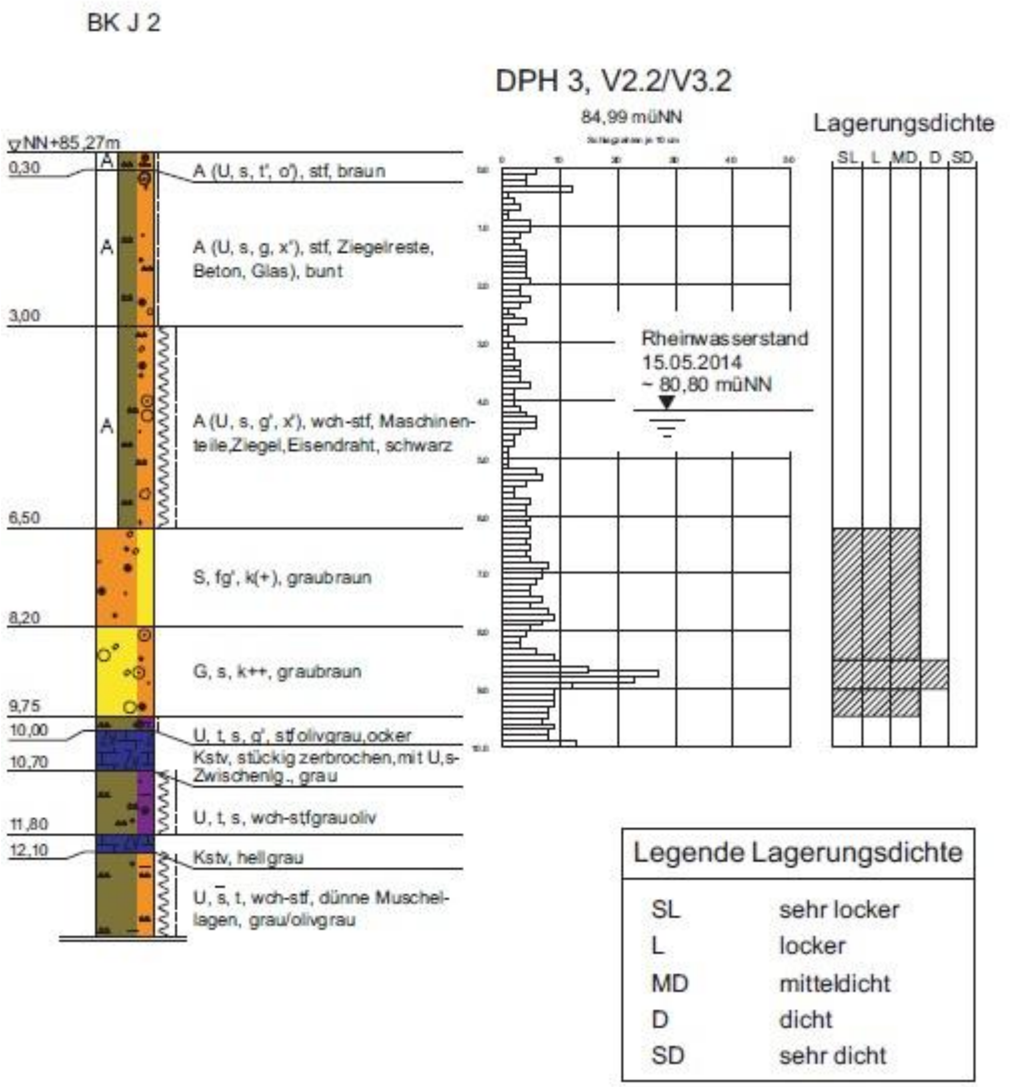


Fig. 52 Borehole data of BK J2 and the related DPH data of DPH 3, V2.2/V3.2 and the converted density

The borehole data is from BK J2 which was described before. But here is the profile of BK J2 combined with the DPH 3, so it is possible to evaluate the compactness respectively the consistency again.

DPH 2

Depth [m]	Soil	N10 average blows	Compactness/ Consistency	E_s after E_s table [MN/m²] E_s min	E_s max
0,00					
0,30	silt, sandy, poor clayey and organicly	7	stiff	6,0	10,0
3,00	silt, sandy, gravelly, poor boulderness	4	soft	3,0	6,0
6,50	silt, sandy, poor gravelly, poor boulderness	6	stiff	6,0	10,0
8,20	sand, poor fine gravelly	7	middle dense	29,0	48,0
9,75	gravel,sandy	8	middle dense	80,0	100,0
10,00	Silt, sandy,clayey, poor gravely	2	hard	15,0	30,0

DPH 3

Depth [m]	Soil	N10 average blows	Compactness/ Consistency
0,00			
0,30	silt, sandy, poor clayey and organicly	5	stiff
3,00	silt, sandy, gravelly, poor boulderness	4	soft
6,50	silt, sandy, poor gravelly, poor boulderness	4	soft
8,20	sand, poor fine gravelly	7	middle dense
9,75	gravel,sandy	10	middle dense
10,00	Silt, sandy,clayey, poor gravely	7	stiff

Tab. 25 Constrained modulus results of DPH 2 and DPH 3, V2.2/V3.2

The two different results of the DPH 2 and DPH 3 show nearly the same structure. The blow counts varies a in a small range. But in general show both DPH’s the same result, which varies between soft and stiff, in the depth of 6.5 m

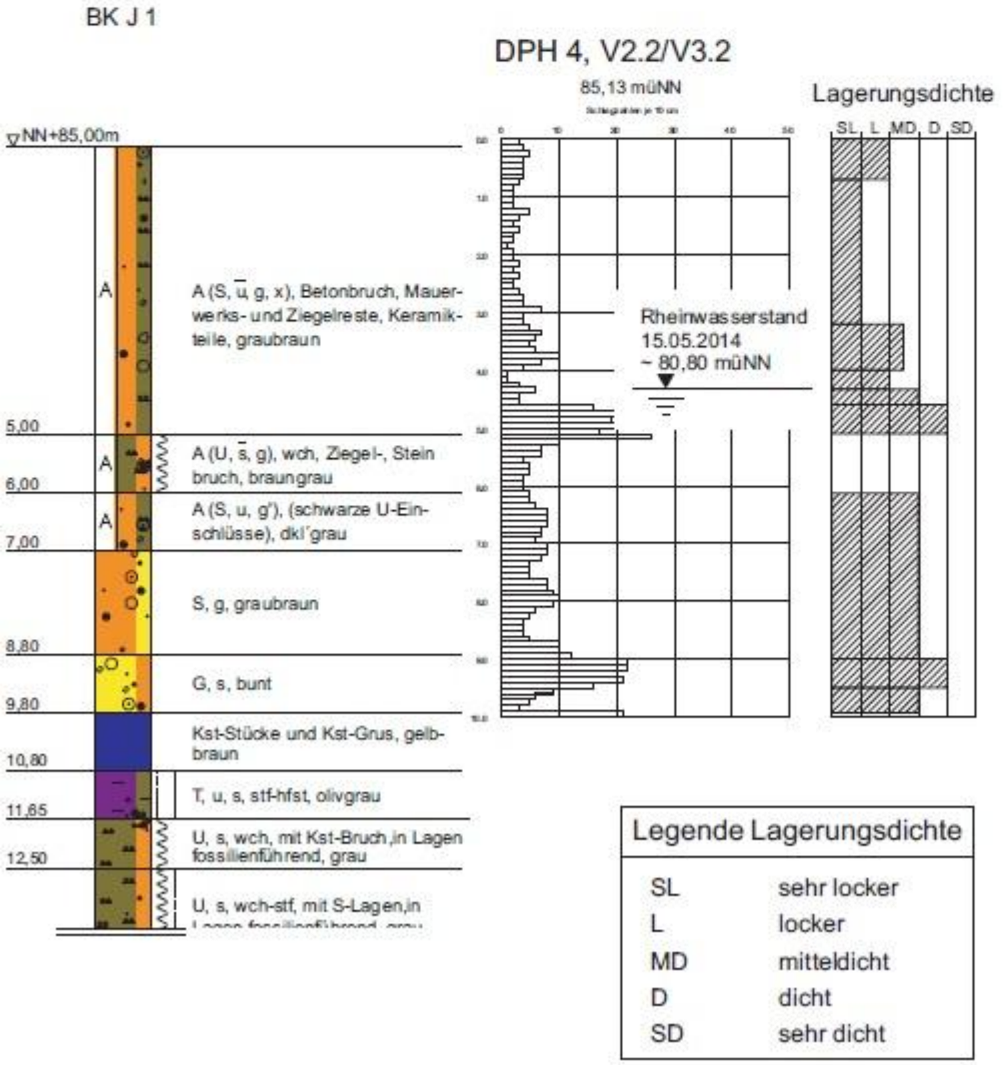


Fig. 53 Borehole data of BK J1 and the related DPH data of DPH 4, V2.2/V3.2 and the converted density

From 0 m to 5 m the borehole data of BK J1 show a loose anthropogenic sediment which is sand, strong silty, gravelly, and boulderness. It includes parts of concrete, brick and ceramic additionally. In the depth of 5 m to 6 m there is a soft silt which is strong sandy and gravelly. Brick and concrete parts are found as well. From 6 m to 7 m sand is located which is silty and poor gravelly. The sand has a middle dense compactness.

From a depth of 7m the fluviatil sediments start in hind of a gravelly sand with a middle density, down to the depth of 8.8 meters. From 8.8 m to 9.8 m is there a sandy gravel with a dense compactness.

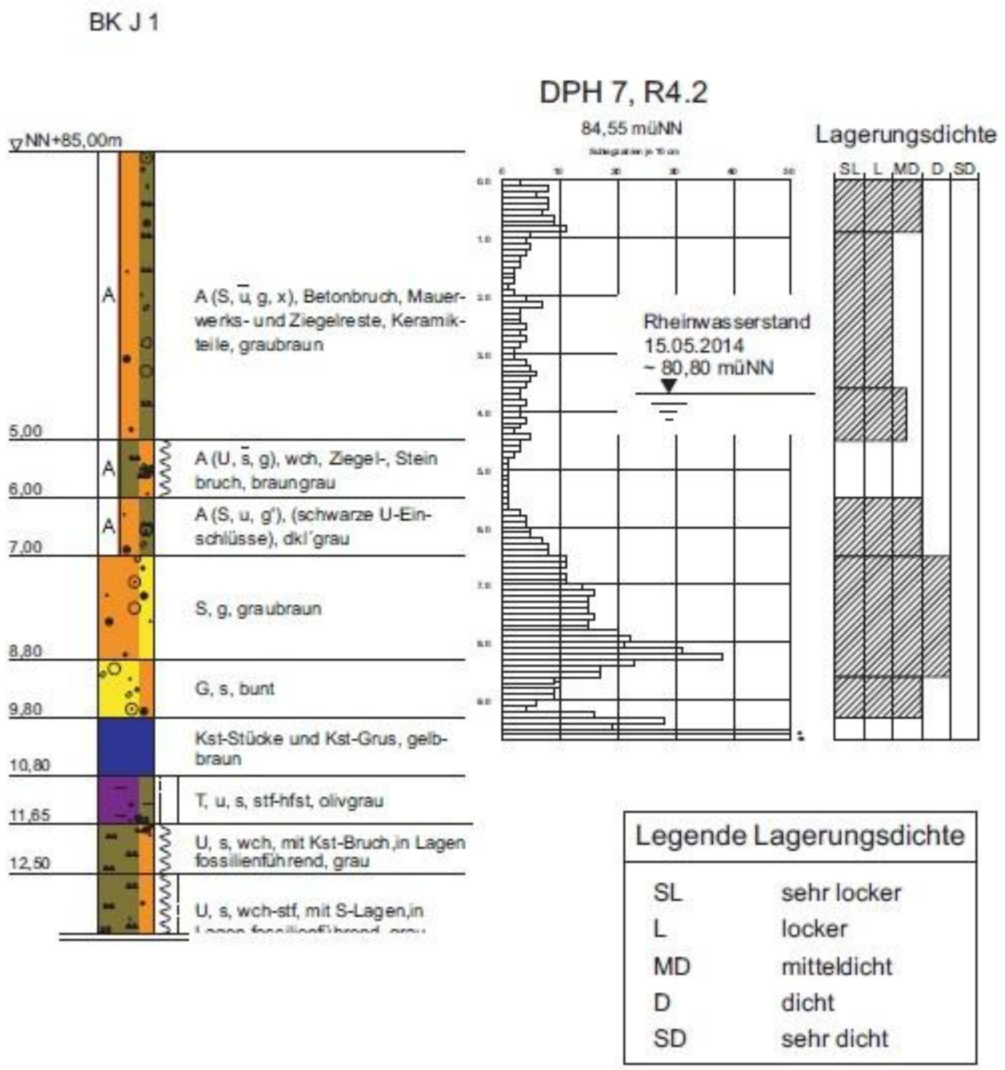


Fig. 54 Borehole data of BK J1 and the related DPH data of DPH 7, R4.2 and the converted density

DPH 4

Depth [m]	Soil	N10 average blows	Compactness/ Consistency	E_s after E_s table [MN/m²] E_s min	E_s max
0,00					
5,00	sand, strong silty, gravelly, boulderness	5	middle dense	29,0	48,0
6,00	silt, strong sandy,gravelly	5	soft	3,0	6,0
7,00	sand, silty, poor gravelly	7	middle dense	29,0	48,0
8,80	sand, gravelly	6	middle dense	29,0	48,0
9,80	gravel,sandy	13	dense	100,0	200,0

DPH 7

Depth [m]	Soil	N10 average blows	Compactness/ Consictency
0,00			
5,00	sand, strong silty, gravelly, boulderness	5	middle dense
6,00	silt, strong sandy,gravelly	2	soft
7,00	sand, silty, poor gravelly	5	middle dense
8,80	sand, gravelly	10	middle dense
9,80	gravel,sandy	18	dense

The data of the profiles of the DPH 4 and the DPH 7 show nearly the same structure. The two profiles have different average blow counts, but the consistencies and the compactness are the same.

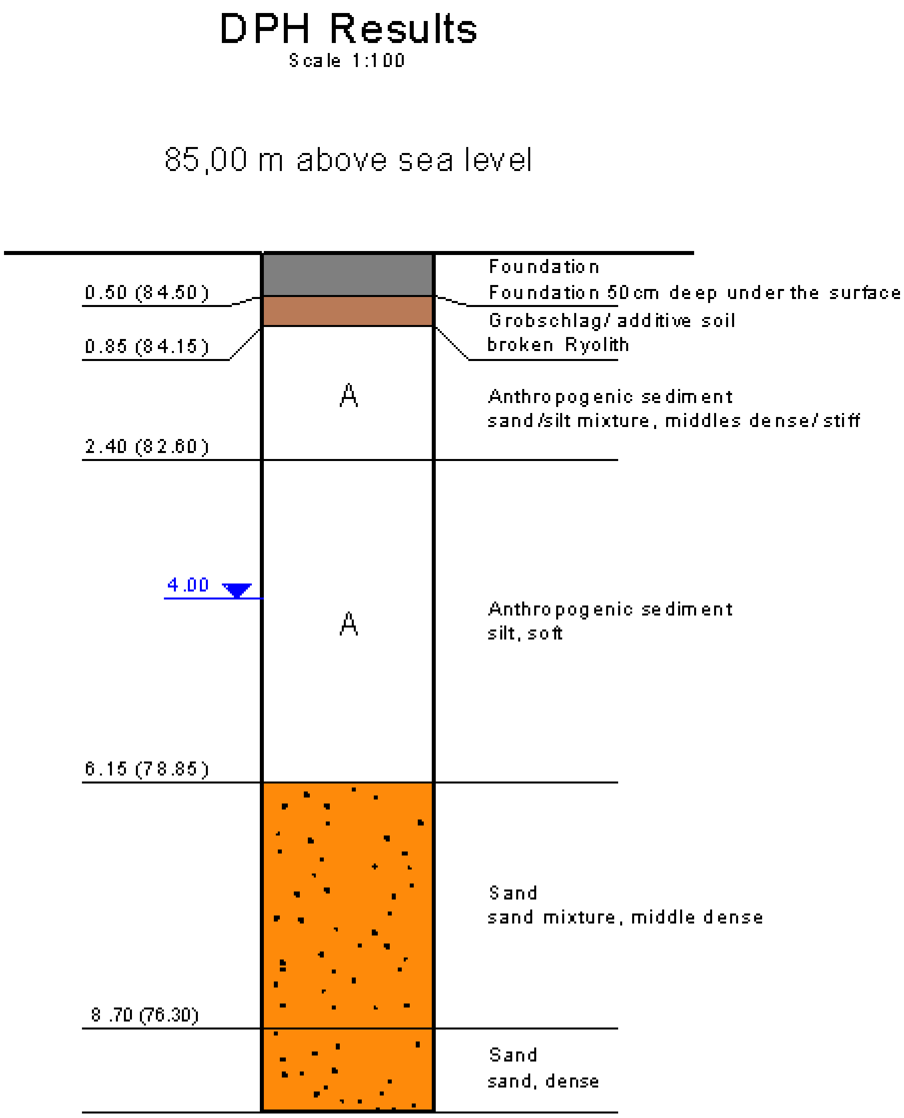


Fig. 55 Idealized DPH profile for the settlement analysis

Fig. 55 shows the idealized profile which is developed out of the DPH data and the drillings of BK J1 to BK J3, according to the work steps of Max Bögl. This idealized profile displays the stratigraphy for the area left of the auxiliary foundation S4.2. Underneath the 30cm embedded foundation is 55cm of crushed Ryolith. From 0.85m to 2.40 meters is a sand / silt mixture which is middle dense or stiff. For this layer it is difficult to evaluate the correct parameters for the later settlement analysis due to the heterogeneity of this layer. Also for the layer from 2.4 m to 6.15 m the consistency state is not that clear, the DPH profiles shows in between 5 m to 6 m they show very low blow counts, but this is very local and not in every DPH profile visible. This can indicate a clay or silt layer like in the CPT profiles. The decision to implement here no clay / silt layer was done due to the fact that DPH results in groundwater show lower values for higher densities and consistencies *(DIN 4094).* In a depth of 6.5m the fluviatil sediments start with a sand mixture, first with a middle dense compactness and after 9 meters the soil is dense. This idealized profile is implemented in the GGU software for the settlement analysis. The technical soil parameters are found in the Fig. 37 of chapter 8 and on the next page the corresponding E_s-values. In Tab. 26 the two different ways for calculating the constrained modulus are the constrained modulus via the constrained modulus table and the stress dependent constrained modulus according to *DIN 4094.*

Depth	Soil	N_{10} average	density	γ	γ'	$\sigma_{ü}$	Z	Z/b	a/b	i	σ_z	v_{min}	v_{max}	w
[m]		blows		[kN/m³]	[kN/m³]	[kN/m²		b=2,5m	10m/2,5m					
0,30	Foundation													
0,85	additive soil			21,00	11,00	11,55	0,55	0,22	4,00	0,79	197,50			
2,40	sand/silt mixture	5 to 7	middle dense /stiff	20,00	10,00	48,55	4,85	1,94	4,00	0,24	60,00	80,00	371,43	0,50
6,50	silt mixture	4	soft	19,00	9,00	93,85	6,20	2,48	4,00	0,16	40,00	74,00	279,80	0,60
9,00	sand mixture	7 to 10	middle dense	20,00	10,00	118,85	8,70	3,48	4,00	0,10	25,00	371,43	410,00	0,50
10,00	sand mixture	14	dense	20,00	11,00	129,85	9,30	3,72	4,00	0,09	22,50	446,38	446,38	0,50

Depth	E_s via E_s chart [MN/m²]		E_s stress d. [MN/m²]		average E_s	Interpretated E_s
[m]	E_s min	E_s max	E_s stress min	E_s stress max		
0,30						
0,85						100,0
2,40	6,0	48,0	7,1	32,9	23,5	20,0
6,50	3,0	6,0	8,0	30,2	11,8	10,0
9,00	29,0	48,0	42,6	47,0	41,6	40,0
10,00	48,0	77,0	53,0	53,0	57,8	60,0

Tab. 26 Shows the result of the different calculation ways for DPH data

8.5 Settlement analysis with calculated constrained modulus values of the idealized profiles

In this chapter is shown the settlement estimation of the idealized soil profile of the CPT and DPH data with a stiff foundation which symbolizes the auxiliary foundation. These steps, idealizing the profiles and assign of the constrained modulus data is done in accordance to the work steps in the company. The exception here is that the constrained modulus is calculated in different ways and interpreted afterwards. The foundation under load has a footing soil pressure of 250 [kN/m²]. The program GGU footing (Version 8.01/ 29.09.2012) is used for the settlement estimation. A bearing capacity failure is not part of this thesis and is not examined. The focus is just on the settlement analysis and the constrained modulus values. The *DIN 4019* provides a settlement correction for different soil types, where the calculated settlement is multiplied by a factor. This correction is not used due to the fact that the geology department of Max Bögl is not working with this correction. The foundation is classified as a stiff one and the tables after *Kany* are used.

In the upper left corner the stratigraphy is defined. The soil plus all the parameters are shown here. In the bottom left corner the parameters of the foundation are defined. The auxiliary foundations are calculated as strip spread footings with a length of 10 meters. In the middle image, the foundation is shown from the side and the corresponding stress and how it develops in the depth. The stress is only calculated to a depth of 20% of the initial loading, in cohesive soft to very soft soils, it should be kept in mind to calculate to the depth of 10% of the initial loading. In the illustration on the right hand side is shown the loading capacity to the foundation size and inside of the diagram the occurring calculated settlement for the different load levels.

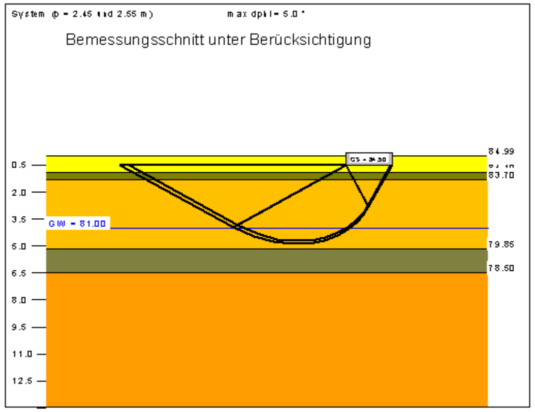
The idealized CPT profile data represent the area between the auxiliary foundations R3.2, S3.2, V1.2 and S2.2. The settlement analysis shows a value of 4.2 cm, which is shown in the settlement estimation on the right hand side at the load level of 250 [kN/m²]. Directly corresponding is the idealized CPT profile with the auxiliary foundation of V1.2. The CPT was done at the same position where V1.2 was build up, for the other auxiliary foundations which are named before is the idealized CPT profile the basis for calculations.

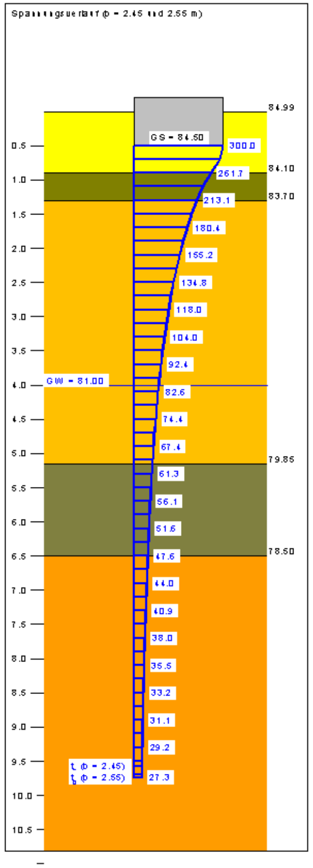
The idealized DPH profile data represent the area between the auxiliary foundations R5.2, S5.2, V2.2, R4.2 and S4.2. The settlement analysis shows a value of 4.0cm at a load level of 250 [kN/m²].

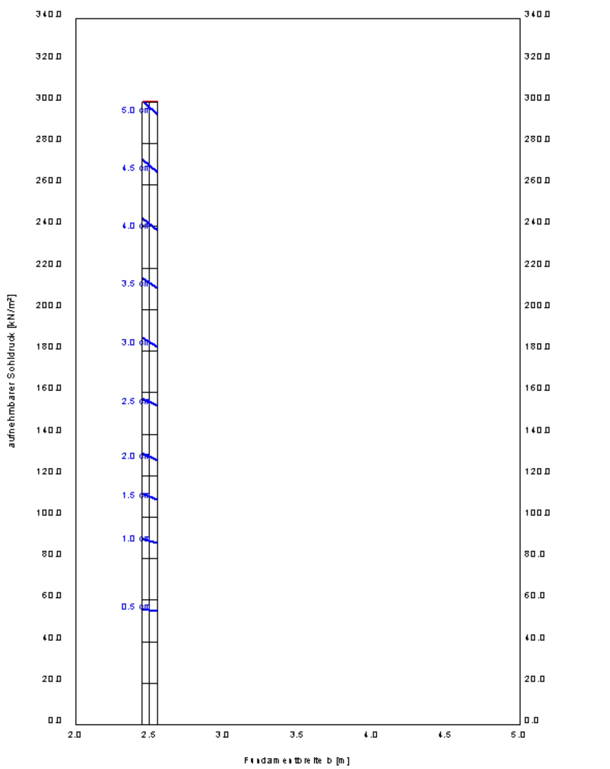
CPT idealized profile, Master thesis Weiss

Boden	γ [kN/m³]	γ' [kN/m³]	φ [°]	c [kN/m²]	E_s [MN/m²]	ν [-]	Bezeichnung
(yellow)	21.0	11.0	37.5	0.0	100.0	0.00	Additive soil
(olive)	20.0	11.0	32.5	0.0	40.0	0.00	A, sand (mechanical compacted)
(light orange)	20.0	10.0	32.0	0.0	15.0	0.00	A, sand mixture
(grey-green)	19.0	9.0	30.0	0.0	5.0	0.00	A, clay
(orange)	20.0	10.0	32.0	0.0	40.0	0.00	sand mixture

GGU-FOOTING / Version 8.19 / 15.07.2015
Berechnungsgrundlagen:
Norm: EC 7
Grundbruchformel nach DIN 4017:2006
Teilsicherheitskonzept (EC 7)
Streifenfundament (a = 10.00 m)
$\gamma_{R,v}$ = 1.30
γ_G = 1.20 BS-T
γ_Q = 1.30
Anteil Veränderliche Lasten = 0.500
$\gamma_{(G,Q)} = 0.500 \cdot \gamma_Q + (1 - 0.500) \cdot \gamma_G$
$\gamma_{(G,Q)}$ = 1.250
zul sigma auf 300.00 kN/m² begrenzt
OK Gelände = 85.00 m
Gründungssohle = 84.50 m
Grundwasser = 81.00 m
Grenztiefe mit p = 20.0 %
Grenztiefen spannungsvariabel bestimmt
aufnehmbarer Sohldruck
Setzungen





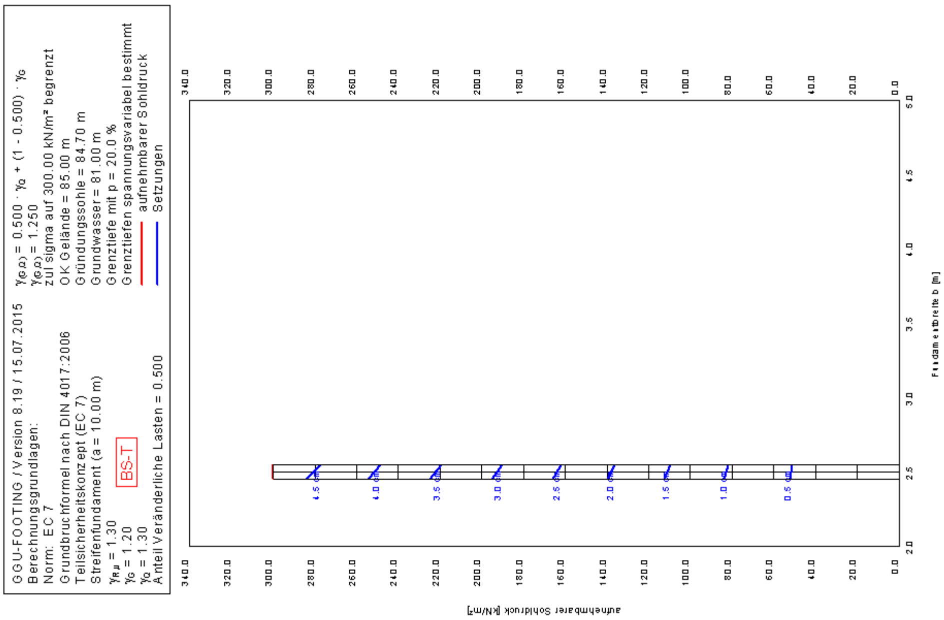


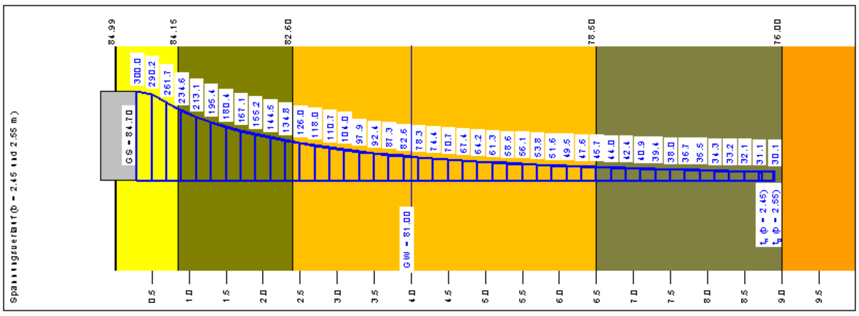
a [m]	b [m]	$\sigma_{0f,k}$ [kN/m²]	zul σ [kN/m²]	zul R [kN/m]	s [cm]	cal φ [°]	cal c [kN/m²]	γ_2 [kN/m³]	$\sigma_ü$ [kN/m²]	t_g [m]	UK LS [m]
10.00	2.45	487.5	300.0	735.0	4.98	32.3 *	0.00	19.44	10.29	9.57	4.73
10.00	2.55	487.5	300.0	765.0	5.11	32.3 *	0.00	19.23	10.29	9.74	4.90

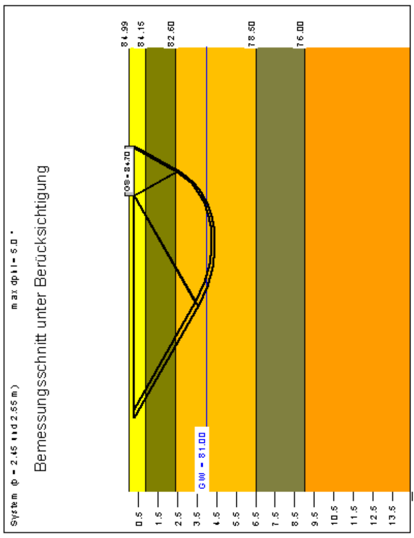
* phi wegen 5° Bedingung abgemindert
zul σ = $\sigma_{0f,k} / (\gamma_{R,v} \cdot \gamma_{(G,Q)}) = \sigma_{0f,k} / (1.30 \cdot 1.25) = \sigma_{0f,k} / 1.63$
Verhältnis Veränderliche (Q)/Gesamtlasten (G+Q) [-] = 0.50

DPH idealized profile, Master thesis Weiss

Boden	γ [kN/m³]	γ' [kN/m³]	φ [°]	c [kN/m²]	E_s [MN/m²]	ν [-]	Bezeichnung
	21.0	11.0	37.5	0.0	100.0	0.00	Luft
	20.0	10.0	30.0	3.0	20.0	0.00	Additive soil
	19.0	19.0	30.0	6.0	10.0	0.00	A, sand/silt mixture
	20.0	10.0	32.0	0.0	40.0	0.00	A, silt mixture
	20.0	11.0	32.0	0.0	60.0	0.00	sand mixture







a [m]	b [m]	$\sigma_{0f,k}$ [kN/m²]	zul σ [kN/m²]	zul R [kN/m]	s [cm]	cal φ [°]	cal c [kN/m²]	γ_2 [kN/m³]	σ_u [kN/m²]	t_g [m]	UK LS [m]
10.00	2.45	487.5	300.0	735.0	4.77	30.6 *	4.45	19.89	6.09	8.74	4.27
10.00	2.55	487.5	300.0	765.0	4.89	30.6 *	4.51	19.86	6.09	8.91	4.43

* phi wegen 5° Bedingung abgemindert
zul σ = $\sigma_{0f,k}$ / ($\gamma_{R,v}$ · $\gamma_{(G,Q)}$) = $\sigma_{0f,k}$ / (1.30 · 1.25) = $\sigma_{0f,k}$ / 1.63
Verhältnis Veränderliche (Q)/Gesamtlasten (G+Q) [-] = 0.50

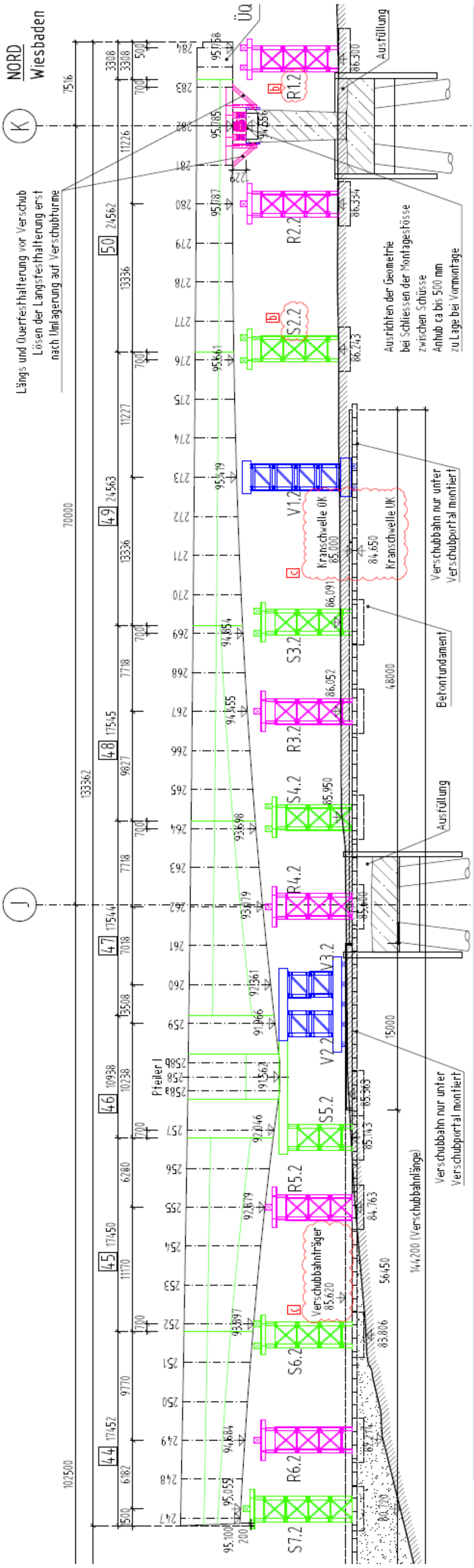
8.6 Comparison of the settlement data

In this chapter is the calculated settlement data are compared to the measured settlement data during construction. In the following table in Fig. 56 the measurement data of the different auxiliary foundations are shown. The data has different last measurement dates due to the fact that not all parts are loaded simultaneously and if an element was not used anymore it was built back. On every side of an auxiliary foundation, East and West, are four measurement points. These were measured weekly and the results apply to the first set of measurement data. The measurements are given in millimeters. The average settlement of an East or a West part of an auxiliary foundation is calculated by adding up the four measurements points and dividing them by four. In the case of a missing measurement value, the result is to be divided by the number of measurements.

The total average settlement of one auxiliary foundation is calculated by adding up the West and the East average settlement to get a result value for the whole settlement of one foundation. It should be kept in mind that these are just average values. Due to the fact that some single values are higher than others on one side of an auxiliary structure a differential settlement would normally occur. But to evaluate the result of the data interpretation and the result of the occurred settlement this is not taken in consideration.

The settlement analysis shows values of about 4 cm, for the left side of structure R4.2 with the DPH and the drillings as basis, and 4.2 cm for the right side of structure R4.2 with the CPT as basis. This is in accordance for all auxiliary foundation. Only the foundation of the structure V2.2 shows a higher settlement than forecasted, with a value 5.6 cm.

The higher occurred settlement, than forecasted are probably caused by the heterogeneity of the anthropogenic sediments. Due to the fact that the DPH's tests were done next to the V2.2 auxiliary foundation the result of the estimation is only partly satisfying. The results of the settlement estimation with the CPT basis show not higher measured settlements than forecasted. This leads also to the statement that a CPT is much more reliable than a DPH. For the future, a settlement analysis in heterogenic sediments needs as much data as possible and is to evaluate if it is possible and economically to build up an idealized profile underneath of every auxiliary foundation, to evaluate for every single structure the heterogeneity and the parameters. That would give a much better forecast situation.



Auxiliary foundation	R5.2		S5.2		V2.2		R4.2		S4.2		R3.2	
Last data measurement	16.03.2015		16.03.2015		16.03.2015		18.08.2014		04.09.2014		04.09.2014	
Measuement position	east	west	east	west	east	west	east	west	east	west	east	west
Settlement in [mm]	-21	-19	-38	-36	-75	-86	-2	-2	-5	-6	-4	-5
	-23	-17	-37	-35	-81	-92	/	/	-4	-11	-4	-5
	-29	-24	-30	-31	-24	-34	-4	-5	-4	-5	-4	-3
	-29	-17	-29	-29	-25	-37	-6	-7	1	-8	-5	-4
Average settlement in [mm]	-25,5	-19,3	-33,5	-32,8	-51,3	-62,3	-4	-4,67	-3	-7,5	-4,25	-4,25
Average Total settlement for each auxiliary foundation in [mm]	-22,375		-33,125		-56,75		-4,33333333		-5,25		-4,25	

Auxiliary foundation	S3.2		V1.2		S2.2		R2.2		R1.2	
Last data measurement	04.09.2014		16.03.2015		18.08.2014		18.08.2014		04.09.2014	
Measuement position	east	west	east	west	east	west	east	west	east	west
Settlement in [mm]	-6	-5	-2	-5	-12	-19	-3	0	-4	-4
	-5	-6	-2	-4	-13	-20	-3	-4	-1	-3
	-10	-11	-4	-4	-13	-16	/	-3	0	0
	-10	-13	-4	-5	-15	-19	-4	-3	1	0
Average settlement in [mm]	-7,75	-8,75	-3	-4,5	-13,3	-18,5	-3,33	-2,5	-1	-1,75
Average Total settlement for each auxiliary foundation in [mm]	-8,25		-3,75		-15,875		-2,91666667		-1,375	

Fig. 56 Auxiliary foundations with the occurred settlement data

9 Conclusion

As a result of this thesis one can say that the common practices for examining a construction site work wellup to certain point. The CPT, SPT and DP are very helpful investigation tools, especially in combination with borehole data. With that information it is possible to understand the stratigraphy and to make some conclusions via the in-situ tests about the compactness or consistency of the soil. The CPT tests provided reliably values for the constrained and the settlement estimation showed a satisfying result, which supports the editor’s impression that the interpretation of the CPT results are easier than the DPH ones. For the CPT test work the soil type classification charts, all data are pointing in the same direction, but there are some outliners as well. The main problems start with the interpretation and determination of the constrained modulus out of field data.

Constrained modulus via E_s-table

The determination of E_s via the E_s-table uses the literature sources and the experience values of the editors which are commonly used in today’s construction sector. But for a precise analysis the magnitude of the values is too wide. The problem is often how to deal with mixtures of soil states, as an example silty sand or clayey sand. For these soils are not even given some values in the literature, it can just be interpreted by experience.

Constrained modulus via α chart according to *DIN 4094*

The results of the average constrained modulus E_s via the α-chart use the information of which soil is tested and the cone tip resistance q_c value of the CPT to evaluate α via the α-table. The problem is that the values become a range with a minimum and a maximum, but the ranges are mostly smaller compared to result of E_s via the E_s-table, especially when the cone tip resistance is low. This leads to the interpretation that this way of calculation can be more precise, but this depends on the correctness of the input parameter α. For a low q_c shows this way a very small magnitude.

Constrained modulus via stress dependent according to *DIN 4094*

This way shows the same tendency like the α calculation, smaller ranges, but the formula is not defined for every area of the cone tip resistance values. There is a gap between 3.5 [MPa] and 5 [MPa] which is not defined. Also the range for q_c is defined too small for the calculation of the stiffness factor v. Clayey material and sand exist with higher and lower [MPa] values, which aren't defined. This is a limiting point of this calculation way.

The conclusion for interpreting or calculating constrained modulus values out of field data is that there will be according to the state of the art, always a magnitude with nowadays sampling and calculation techniques. The three different ways of calculation parallel to each other have shown that the two techniques which include the cone tip resistance have sometimes a smaller range. From the logical point of view, does it make sense to calculate with the cone tip resistance as a parameter, to get an in-situ value of the soil. The solution via the constrained modulus table shows mostly the biggest magnitudes, due to the fact that this way is commonly used in today's engineering practice, would the result be a recommendation for calculating the constrained modulus with all three calculation ways, for CPT data and two for DPH, to limit the magnitude to the smallest amount. Due to the fact that evaluations of the constrained modulus with a cone tip resistance is more reliable, one would recommend for CPT data a higher focus on the calculation ways via the α table and the stress dependent constrained modulus. For DPH data it makes sense as well to use the N_{10} value to determine the stress dependent constrained modulus. At least the additional calculation ways can give a confirmation of the interpreted values. Caused by the ranges of the constrained modulus, has to be chosen for the settlement analysis a value which represents a high reliability. This leads to a magnitude which depends on the knowledge and experience of the editor.

At least the additional calculation ways can give a confirmation of the interpreted values.

Even when the result is still a magnitude of the constrained modulus it is possible to evaluate a most reliable range for the different areas.

9.1 Future prospects

Nearly all papers and references which themed the correlation or the results of the different tests show that further studies should be done and there is an engineering judgment necessary to evaluate the data. The problem is that a correlation for a parameter works only for one soil type and often shows a stratigraphy profile a heterogenic soil mixture. After this thesis, further studies should be done as well, but some main points should be considered to get a much better result.

- In general, a higher standing for a geotechnical report in a construction project, this means more data to evaluate the ground parameters. However, the limiting factor here is money and time. Companies try to work as economically as possible, which often means that there are not enough geologist or geotechnical engineers employed. The question is can it be more economically for a company to employ more geologist / geotechnical engineers caused through their knowledge of the ground behavior and their generated savings in a construction.

- Better technical equipment for sampling. There are quite good possibilities of combining methods for example, CPT combined with seismic sensors or the combination of CPT and a temperature sensor, etc. Which combination should be used is the question for the geotechnical engineers and the details of a project. For a right choice a good education is necessary. These CPT combinations are already on the market but are not yet often used which depends on the knowledge and on the costs.

- Developing a more precise and comprehensible chart for the α values. To get more information how the α value behaves and how these values are estimated, to get in the end a more detailed subdivision. Different authors like *Robertson, (2009)* developed a correlation related to the soil behavior type index. Other authors were experimenting with fixed α values, a further study should examine these. These values have to be confirmed by real settlement data and a back calculation to a CPT profile.

- Developing a soil type classification chart which includes corresponding constrained modulus values. Due to the fact that the soil type charts use the cone tip resistance and the sleeve friction and their relative the friction ratio, to classify the soil. These parameters can be useful to develop such a chart.

- A detailed study for the stress dependent determination way to close the gap between 3.5 [MPa] and 5 [MPa] and to define areas out of the [MPa] ranges, which is necessary because some materials show values out of the defined range.

- The newer construction scene starts the usage of 4D computer programs, which shows the development of a building with all technical details. A possibility could be that it should be standard to build up a 3D model of the underground for bigger projects for a better understanding of the stratigraphy. Additional the usage of Finite Element Analysis for settlement calculation, which can be done as well in 4D. So when a virtual model is built up and compared to the live construction, a data base could be built up and the settlement data can be implemented in the virtual model. A real live time settlement analysis could be done were the data can be customized to state.

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