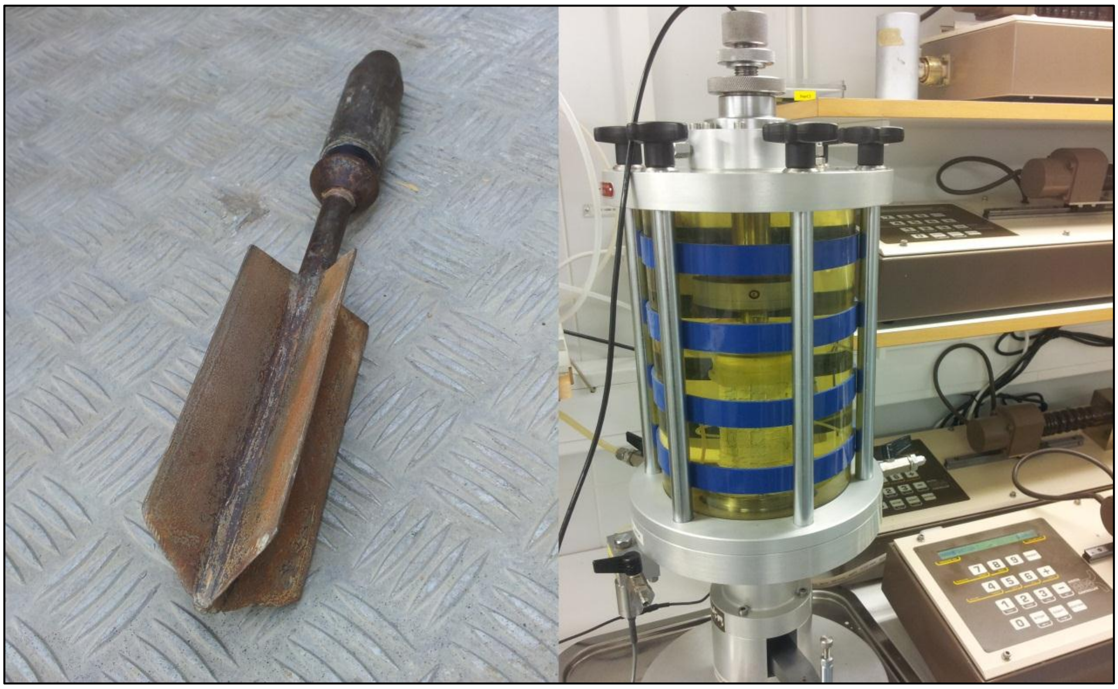

Correction of shear strength in cohesive soil

A comparison focused on vane tests in west Sweden

Master of Science Thesis in the Master's Programme Geo and Water Engineering

MARCUS JONSSON
CAROLINA SELLIN

Department of Civil and Environmental Engineering
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CHALMERS UNIVERSITY OF TECHNOLOGY
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Cover:
Left: steel vane with clutch. Right: triaxial cell with a sample mounted.
Photo: Marcus Jonsson

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ABSTRACT

Two commonly used methods for determining the direct shear strength for clay are the vane test and the fall cone test. The measured values should be adjusted with a correction factor, on recommendations from the Swedish Geotechnical Institute, in order to be compatible with the direct shear strength. Comparisons with direct shear tests have raised the question whether the corrected values are representative for clays in the western part of Sweden or not. This master thesis is comparing laboratory test results and empirical relations with corrected and uncorrected test results from vane tests and fall cone tests done in clays from western Sweden. Existing test results are collected for seven locations, where each location is studied and analysed separately. The method in this thesis is to first evaluate the shear strength for the location, called best estimated shear strength, based on results from Direct Shear tests, triaxial tests, Swedish empirical relations for CRS tests and evaluated CPTs. A trendline is evaluated for the vane test results and is compared with the best estimated shear strength. All results from vane tests are weighted equally, except obviously faulty values, which are deleted. The same method is used for the fall cone tests. The results show that a majority of the locations have a good correlation for shallow depths between corrected shear strength from vane tests and the best estimated shear strength. This correlation is less distinct for fall cone tests, which usually exhibit a larger scatter. The trends for uncorrected shear strength align better with the best estimated shear strength at larger depths. Two locations stand out. One location had good correlation between corrected shear strength and best estimated shear strength for all depths. The other location did not have a correlation at any depth, neither for corrected nor uncorrected shear strength. A majority of the locations also showed good correlation between empirical relation and results from direct shear tests, with slightly lower values from the test results. The main conclusion is that the corrected shear strength is a good estimate for shallow depths, but gives an underestimation at larger depths for the investigated locations.

Key words: Gothenburg, clay, Göta River (Göta Älv), shear strength, vane test, fall cone test, correction factor

Contents

Preface

This master thesis is a method evaluation and case study of the correction factor for shear strength in cohesive soil for seven locations in west Sweden. The thesis has been carried out from January 2012 to June 2012 at the Department of Civil and Environmental Engineering, Division of GeoEngineering, Geotechnical Engineering Research Group, Chalmers University of Technology. The research question was presented by Urban Högsta and Ola Skepp at SWECO Infrastructure Göteborg and developed together with the examiner and main supervisor Göran Sällfors, Professor at the Division of GeoEngineering.

First of all, we would like to thank our supervisor Göran Sällfors for providing us data, but mostly for a never-ending support and encouragement throughout our work.

We would like to thank all companies involved for their co-operation and involvement to make this thesis possible, as well as Peter Hedborg, Mats Olsson and Ingemar Forsgren for taking their time to explain theory and practice.

We would like to thank all co-workers at SWECO Infrastructure, especially Urban Högsta and Ola Skepp for their project idea and their support. We would like to give our appreciation to Magnus af Petersens for the everyday help and for answering our questions.

Göteborg, June 2012

Carolina Sellin & Marcus Jonsson

Notations and abbreviations

Roman letters

A	cross-sectional area
a	material parameter for empirical relation
a_c	area factor for CPT evaluation
a_{Active}	material parameter for clay in active shear zone for empirical relation
a_{Direct}	material parameter for clay in direct shear zone for empirical relation
$a_{Passive}$	material parameter for clay in passive shear zone for empirical relation
b	material parameter for empirical relation
C	material parameter for calculating direct shear strength from CPT
c'	cohesion
D	diameter
g	gravity
H	height of vane
h	sample height in direct shear test
i	cone penetration depth in fall cone test
K_0	earth pressure coefficient at rest
k	constant for cone angle in fall cone test
q_t	uncorrected cone tip resistance
q_c	corrected cone tip resistance
T	shear force in direct shear test
Q	mass of cone in fall cone test
u	pore pressure
u_2	pore pressure measured directly behind cone in CPT
w_L	liquid limit

Greek letters

Δs	horizontal deformation
γ	unit weight

γ' effective unit weight

γ_{DS} shear strain

μ correction factor

ρ density

σ total stress

σ_1 major principal total stress

σ_{1f} major principal total stress at failure

σ_3 minor principal total stress

σ_{3f} minor principal total stress at failure

σ' effective stress

σ'_1 major principal effective stress

σ'_3 minor principal effective stress

σ'_c preconsolidation pressure

σ'_{h0} effective horizontal in situ stress

σ'_{v0} effective vertical in situ stress

T torque in vane test

τ'_f drained shear strength

τ_f uncorrected undrained shear strength

τ_k uncorrected undrained shear strength from fall cone test

τ_v uncorrected undrained shear strength from vane test

τ_{fu} corrected undrained shear strength

τ_{fu}^A undrained active shear strength

τ_{fu}^D undrained direct shear strength

τ_{fu}^P undrained passive shear strength

φ' effective friction angle

Abbreviations

CPT Cone Penetration Test

CRS	Constant Rate of Strain
ESP	Effective Stress Path
NC	Normal Consolidated
OC	Over Consolidated
OCR	Over Consolidation Ratio
SGI	Swedish Geotechnical Institute

1 Introduction

One of the disciplines within geotechnical engineering is stability calculations, where the critical parameter is shear strength. High shear strength often results in low construction cost, while low shear strength often requires ground reinforcements and more restrictions on construction sites. It is therefore of great importance to determine the shear strength with a reliable and cost-effective method.

1.1 Background

There are several methods to determine shear strength today, both with field and laboratory tests. One of the most common tests for shear strength in Sweden is the vane test, which measures the undisturbed shear strength directly in the soil. Another common test method is the fall cone test which is performed in a laboratory. These methods are relatively cheap and easily executed. Studies have shown that the obtained shear strength from these two test methods should be corrected in order to give a comparable value to the direct shear strength (Larsson, et al., 2007). The recommendation today from the Swedish Geotechnical Institute (SGI) is to correct all vane and fall cone tests with respect to the liquid limit. Comparisons made with direct shear tests have raised a discussion among geotechnical engineers, whether the corrected shear strength is representative for the soil in the western part of Sweden or not. Chalmers University of Technology (CTH) and the consultant company SWECO are therefore interested in an investigation of the correction factor and if it should to be revised or refined.

1.2 Aim

The purpose of the report is to investigate if the corrected shear strength from vane and fall cone tests corresponds to the estimated shear strength. This is obtained from more advanced laboratory tests such as triaxial tests and direct shear tests. Swedish empirical relations are also taken into consideration. The report will discuss if the correction factor can be revised to better fit the conditions in the Gothenburg region. The thesis is written to raise the discussion about vane test correction and present actual result of the need of revising the correction factor or not.

1.3 Method

The report is based on several methods. A desk study was performed in the initial phase to give a deeper knowledge of the test methods as well as of theoretical soil mechanics. Study visits were done to observe and learn the practical part of both laboratory and field tests.

A number of geotechnical locations were chosen to represent different conditions in the western part of Sweden, which all consist of rather deep clay formations. Results from field and laboratory tests were collected from the Geotechnical Engineering division at Chalmers, SWECO Gothenburg and other geotechnical companies in the

Gothenburg region. The locations are evaluated separately and the test data is only representative for the specific location.

The test results for each location are summarized and calculated in Microsoft Excel 2010, see Table 1.1. The calculations were performed based on SGI:s regulations for cohesive soil. (Larsson, et al., 2007)

Table 1.1 Test procedures and how the result is used in the report.

Test	How the data is interpreted
Piston Sampling	(Input values to other tests)
Direct Shear Test	Direct
Fall Cone Test	Calculation with correction factor
Vane Test	
Cone Penetration Test	
Oedometer Test	Empirical calculated value
Triaxial Test	Direct and empirical calculated value

The calculated shear strength is first analysed from a geotechnical perspective, where unreliable or faulty values are removed. The values are then analysed with a statistical and geotechnical perspective to identify trends of the shear strength for each location. Vane tests and fall cone tests are presented with one trend each, and the other methods are presented as one best estimated trend with depth. The correlation between those trends will be discussed an analysed with respect to the correction factor.

1.4 Delimitations

The report will focus on seven locations in the Gothenburg region, where vane tests have been performed in glacial and post-glacial marine clay. The clay considered in this report is limited to normal to lightly overconsolidated clay. The report only presents the test procedures from which results are used, but does not discuss the accuracy of the standardized methods used today. Also, it does not cover the individual differences of drill rig, laboratory equipment or staff performance. Only the empirical relations presented in SGI Information 3 are used.

2 History of the correction factor

The history of the Swedish correction factor is a summary of SGI Information 3 by Larsson, et al. (2007). The initial correction factor was introduced for fall cone tests. The correction factor was based on comparisons with pile loading tests done in 1900 and back-calculations from failures with circular slip-surfaces. As geotechnical test methods were further developed, the fall cone tests were calibrated again during the 1930's. That included data from full-scale loading tests, registered failures and landslides. Measurements showed that the fall cone test had to be corrected with respect to the liquid limit.

The vane test that is used today was introduced by Cadling and Odenstad in 1950. The test was calibrated based on back-calculated landslides and on one full-scale loading test. The fall cone test was then re-calibrated based on the vane test. It was known that the measured strength values needed to be corrected, the correction from fall cone test was then implemented. The correction was depending on the liquid limit, but no standard had been chosen for determining the corrected shear strength.

To set a Swedish recommendation for the correction, SGI had a technical meeting in 1969. It was decided that measured values from both vane test and fall cone test must be adjusted with a correction factor according to the equation;

$$\tau_{fu} = \mu \cdot \tau_f \tag{2.1}$$

where τ_{fu} = corrected undrained shear strength

μ = correction factor

τ_f = uncorrected undrained shear strength

The correction factor μ is depending on the liquid limit of the soil and the recommendation from SGI was to use Figure 2.1 below. This reduction was by some considered to be undersized and thereby a risk of overestimating the strength of the soil. To cope with this, the recommendation was therefore to use carefully and somewhat conservative chosen values of strength.

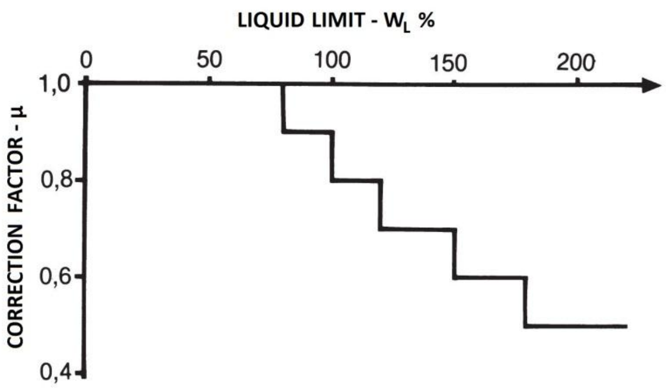


Figure 2.1 Correction factor from 1969

Research has been made to refine this correction. In Andréasson study in 1974, Bjerrum's theories for the plasticity index were converted to the use of liquid limit. This study was found to give similar design values of shear strength, as when using the SGI method from 1969 with carefully chosen values. This method was therefore not chosen as a new recommendation from SGI. Another study was performed by Helenelund in 1977, which corresponded well with Andréasson's method. The two methods diverges when the liquid limit is high, where Helenelund's method has a larger reduction of shear strength. This method was not implemented either but these studies formed the basis for the new SGI recommendation in 1984.

The present standard for correction of shear strength was developed in 1984 and is based on both prior experience and newly gathered information. Empirical relations from preconsolidation, loading and Atterberg limits were taken under consideration when developing the new standard. The factor of correction is presented in equation 2.2 below and is shown in Figure 2.2.

$$\mu = \left(\frac{0.43}{w_L}\right)^{0.45} \geq 0{,}5 \tag{2.2}$$

where w_L = corrected undrained shear strength

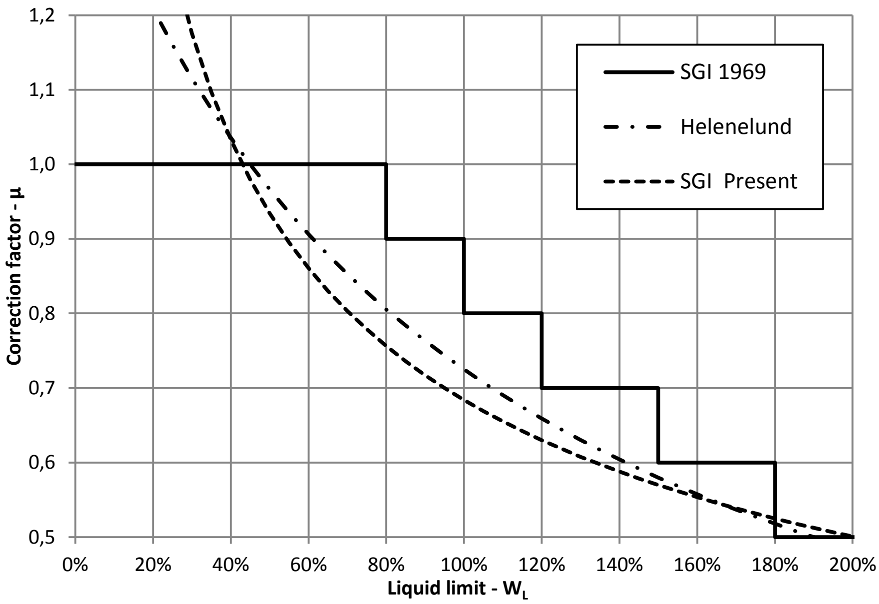


Figure 2.2 Overview of correction methods

The correction factor was based on results from tests in clays with an OCR of approximately 1.3, the method of correction is therefore applicable for normally and lightly overconsolidated clays. Clays with OCR≥1.5 are defined as overconsolidated and are also corrected with regard to OCR. The correction factor for overconsolidated clays is calculated by means of equation 2.3 and results in lower values on µ. The correction for overconsolidated clays is based on an investigation made by Larsson and Åhnberg (2003 cited in (Larsson, et al., 2007)) where overconsolidated Swedish clays were studied.

$$\mu = \left(\frac{0.43}{w_L}\right)^{0.45}\left(\frac{OCR}{1.3}\right)^{-0.15} \tag{2.3}$$

where OCR = overconsolidation ratio

3 Theory of cohesive soil behaviour

3.1 Soil Properties

To simulate the stress situations in the ground, one must know what parameters to evaluate and how they influence the stability. It is also important to know what stress conditions that are applicable in a certain situation. This chapter presents relevant theories for stability problems.

3.1.1 Stress situations

The stress situation is crucial for determining the stability. The total stress for horizontal ground surface is defined in equation 3.1. The effective stress also includes the pore pressure in the ground, see equation 3.2.

$$\sigma = \gamma \cdot z \tag{3.1}$$

$$\sigma' = \sigma - u \tag{3.2}$$

where σ = total stress

σ' = effective stress

γ = density

z = depth from surface

u = pore pressure

If the soil has been exposed to a greater stress than the present situation, the soil grains will be denser packed and have a “memory” of this stress situation. It could for instance occur in an eroded river valley, as shown in Figure 3.1.

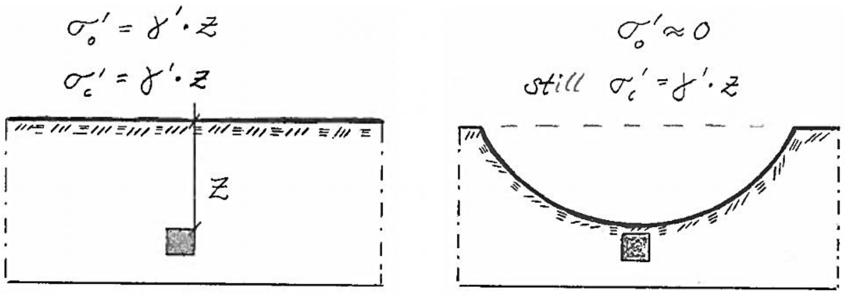


Figure 3.1 Stress situation before and after erosion of a river valley.

This phenomenon is called preconsolidation pressure, σ_c', and will only be discussed for loose, sedimented clay in this chapter. It also occurs in the dry crust of the clay, where the strength is increased due to dehydration of the clay, fluctuations of the ground water and effects of weathering (Larsson, et al., 2007). The theoretical definition of the consolidation state for clay is

$\sigma_{v0}' = \sigma_c' \rightarrow$ normally consolidated

$\sigma_{v0}' < \sigma_c' \rightarrow$ overconsolidated

Clays often have a certain degree of overconsolidation. This is due to the creep process in the soil structure, but also other processes can contribute. Therefore, this value is often presented as the overconsolidation ratio which is

$$OCR = \frac{\sigma'_c}{\sigma'_{V0}} \tag{3.3}$$

where σ'_c = preconsolidation pressure

σ'_{V0} = effective vertical in situ stress

$\mathrm{OCR} \leq 1.5 \rightarrow$ normally to lightly overconsolidated (NC)

$\mathrm{OCR} \leq 1.5 \rightarrow$ overconsolidated (OC)

For marine clays on the Swedish west coast, the OCR is seldom lower than 1.3. Areas with lower OCR occur and could be caused by changes in the soil profile. These changes can be filling material or changes in pore pressure, which contribute to progressing settlements.

3.1.2 Soil conditions

The soil is exposed to different shear stresses depending on the loading direction. The force which acts along a slip surface can be induced by a heavy load on the top of a slope. The soil then needs to have a resistance in order to keep a global equilibrium. The resisting force for the soil is measured as the shear strength. The shear strength varies with the loading direction, due to anisotropy in the material. It also varies based on the clay content in the soil.

Cohesive soils have a very low hydraulic conductivity and are practically always saturated with water. Additional loads are therefore initially taken by the pore pressure, creating a pore over pressure. This is called an undrained situation and is measured on a total stress basis;

$$\tau_{fu} = \frac{\sigma_{1f} - \sigma_{3f}}{2} \tag{3.4}$$

where τ_{fu} = undrained shear strength

σ_{1f} = major principal total stress at failure

σ_{3f} = minor principal total stress at failure

If the cohesive soil has draining layers or has had the same load situation over a long time, the pore water is expelled and the soil skeleton gradually carries the load instead. This means that a change in total stress gradually leads to a change in effective stress. This is called a drained situation and is measured on an effective stress basis;

$$\tau_f' = c' + \sigma' \tan(\varphi') \tag{3.5}$$

where τ_f' = drained shear strength

c' = cohesion

φ' = friction angle

A fine grained soil, such as cohesive soils, has different consistency and soil behaviour depending on the clay content and the natural water content. Atterberg defined in 1913 four states for the consistency and the transition between them, presented in Figure 3.2.

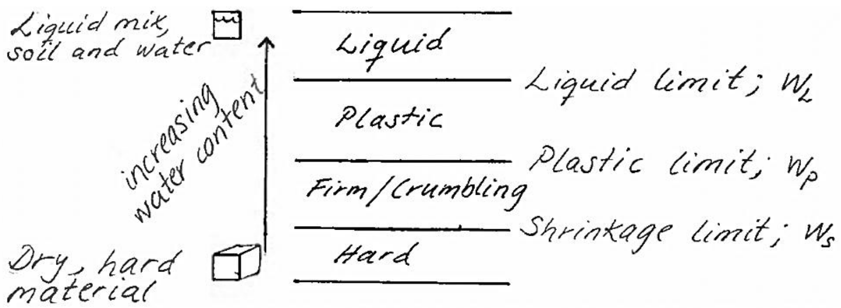


Figure 3.2 Definition of Atterberg limits.

3.2 Soil modelling

The stress situation in the soil is normally estimated with results from field and laboratory tests, but the results must be put in a context to give an overall estimation. There are different kinds of soil models, covering the complexity of the soil in different ways. This chapter will focus on the most common soil model for stability problems, where the shear strength is a crucial parameter.

3.2.1 Shear zones in a stability problem

The soil along a shear surface in a slope has different stress situations depending on where the measuring is done. The vertical stress is the major stress in the top of the slope, while the horizontal stress is the major stress at the toe. As the soil has a stress induced anisotropy, the shear strength will differ with stress direction (Kompetenscentrum, u.d.). The slip surface is therefore often divided in to three parts; active, passive and direct zone, to take that in to account, see Figure 3.3. The shear strength for the direct shear zone can be measured both in field and laboratory test, while the active and passive shear strength can only be measured in triaxial tests.

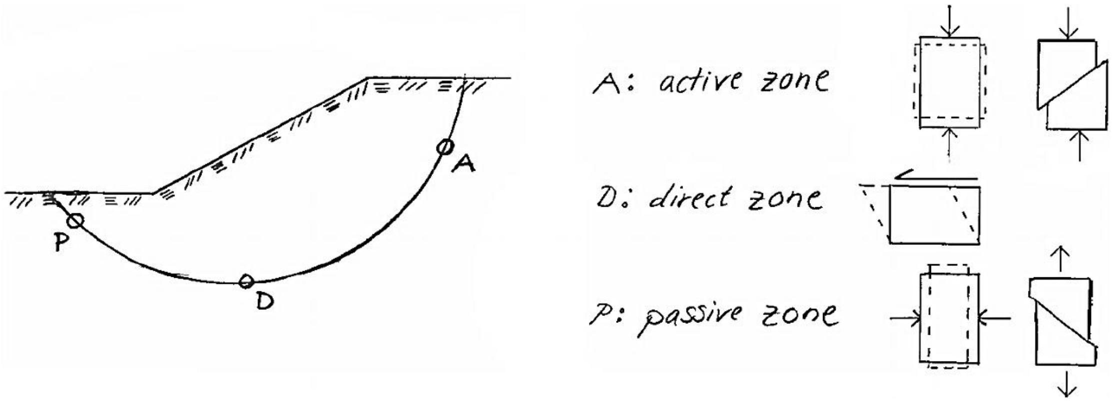


Figure 3.3 Definition of shear zones in a slope stability model.

3.2.2 Mohr-Coulomb failure theory

Given the major and minor stress for the soil and assuming that they are vertical and horizontal, Mohr's circle defines the stresses in all other directions. (Kompetenscentrum, u.d.) The radius of the circle equals the shear strength and the centre of the circle is the average of the two effective stresses.

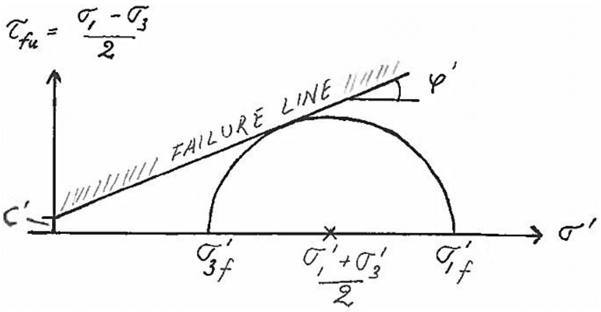


Figure 3.4 The Mohr-Coulomb failure criterion for active shear strength.

The Mohr-Coulomb failure criterion is a commonly used model for defining the failure criterion for the active and passive zone. The failure criterion is based on how the soil responds to stresses acting on a sample in three dimensions. If there is a stress situation which tangents the failure criterion, failure will occur in the soil and the maximum shear strength is reached. It is therefore not possible to have a stress situation outside the failure criterion. The parameters needed for defining the failure line is cohesion and friction angle, which are evaluated from laboratory tests.

3.2.3 Yield envelope

For a soil with a pronounced preconsolidation pressure, the soil will undergo small deformations as long as the stress is less than the preconsolidation pressure;

$\sigma'_v < \sigma'_c \rightarrow$ small strains

$\sigma'_v \geq \sigma'_c \rightarrow$ large strains

This criterion can be combined with the Mohr-Coulomb failure criterion and gives a so called yield envelope. The soil is restricted with the failure lines for active and passive shear strength, see Figure 3.5. The stress is less than σ'_c within the yield envelope for the major stress and less than $K_0 \cdot \sigma'_c$ for the minor stress. This boundary is referred to as the σ'_c-line in the report. If the relation between σ'_1 and σ'_3 exceeds the envelope, large deformations or failure will occur.

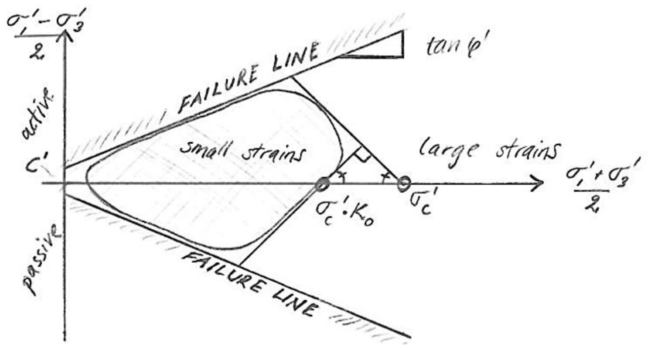


Figure 3.5 Principal sketch of two-dimensional yield envelope.

This method can be utilized on test results from consolidated, undrained triaxial tests, since σ'_1 and σ'_3 are monitored during the test.

In theory, the shape has clearly defined corners, but test results show that the shape has more likely rounded corners. It also showed that the specific shape varied for the clays depending on clay content, structure and sample disturbance. (Larsson & Sällfors, 1981)

3.3 Empirical relations

Empirical relations should be used to get a first estimation of what shear strength to expect for a given location. The results can later be compared with actual test results. It can also be used as guidance to which areas further investigations should be focused on.

Ladd & Foott (cited in Karlsrud, 2010) presented the SHANSEP method in 1974, which stands for Stress History And Normalised Soil Engineering Properties. The concept is that the shear strength is normalized with respect to the vertical in situ stress and compared with the stress history of the soil. This led to a general equation;

$$\tau_{fu} = S \cdot \sigma'_{V0} \cdot OCR^m \tag{3.6}$$

where S = material parameter

m = material parameter

Karlsrud (2010) presented results from shear strength tests on samples taken with a block sampler in Norwegian clays. He used the SHANSEP method and presented trends of the parameters S and m for each of the three shear zones active, direct and passive. Generally Norwegian, marine clay contains more silt than the Swedish clays and therefore have other properties. Typical values for Norwegian, marine clays are w_L= 40 ±5% and γ = 17-21 kN/m^3 (Sandven, u.d.). Two equations which are used in Norway and are comparable to Swedish empirical relations are

$$\tau_{fu}^{A} \approx 0.3 \cdot \sigma'_0 \cdot OCR^{0.8} \tag{3.7}$$

$$\tau_{fu}^{A} \approx 0.32 \cdot \sigma'_0 \cdot OCR^{0.65} \tag{3.8}$$

where τ_{fu}^{A} = undrained active shear strength

SGI presents one empirical relation for shear strength in cohesive soil (Larsson, et al., 2007). The method depends on the type of soil, the loading situation and the preconsolidation pressure. The loading situation is divided into the three shearing zones; active, direct and passive. The general equation is

$$\tau_{fu} = a \cdot \sigma'_c \cdot OCR^{-(1-b)} \tag{3.9}$$

where a = material parameter

b = material parameter

The factor a takes the soil type and loading situation in to account and b is usually estimated to 0.8 but can vary from 0.7 to 0.9. For clay, the three cases will then be

$$\tau_{fu}^{A} \approx 0.33 \cdot \sigma'_c \cdot OCR^{-0.2} \tag{3.10}$$

$$\tau_{fu}^{D} \approx \left(0.125 + \frac{0.205 \cdot w_L}{1.17}\right) \cdot \sigma'_c \cdot OCR^{-0.2} \tag{3.11}$$

$$\tau_{fu}^{P} \approx \left(0.055 + \frac{0.275 \cdot w_L}{1.17}\right) \cdot \sigma'_c \cdot OCR^{-0.2} \tag{3.12}$$

where τ_{fu}^{D} = undrained direct shear strength

τ_{fu}^{P} = undrained passive shear strength

Equation 3.10 is based on a database of test results, while equation 3.11 and 3.12 are based on empiricism.

When comparing the Swedish and Norwegian empirical relations, it can be concluded that the Norwegian method is somewhat more conservative. Given an OCR of 2, the Norwegian empirical relations are in between the direct and active shear strength from equation 3.10 and 3.11. A common OCR for the Gothenburg area is 1.3-1.5. If this is used in the Norwegian empirical relations and in equation 3.10, the three methods are similar but with lower values for the Norwegian equations.

This report focuses on direct shear strength in clays in the western part of Sweden and therefore only the empirical relations from SGI are considered. The corrected shear strength from vane tests is defined as direct shear strength.

The preconsolidation pressure used in these equations is evaluated from oedometer tests. Only the oedometer test of type Constant Rate of Strain (CRS) is discussed in this report. The method used today for evaluation of σ'_c includes a correction for the loading speed (Sällfors, 1975). It is based on a calibration down to 20 m depth and might not be sufficient for deeper levels, as σ'_c requires higher loading speed for great depths. For samples at greater depths, a friction can be developed between the soil sample and the oedometer ring. This can lead to an overestimation of σ'_c. A comparison between σ'_c evaluated from CRS tests and triaxial tests shows no difference between the test methods down to almost 30 m depth. Below that, the CRS tests give higher σ'_c than the triaxial test, and this difference increases with depth. The empirical relations below 30 m should therefore be used with caution.

Given a triaxial test, the direct shear strength can be estimated from back-calculations from the equations given below. The empirical relation is then independent of the preconsolidation pressure, but still requires a measured liquid limit.

$$\tau_{fu}^{D}(\tau_{fu}^{A}) \approx a_{Direct} \cdot \frac{\tau_{fu}^{A}}{a_{Active}} = \left(0.125 + \frac{0.205 \cdot w_L}{1.17}\right) \cdot \frac{\tau_{fu}^{A}}{0.33} \tag{3.13}$$

$$\tau_{fu}^{D}(\tau_{fu}^{P}) \approx a_{Direct} \cdot \frac{\tau_{fu}^{P}}{a_{Passive}} = \left(0.125 + \frac{0.205 \cdot w_L}{1.17}\right) \cdot \frac{\tau_{fu}^{P}}{\left(0.055 + \frac{0.275 \cdot w_L}{1.17}\right)} \tag{3.14}$$

where $\tau_{fu}^{D}(\tau_{fu}^{A})$ = undrained direct shear strength evaluated from undrained active triaxial tests

where $\tau_{fu}^{D}(\tau_{fu}^{P})$ = undrained direct shear strength evaluated from undrained passive triaxial tests

4 Determination of shear strength from tests

The shear strength can be measured either directly in the soil with a field test, or by taking samples and performing tests in a laboratory. There are mainly two methods in use today for measuring the shear strength in the field; vane test and CPT. The recorded values are adjusted with respect to the liquid limit and OCR. The sampling is performed either undisturbed to maintain the structure and the stress history of the soil or disturbed, where the actual soil content is of interest.

An experiment was performed in Ellingsrud, Norway, where the stability was calculated from field and laboratory tests separately. The factor of safety, F, was calculated for the most critical slip surface. The vane tests were corrected according to Bjerrum's method which does not differ substantially from the present SGI correction. The section was then loaded until failure two year later, and back-calculations showed the actual factor of safety differed from the expected conditions (Karlsrud, 2010);

$F_{\min}$ = 1.12 for triaxial and direct shear tests

$F_{\min}$ = 0.57 for vane tests

$F_{failure}$ = 0.87-1.09

This shows that the choice of investigation method is crucial for the calculation result. As seen from the back-calculations, the actual factor of safety does not correspond with any of the predicted. Triaxial and direct shear tests are believed to simulate the shear strength better than field tests, as the test procedure is more controlled in the laboratory for undisturbed samples.

Undisturbed sampling in Sweden is usually performed with a piston sampler which has a standardized sample diameter of 50 mm (Bergdahl, 1984). One sampling provides three plastic tubes with soil, each 17 cm. The middle tube and the upper part of the lower tube are generally considered to be the least disturbed, while the upper tube is used for index testing (Sällfors, 2001). The entire width of the sample is used in the tests. A method that has shown a very good quality, but is less used, is the block sampling. A block of 250 mm diameter and 350 mm height is cut from the soil. Smaller samples are then cut in the laboratory from the core of block in order to preserve the in situ situation. This is especially of interest for sensitive clays (Sandven, u.d.).

When comparing results in Norway from block samples with 54 mm and 75 piston samples for direct shear tests, the block samples show higher shear strength than the piston samples. Triaxial tests for active and passive shear strength show the same tendencies, τ_{fu}^{A} being 10-50% higher and τ_{fu}^{P} 0-10% higher for block samples. The time to failure in the test also influences the shear strength. With a reference time of 140 min, it is clear that the measured shear strength in laboratory tests varies with the loading time to failure (Karlsrud, 2010);

$$\tau_{fu,t=15\sec} \approx 1.35 \cdot \tau_{fu,t=140\min}$$

$$\tau_{fu,t=2months} \approx 0.85 \cdot \tau_{fu,t=140\min}$$

4.1 Vane test

The vane test is an in-situ test which means that it is performed directly in field. A vane of crossed steel plates is pressed down in the soil and the torque is measured when the vane is rotated. To be able to analyse clays with different shear strengths, there are four different sizes of vanes available, which all have the proportion H/D=2. There are two types of vane test equipment used in Sweden, with or without casing of the rod. The method with casing is troublesome to perform but the rod is covered which mean that all torque is created at the vane. The method without casing causes friction on the rod which gives a higher torque. A clutch on top of the vane is used to separate the torque generated in the vane (Larsson, et al., 2007). The two methods are displayed below in Figure 4.1.

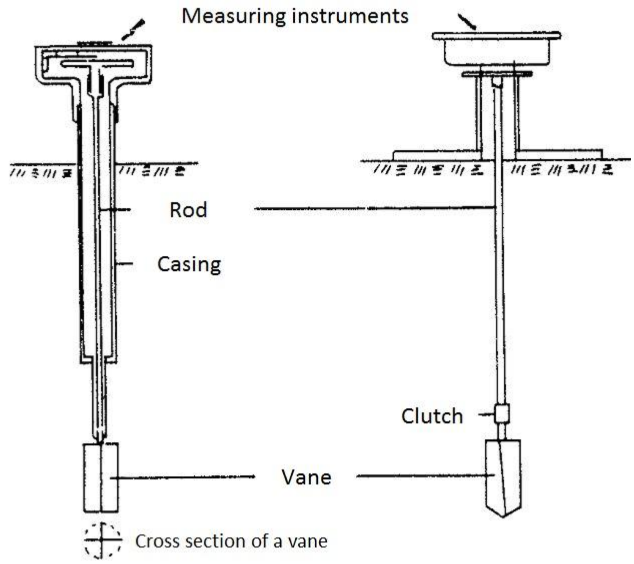


Figure 4.1 Sketch of vane test equipment.

To calculate the shear strength it is assumed that the vane creates a fully mobilised cylinder and the sleeve friction is constant during the test. Given the torque and surface area of the cylinder, equation 4.1 below give an average value of the uncorrected shear strength in the soil (Bergdahl, 1984).

$$\tau_v = \frac{6}{7} \cdot \frac{T}{\pi \cdot D^3} \tag{4.1}$$

where τ_v = uncorrected undrained shear strength

T = torque

D = diameter of the vane

When using values from vane tests to determine the shear strength in the soil, the liquid limit needs to be taken under consideration and the measured value needs to be corrected according to the SGI recommendation (equation 2.2 and 2.3).

When performing a vane test it is very important to follow the prescribed testing procedure. Research done in 1960 and beginning of 1970 show that the results differ depending on the rotation speed and the time between insertion and testing. When changing the rotation speed, short time to failure will result in high values of shear strength and a longer time to failure will give an impression of low shear strength. When changing the waiting time, measured values increase with time up to about 24 hours. This increase in strength is most likely due to reconsolidation of the soil close to the vane. To avoid diverging test data, the standard is set to wait 3-5 minutes before starting the test and to have 1-3 minutes testing phase before failure (Larsson, et al., 2007).

4.2 CPT

The Cone Penetration Test is a field test performed with a drill rig, where a rod with 60° angle is penetrating the soil with a constant rate of 20 mm per second (Larsson, 2007). It is shaped like a cone with a measuring device placed in the tip for recording the cone tip resistance. A sleeve is placed behind the cone to record the friction when pushing the rod down. The most popular method today records the pore pressure with a porous filter placed directly behind the cone; see Figure 4.2 (Sandven, u.d.). Swedish regulations require an inclinometer installed, to ensure the quality of the test. A computer is connected to the instrumentation in the cone and logs values for the four parameters every 2-2.5 cm.

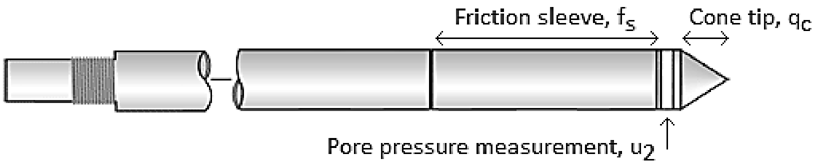


Figure 4.2 Principal sketch of CPT probe.

The recorded cone tip resistance includes the pore pressure measured in the tip, so this value needs to be corrected;

$$q_t = q_c + (1 - a_c)u_2 \tag{4.2}$$

where q_t = corrected cone tip resistance

q_c = uncorrected cone tip resistance

u_2 = pore pressure

a_c = area factor

There are two methods to evaluate the shear strength from the CPT. The commonly used method is based on q_t. The other method is based on the excess pore water pressure and is useful for situations with extremely loose NC clay, for instance a seabed (Larsson, et al., 2007). In loose NC clay the excess pore water pressure often has a higher accuracy compared to the cone tip resistance. The accuracy of the method is mainly dependent on precision used when the CPT was executed, but the estimation will be more precise with other soil properties defined. The most precise shear strength evaluation is dependent of preconsolidation pressure, vertical stress and liquid limit with the equation (Larsson, 2007)

$$\tau_{fu}^{CPTU} = \frac{q_t - \sigma_{v0}}{C} \cdot \left(\frac{OCR}{1.3}\right)^{-0.2} \tag{4.3}$$

where $C = 13.3 + 6.65 \cdot w_L$

The soil should be homogeneous to give a representative result. An inhomogeneous soil with cracks gives an overestimation of the shear strength, where the actual shear strength is about half of the calculated (Larsson, 2007). If the liquid limit is not given, a rougher estimation can be made, where C is replaced with the values given in Table 4.1.

Table 4.1 Value of the variable C for specific soil types.

Soil classification	C-value
Sulphide soil	20
Silt	14.5
Clay	16.3
Gyttja	24

4.3 Fall cone test

The fall cone test is a laboratory test that is used for determining the shear strength, sensitivity and liquid limit. The principle of the fall cone test is that a cone with a certain weight and angle of the tip is released into a soil sample. Depending on the strength of the sample, the angle of the tip can be chosen to 30° or 60° and the weight is chosen among 10g, 60g, 100g or 400g. When testing, a 15 mm slice of the undisturbed sample is placed under the cone and its tip is set to touch the top of the sample. The cone is then released and the penetration depth is measured from the scale. The test device is shown in Figure 4.3 below.

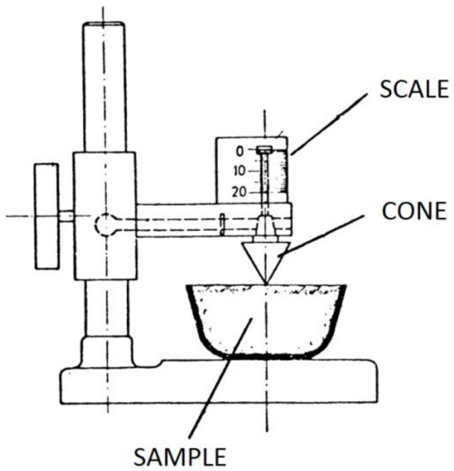


Figure 4.3 Conceptual model of a fall cone test device

The testing procedure is carried out three times and the mean value of the penetration is determined. When calculating the shear strength of the soil sample, equation 4.4 is used for the characteristic value of shear strength.

$$\tau_k = \frac{k \cdot Q \cdot g}{i^2} \tag{4.4}$$

where τ_k = uncorrected undrained shear strength

k = 0.25 for the 60° cone and 1.0 for the 30° cone

Q = mass of the cone (g)

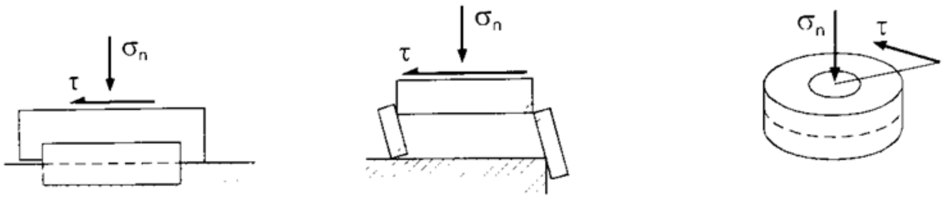
g = gravity

i = cone penetration in (mm)

The characteristic value for the shear strength, τ_k, needs to be corrected with respect to the liquid limit. This is made in the same way as for characteristic values from vane test, but the OCR is not included. The shear strength from fall cone test is therefore only corrected according to equation 2.2.

4.4 Direct shear test

Direct shear test is a laboratory test which measures the direct shear strength in a soil sample. The direct shear occurs between the active and the passive shearing zone, see Chapter 3.2.1 (SGF:s Laboratoriekommité, 2004). The test is executed under drained or undrained conditions and the load is applied stepwise, continuous or cyclic. The load can be applied along a predefined surface (A), along the entire sample height (B) or as a radial torque (C), see Figure 4.4.



(A) Direct shear box *(B) Simple shear* *(C) Ring shear*

Figure 4.4 Principle methods for different kinds of direct shear tests.

The direct simple shear test (B) is the most common test of the three. The sample has a height of 20 mm and a diameter of 50 mm. The procedure itself has two phases; consolidation phase and shearing phase.

The sample is first enclosed in a rubber membrane with saturated porous filter stones placed in both ends, which gives a two-way drainage. This is mounted in the apparatus, with the membrane tightened in top and bottom to have a watertight cell. A number of thin support-rings are placed over the membrane, with a distance-plate placed between each ring, see Figure 4.5. The soil is exposed to vertical stress in the consolidation phase and the distance-plates are removed as soon as the rings are fixed in place.

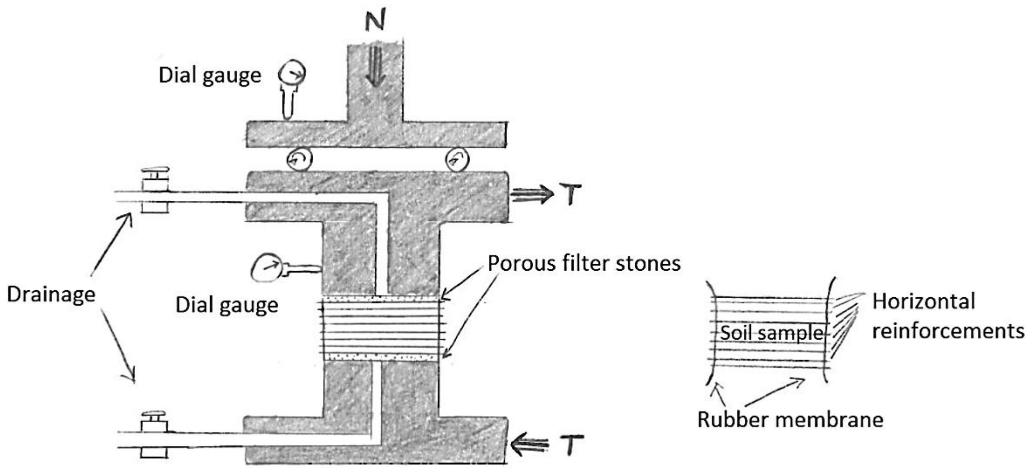


Figure 4.5 Principal sketch of direct simple shear apparatus and how the sample is mounted.

The consolidation phase means that the soil will be exposed to its previously known vertical stress, to better simulate the in situ-conditions in the soil. This is performed under drained conditions, where the excessive pore water will be lead out through the filter stones. The loading in this phase depends on the OCR of the sample. If it is an OC-clay, the sample will be loaded vertically to up to $0.85 \cdot \sigma'_c$. The sample is then unloaded until the in situ stress is reached. If the sample is an NC-clay, the sample will be loaded directly up to the vertical in situ stress (SGF:s Laboratoriekommité, 2004). A higher vertical stress can lead to an extra consolidation of the sample and give large deformations.

The consolidation phase is followed by the shearing phase. If the test is performed undrained, the drainage will be closed for this phase. A horizontal force is applied continuously or with given load increments and the horizontal deformation is recorded with a dial gauge, see Figure 4.5. According to Swedish standard, a stepwise loading is performed with $0.05 \cdot \sigma'_c$ per 30 minutes until the horizontal movement measures 0.025 radians. The load steps are then reduced to half with a consolidation time of 15 minutes.

The result is presented in a diagram over shear stress, τ, and shear strain, γ_{DS}. The shear stress is defined as the ratio of shear force, *T*, and cross-sectional area, *A*, see equation 4.5. The shear strain is a function of horizontal deformation, Δs, and sample height, *h*, presented in equation 4.6. The maximum value of the shear stress equals the shear strength of the sample.

$$\tau = \frac{T}{A} \tag{4.5}$$

$$\gamma_{DS} = \arctan\left(\frac{\Delta s}{h}\right) \text{ (radians)} \tag{4.6}$$

This is the only method to determine direct shear strength in the laboratory where the vertical in situ stress is taken into account. This gives a better simulation of the shear strength than the fall cone test, where only the present shear strength is measured. The drawbacks for this method are that the pore pressure is not measured in this test, neither the actual vertical load in the pedestal.

4.5 Triaxial test

The triaxial test is performed on a soil sample and simulates the in-situ stresses in three dimensions. The specimen is normally cylindrical, meaning that both horizontal stresses are equal. The test is performed on undisturbed samples, either under drained or undrained conditions. The test result provide data of the total stress in each direction, pore pressure and deformation.

The cylindrical sample is first encircled with a filter paper to allow radial drainage. (Janbu, 1973) A saturated, porous filter stone is placed on the pedestal in the triaxial cell and the sample is placed on top of that, see Figure 4.6. The same kind of filter stone is placed on top of the sample, with a cap on top of it. The specimen, the filter stones, the pedestal and the top cap are covered with an impermeable rubber membrane. Two O-rings are placed on each end to keep it watertight. An acrylic glass cylinder is placed over the entire sample, mounted and tightened in place with screws, see Figure 4.6. (Sandven, u.d.) The triaxial cell is then filled with paraffin oil to be able to create the cell pressure (Hedborg, 2012).

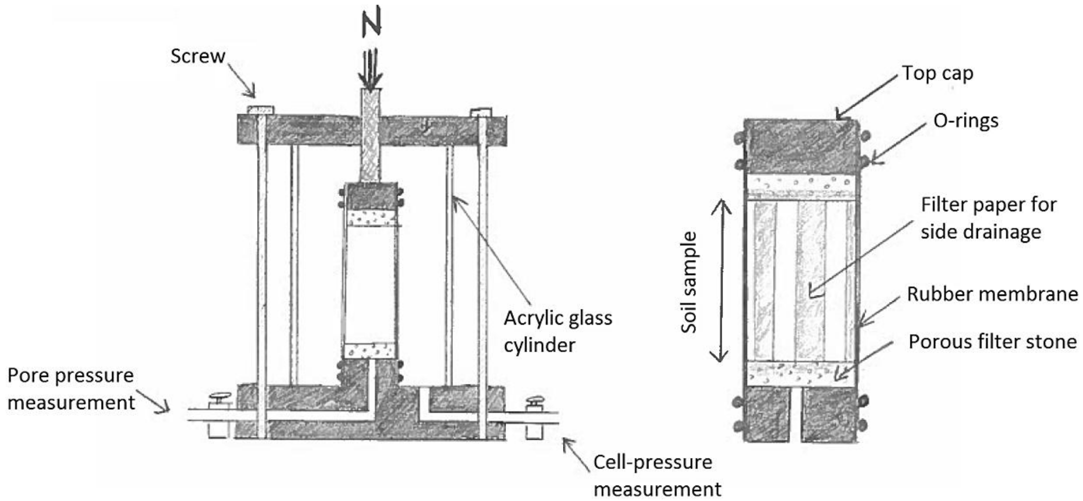


Figure 4.6 Cross-section of triaxial cell with mounted soil sample.

The test is performed in two phases; consolidation phase and shearing phase. In the consolidation phase, the specimen is loaded under drained conditions until the in situ stresses σ'_{v0} and $\sigma'_{h0} = K_0 \cdot \sigma'_{v0}$ are reached (Kompetenscentrum, u.d.). The vertical stress is applied with a loading piston on the top cap and the horizontal stress is applied by increasing the cell pressure in the triaxial cell, see Figure 4.6. It is thereafter left with this stress to consolidate before shearing phase. The consolidation phase takes approximately 24 hours.

The shearing phase is where the sample will be loaded to failure and is performed either drained (open drainage tubes from the sample) or undrained (drainage tubes are closed). Both cases require fully saturated tubes and filter stones. That is necessary in drained conditions to get a corrected measurement of the expelled pore water. In undrained conditions, the pore pressure will increase with the loading and the volume is kept constant. If there are air voids in the measuring device, they will be compacted and a result in a volume change. The loading in drained conditions must be performed without creating any excessive pore pressure.

The test is normally performed to evaluate the shear strength in active or passive shearing zone, see Chapter 3. The horizontal, radial stress, $\sigma_2 = \sigma_3$, is kept constant in both cases. A test of the active shear strength is where the specimen is loaded vertically until failure, also called compression test. The passive test is an indirect tension test, where the vertical stress is decreased until failure, also called extension test (Sandven, u.d.). How the triaxial cell is adjusted for each test is presented in Table 4.2.

Table 4.2 Summary of test procedures for triaxial tests.

Test procedure	σ_1	σ_3	Pore pressure tubes	Volume
Active undrained test	Increase	Constant	Closed	Constant
Passive undrained test	Decrease	Constant	Closed	Constant
Active drained test	Increase	Constant	Open	Changes
Passive drained test	Decrease	Constant	Open	Changes

4.5.1 Results and interpretation

The result from the test is presented in a $\tau_{fu} - \sigma'_{mean}$ diagram where every point is plotted as a Mohr circle in a Mohr-Coulomb diagram, see Figure 4.7 (Kompetenscentrum, u.d.). The curve is called Effective Stress Path (ESP) and is directly correlated to the yield envelope.

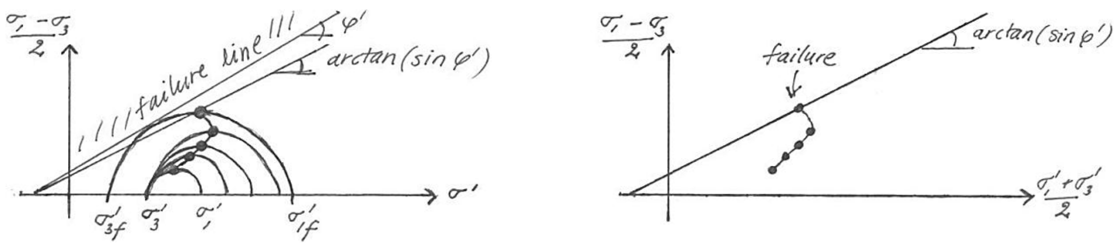


Figure 4.7 Mohr-Coulomb diagram for a consolidated, undrained active triaxial test and conversion in to a diagram for corresponding ESP.

The shearing phase is presented in the left figure. In this test, the horizontal total stress is constant, while σ_1 is increased. Mohr circle is therefore increasing, with the centre point moving to the right. The pore pressure is only slightly affected, as it is only small strains within the yield envelope. In theory, large deformations would start to occur when Mohr's circle reaches the σ'_c-line in the yield envelope. But the sample is fixed in the triaxial cell with closed drainage channels and cannot get deformations. The volume change is therefore taken by an excessive pore pressure. That makes the effective stresses σ'_1 and σ'_3 decrease and centre of Mohr circle is moved to the left, towards the failure line.

The shape of the ESP can be interpreted to obtain friction angle, attraction and dilatancy. For undrained tests, the maximum shear strength in the graph represents the shear strength of the sample for active or passive conditions.

An undrained triaxial test for NC-clay reaches the σ'_c-line before failure. The preconsolidation pressure can be interpreted graphically from the ESP, see case 2 in Figure 4.8. The definition of OC-clay is that the in situ stresses are small in comparison with σ'_c. If the OC-clay is consolidated to the in situ stresses in the triaxial test, the ESP might not reach the σ'_c-line before failure, as case 1 in Figure 4.8. In this figure, the ESP for case 1 goes downwards parallel to the failure line. This shows that the material is contractant. The ESP for OC-clay can also at first go upwards parallel to the failure line, if the clay is dilatant.

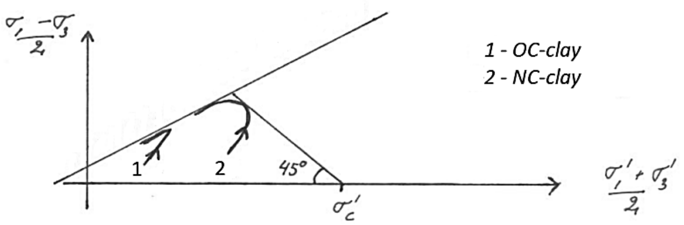


Figure 4.8 Two clays consolidated to in situ stresses.

The evaluated active shear strength differs between NC-clay and OC-clay. The difference is directly correlated with the OCR, as presented in the empirical relation;

$$OCR < 1.5 \Rightarrow \tau_{fu}^{A} = \sigma'_c \cdot 0.33 \tag{4.7}$$

$$OCR \geq 1.5 \Rightarrow \tau_{fu}^{A} = \sigma'_c \cdot 0.33 \cdot OCR^{-0.2} \tag{4.8}$$

This means that case 1 and case 2 both give the highest measured active shear strength in a triaxial test, but the OC-clay has lower value, due to the OCR. Figure 4.9 shows examples of empirical active shear strength based on the equations above. The triaxial test done on clay with low OCR aligns with the empirical relation from equation 4.7. The triaxial test done on clay with high OCR deviates from that line and aligns better with the empirical relation for OCR=2. This needs to be taken into consideration when comparing triaxial tests with the empirical relations.

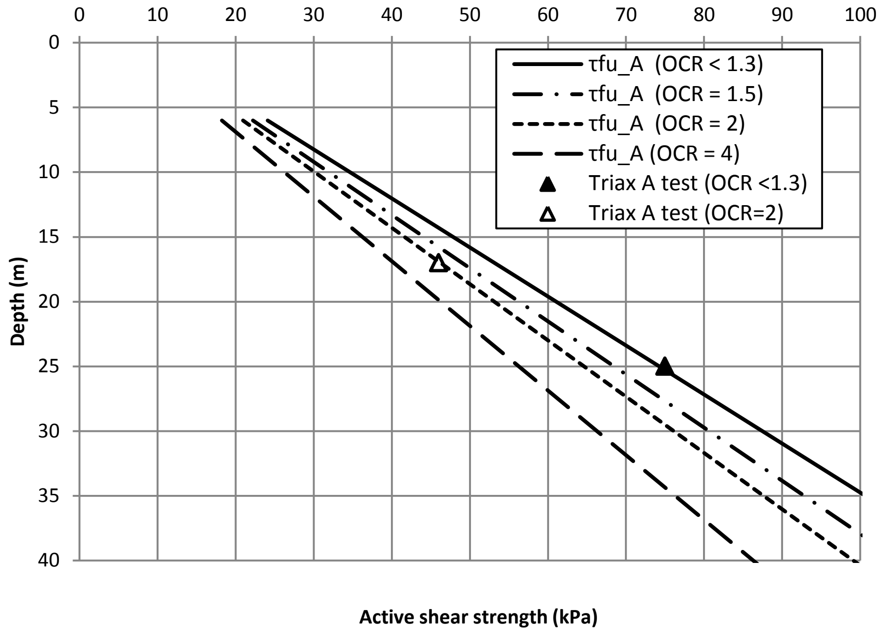


Figure 4.9 Empirical relations for active shear strength with two active triaxial tests.

In this report, most of the clays are normal or lightly overconsolidated. Equation 4.7 is therefore used as the empirical relation for comparing with active triaxial tests. Triaxial tests performed on overconsolidated clay are placed in brackets, as they might not be comparable with the equation 4.7.

5 Determination of shear strength for a location

5.1 Method for a random location

When determining the shear strength for a location it is important to get a general understanding of the geology in the region where the site is located. Geological maps are an instrument to identify the general soil type in the area and to give an indication of what geological history the soil has been exposed to. Slopes and valleys can origin from erosion or a bedrock valley filled up with sediments. The soil will have different stress situations and ground water flows depending on how the soil layers have been formed on top of the bedrock. The most significant difference between the two sedimentation processes is how the shear strength is increasing; either with depth or with elevation. If the location is in an urban area, the soil can also have an artificial filling added on top, which usually can be detected from city maps and documentation.

A rough estimation of the shear strength for an area can be done with empirical relations. By assuming the unit weight, OCR and liquid limit of the soil, an approximate value of the shear strength can be calculated using equation 5.1. This will give an indication of where or if further investigation is needed, depending on the geotechnical challenge at hand.

$$\tau_{fu}^{D} \approx \left(0.125 + \frac{0.205 \cdot w_L}{1.17}\right) \cdot \sigma_c' \cdot OCR^{-0.2} \tag{5.1}$$

where $\tau_{fu}^{D} =$ direct shear strength

$w_L =$ liquid limit

$\sigma_c' =$ preconsolidation pressure

$OCR =$ Over Consolidation Ratio

If the soil is known to be clay in western Sweden, the unit weight is typically 15 to 16 kN/m^3, marine clays along the west coast of Sweden normally has OCR $\geq$ 1.3 and the rule of thumb for marine clays in the Gothenburg region is $w_L = 75\%$. When using these parameters, a simplified equation can be used for approximation of the shear strength;

$$\tau_{fu}^{D} \approx \frac{\sigma_c'}{4} \tag{5.2}$$

When evaluating the need of geotechnical tests, routine tests should confirm whether the typical values for Gothenburg are applicable for this location or not. More tests should be performed in areas that have a low factor of safety or a high uncertainty. This could for instance be where the elevation alters or where the geological history is unclear. When evaluating and comparing test results, it is important to take into account that samples might be disturbed. This can be caused when collecting or handling samples both in field and in the laboratory. The test procedure both in field and laboratory might give unexpected test results if not properly performed. It is therefore essential to examine results from field measurements and make sure that the tests have been executed according to Swedish standards. Results from several boreholes must be considered to find a trend to rely on rather than specific values. To get a general idea of the shear strength for the location, test results should be compared with each other, even if the test method differs. For instance, σ_c' from a CRS test can be compared with σ_c' obtained from a yield envelope in an undrained triaxial test, see Chapter 3.2.3.

5.2 Working procedure on studied locations

All seven locations in this report have been analysed with the same procedure. Different techniques have been used when determining the soil properties through the profile. For the density, given values from laboratory investigation have been used. If more than one value is given for the same level, an average is calculated. To be able to calculate the in situ stress for each metre, the density between given values are interpolated. The method with interpolation is also used for the pore pressure when the measurements differ from a hydrostatic situation. For the liquid limit, we have chosen to use intervals with the same value through the profile. This method is used to better follow the trend of the liquid limit and avoid local deviations in single tests.

When determining our best estimated shear strength for each location, we have chosen to plot values from direct shear tests, triaxial tests and the empirical relation from σ_c'. To be able to show a justified comparison, values are normalized with respect to the direct shearing phase. The active and passive triaxial tests are therefore calculated with equation 3.13 and 3.14 and are displayed as; "Empirical f(triax A)" and "Empirical f(triax P)". To determine the empirical relation from σ_c', the preconsolidation pressure from boreholes in the area are plotted and a trend is evaluated. This trend of σ_c' together with the calculated in situ stress gives OCR on each level. σ_c' is then used in equation 3.11 to get an empirical value for the corresponding direct shear strength which is presented as "Empirical f(σ_c')".

To display the relation with the active shearing phase, the active triaxial test results (Triax active) are also plotted. These values can be compared with the empirical line for active shear strength calculated with equation 3.10 using σ_c'. This empirical line is displayed in the figures as; "Empirical active f(σ_c')". For all studied locations, values are presented according to their elevation; metres above sea level.

The purpose when presenting these values is to get a picture approximately of what direct shear strength the laboratory tests indicate. By making a best estimated trendline for the direct shear strength obtained from laboratory tests, we can compare these to results from vane and fall cone tests. The vane and fall cone tests correspond approximately to the direct shear strength of the soil, so the result should be very similar to the best estimated shear trend. For this trendline, we have added dotted boundaries for ±10% due to the scatter of test results.

The best estimated trendline from laboratory tests is then plotted together with corrected values from vane and fall cone tests to show how they correlate. These corrected values form a scatter of points which can deviate due to different source of errors. To get a general understanding for what shear strength the vane and fall cone tests give for the location, a trendline is drawn for the vane tests and one for the fall cone tests. When making the trendline, values that differ significantly are excluded. These values are marked grey and presented as "deleted values". The same procedure is followed for the uncorrected values to show the trend without the correction.

To make it easier to follow the presented data in the figures, a template for the symbols is used. Direct shear tests, vane tests and fall cone tests are displayed with different colours but that code is the same for each borehole. The template is presented in the Figure 5.1 below with a description on the right hand side.

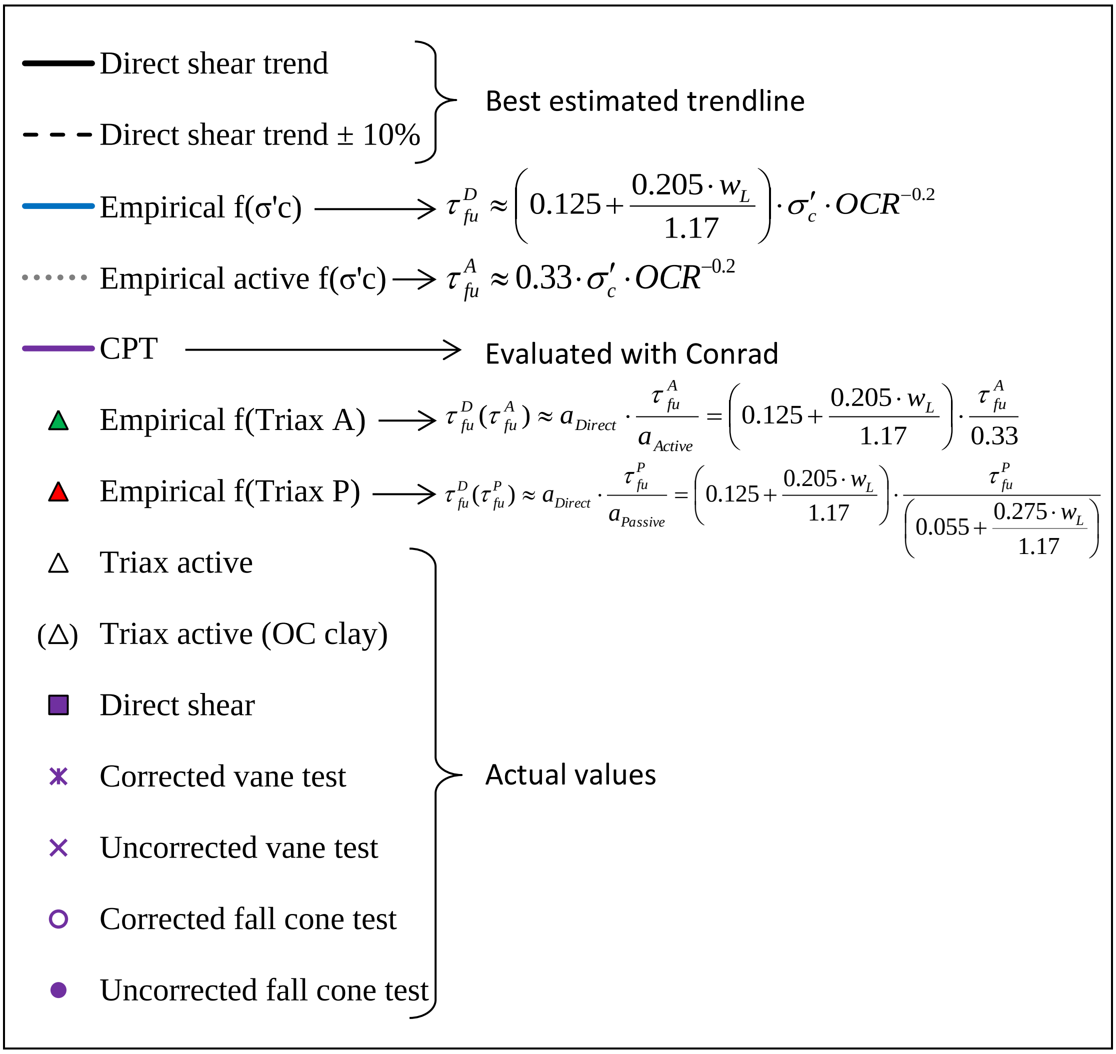


Figure 5.1 Template for presentation of data.

6 Presentation of locations

This chapter presents a summary of the geological and geotechnical conditions for the seven locations investigated. It presents two locations in Gothenburg city followed by four locations along the Göta River, presented from south to north. The most northern location is situated north of Lilla Edet, by a small creek that has its discharge in the Göta River. The seventh location is situated east of Gothenburg city, by the creek Kvibergsbäcken. This creek is connected to Säveån approximately 4 km east of Göta River. The locations are chosen to represent the clay in Gothenburg and in the surrounding region around Göta River. The shear strength is presented graphically in the report, with enlarged graphs in each appendix. All piston sampling described was done with standard piston sampler of type St II.

6.1 Gothenburg Central Station

The central station is located in the centre of Gothenburg, in an area where many infrastructural projects have been performed. The Gothenburg Central Station will be expanded with railway as a part of the infrastructural project Västlänken. The nearby area was recently excavated when building an entrance to the road tunnel Göta Tunnel. A property close to the Central Station and this tunnel was investigated for a new building, called Regionens Hus. Geotechnical investigations have been done for all these locations, of which we have taken part of some of the results. The collected data was performed by more than three companies on different occasions.

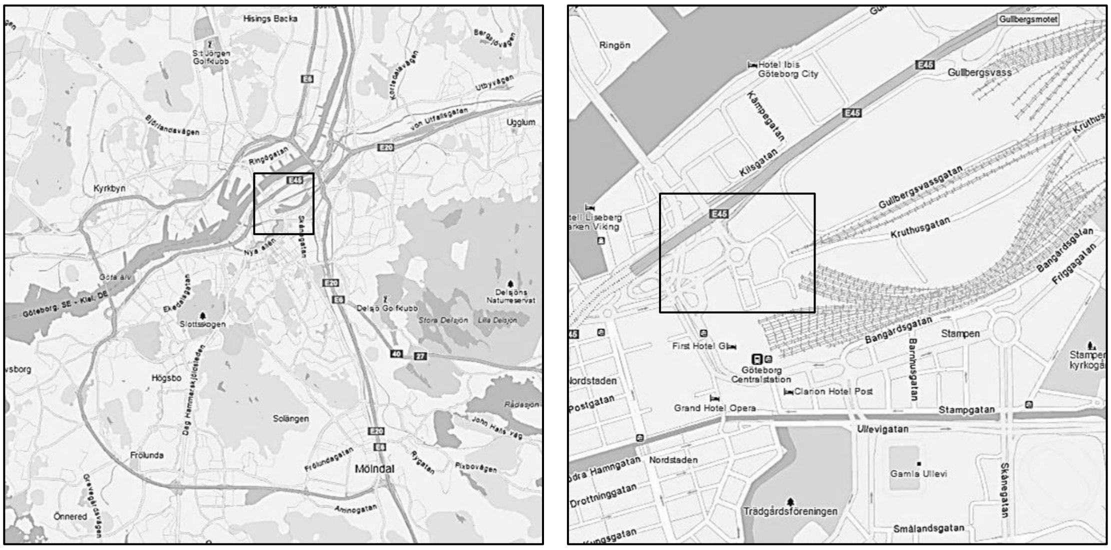


Figure 6.1 Map of the Central Stations position (borehole plan is presented in appendix 1)

6.1.1 Site investigation

The area is flat and the surface on which the investigations have been done have a maximum height difference of 1.2 m. The area spreads over 350 × 400 m, approximately 500 m from Göta River. It includes the Central Station, the northern entrance to the Göta Tunnel and the southern connection of Göta Älv Bridge, see Appendix 1 for borehole plan. The entire area consists of clay with filling on top. The pore pressure was measured at borehole 50001 at three depths. It is estimated to be hydrostatic with the ground water table at level 10.4 m and have a slight artesian pressure at deeper levels. The deepest measurement, at level -37.7 m, has two recordings. One of them is not considered here, as it showed a pore overpressure corresponding to a phreatic level 6 m above surface. The liquid limit is evaluated from laboratory tests done on two boreholes by the Central Station.

The top metres were investigated with helical auger. The soil consists of clay, according to measurements down to 90 m depths. We have chosen to only present results down to 50 m depth. Deeper measurements have higher probability of sample disturbance or disturbance on the measuring equipment.

The clay is medium sensitive, with values varying between 8 and 16. It is a slightly more sensitive between level 7 m and -3 m, where the sensitivity is 16 to 26. The layering and the chosen pore pressure are presented in Table 6.1.

Table 6.1 Soil layering and summary of soil properties for Gothenburg Central Station.

Level (m)	Material	ρ (t/m^3)	w_L (%)	Δu (kPa/m)
12 – 9	Mixed fill and gravelly sand	No data		0
9 – 6	Silty clay	1.60 – 1.65	70 – 75	10
6 – 0	Sulphide-bearing, silty clay	1.57 – 1.62	70 – 75	10
0 – (-3)	Sulphide-bearing, silty clay	1.61 – 1.63	65	10.65
(-3) – (-13)	Sulphide-bearing, silty clay	1.58 – 1.66	75	10.65
(-13) – (-38)	Sulphide-bearing, silty clay	1.62 – 1.66	75	10.22

The preconsolidation pressure is evaluated from 28 CRS tests taken in three boreholes, which give an OCR of 1.3 – 1.4 over all depths. The preconsolidation pressure is also evaluated from the failure envelope obtained in the active triaxial tests. As seen in the graph in Appendix 1, the two methods correspond well with each other and a trendline is drawn by means of linear regression.

There are six CPT tests performed, which have been evaluated with the software CONRAD. They all have sounding class CPT2, except for borehole 7, which has CPT1. In addition to this, there are vane tests performed in 10 boreholes and fall cone tests in four boreholes. All tests used in our estimation of the shear strength are presented in Table 6.2.

Table 6.2 Number of tests used in the evaluation of best estimated shear strength.

Laboratory method	Number of tests	Number of boreholes
Active, undrained triaxial test	4	1
Passive, undrained triaxial test	3	
CRS test	7+7+14	3
DS test	4	1
CPT	-	6

6.1.2 Presentation of test data

The calculated shear strength for each of the six CPTs is presented in Appendix 1. The average of these tests is also presented in the Figure 6.2 with the other test methods. The figure shows that the shear strength evaluated from CPT is substantially lower than the other test results. The low values can be a consequence of using different input data than for the other evaluations. The tests were performed close to the Central Station and the calculations were based on a hydrostatic level at +11.1 m, which differs from the three pore pressure measurements performed in borehole 50001. The calculation is based on both density and liquid limit, so differing values affect the result. Another source of error for the low CPT can have been caused when performing the tests.

Our best estimated trendline for shear strength is a linear regression of the empirical f(triax A), f(triax P) and the DS tests, except for the deepest measurement of DS. This specific measurement is lower than a linear trend for the triaxial tests. As the triaxial test simulates the in-situ conditions better than the DS test, we choose to disregard the deepest DS test. The linear regression for these tests has a similar trend as the empirical f(σ_c')-line. It is based on OCR and w_L, which is why there is a vertical line on shallow depth, due to high OCR, and a bend at level -3 m, due to a shift in liquid limit.

There are four active triaxial tests performed for the location. These are plotted as open triangles with the empirical line for active shear strength based on σ_c'. Three of the tests follow the empirical active f(σ_c')-line, while the deepest differs 10 kPa. This shows that the empirical relation between preconsolidation pressure and shear strength corresponds well for this location.

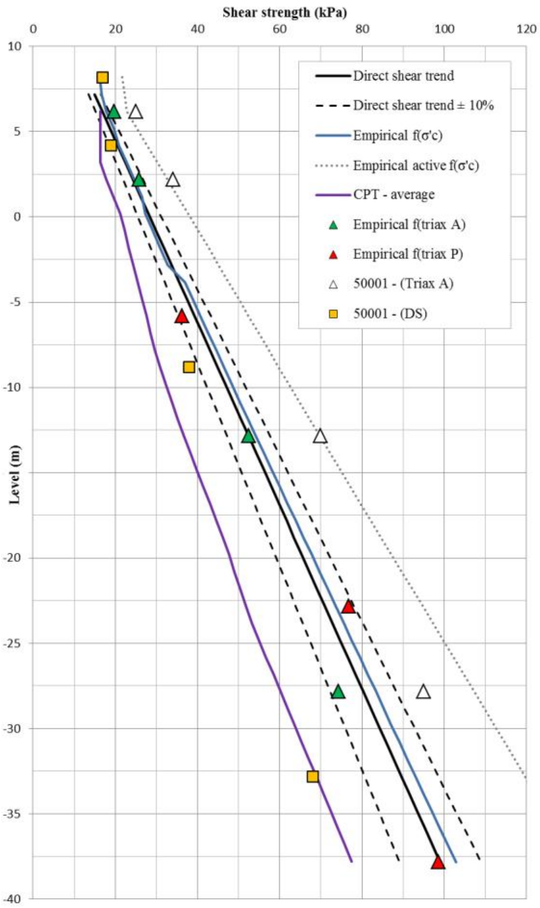


Figure 6.2 Best estimated shear strength from laboratory tests (for explanation of points and lines see figure 5.1).

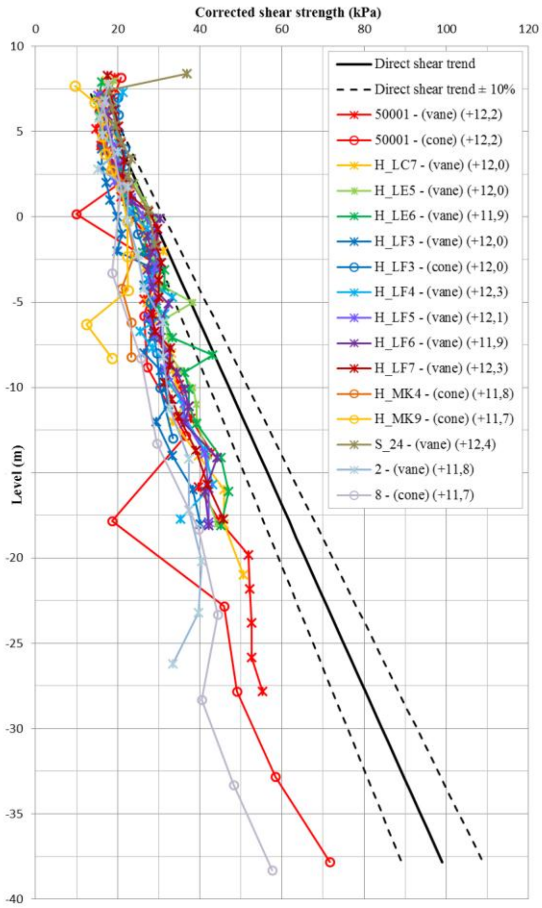


Figure 6.3 Corrected shear strength from vane tests and fall cone tests.

The trendline for best estimated shear strength is placed in the same diagram as all vane tests and fall cone tests in Figure 6.3. The results from different boreholes align with each other, with some of the fall cone tests slightly lower. The top metres are within the ±10%-margin, but there are only two values below level -4 m that are within the range. That corresponds to a depth of 15 m.

All vane tests and fall cone tests below the top five metres are presented in Figure 6.4 and Figure 6.5. There are results from four fall cone test that are much lower than the others, which we have deleted in the further evaluation, marked “Deleted values”. When creating the trendline we have taken an average for each method to fit the tendency of the values with equal weight on each of them.

The trendline for corrected vane test has the same inclination as the best estimate-trendline down to the level 0 m in Figure 6.4 and it has a lower inclination below that level. The trendline for corrected fall cone tests is outside the ±10%-margin for all levels.

For the uncorrected values in Figure 6.5, the trendline for vane tests is above our best estimate-line down to the level -2 m. This corresponds to a depth of 14 m. The trendline is within the ±10%-margin between level -2 m and -16 m. The corresponding line for fall cone test is within the ±10%-margin down to the level -7.5 m. The trendline for vane tests is lower than the best estimated trendline -10% from level -16 m and the trendline for fall cone tests is lower from level -7.5 m.

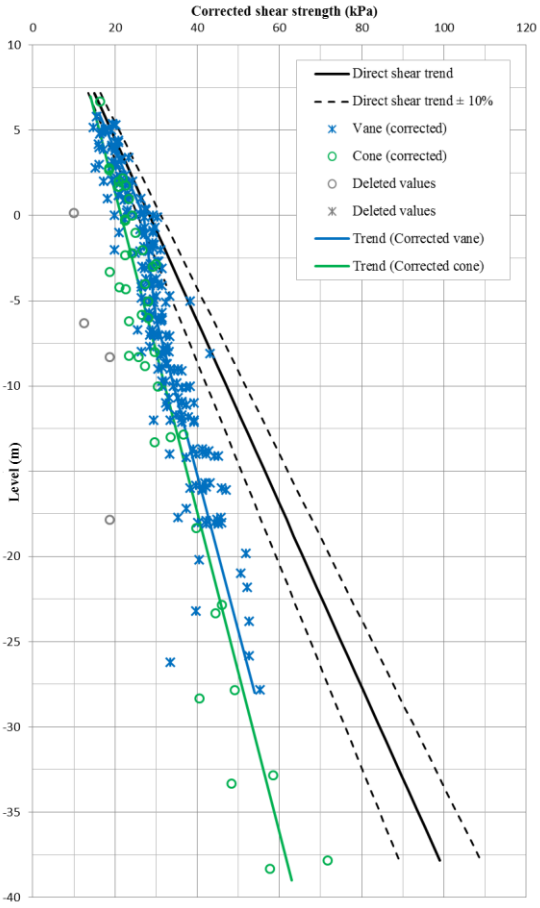


Figure 6.4 Corrected shear strength with linear trend.

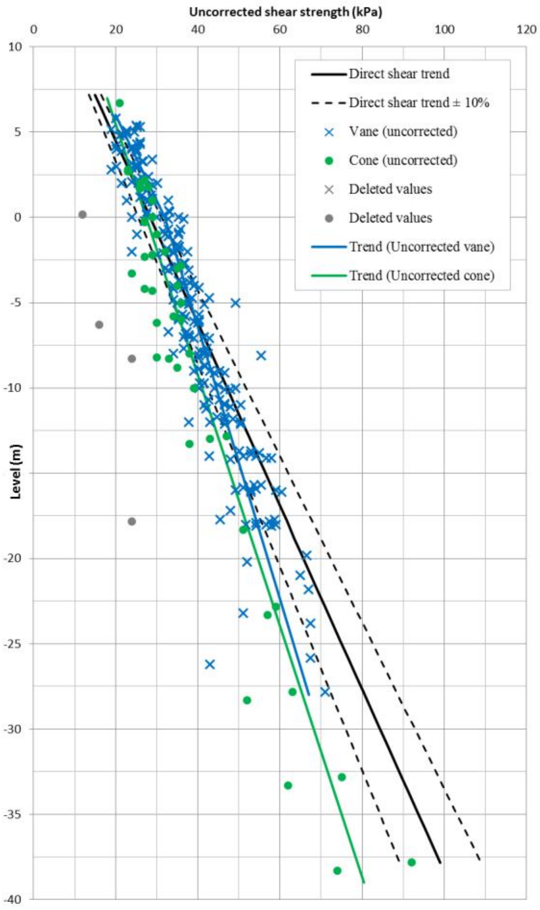


Figure 6.5 Uncorrected shear strength with linear trend.

6.2 Casino Cosmopol

Casino Cosmopol is, just as the previous location, investigated as part of the infrastructural project Västlänken, where a railway tunnel is planned to be built. It is located by the riverside of Göta River on mainland in the centre of Gothenburg, just outside Casino Cosmopol. The area is flat and consists of clay with filling on the top metres. This part of the harbour has previously been subject to filling and loading.

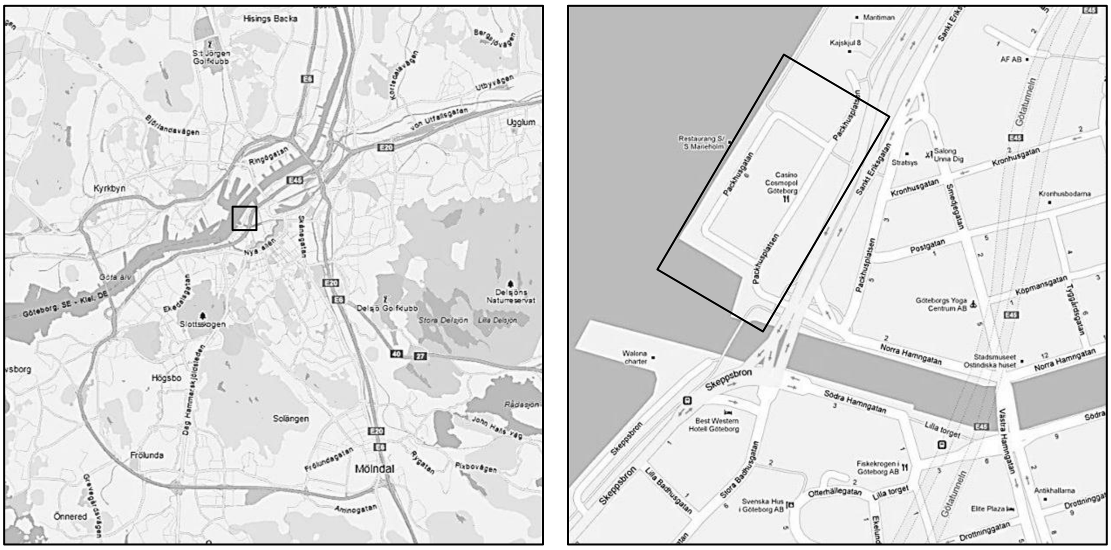


Figure 6.6 Map of Casino Cosmopols position (borehole plan is presented in appendix 2)

6.2.1 Site investigation

The boreholes taken into account in this report are located within an area of 150 × 50 m and have a height difference of maximum 0.6 m, borehole plan can be viewed in Appendix 2. There are three pore pressure measurements, located at borehole 08001, measured in spring and autumn 2005. We have chosen to interpolate the mean value from each recorded level, with the water level starting one metre below surface. The pore pressure is assumed to be hydrostatic at deeper and shallower depths than the measurements, see Table 6.3. The pore pressure at level 1.4 m and level -18.6 m are each an average of two recordings. The value at level -8.6 m is a single recording, as the other measurement showed a high artesian pressure.

The layering is determined from borehole 08001, where helical auger is used down to three metres depths and piston sampling below that. Based on the soil samples, the clay is low to medium sensitive, with no values exceeding 20.

Table 6.3 Soil layering and summary of soil properties for Casino Cosmopol.

Level (m)	Material	ρ (t/m^3)	w_L (%)	Δu (kPa/m)
11.4 – 9.4	Fill and gravelly sand	*No data*		0
9.4 – 8.4	Clay with organic material			10
7.8 – 1.4	Shell-bearing silty clay	1.67 – 1.77	70	10
1.4 – (-3.6)		1.61 – 1.70	75	7.9
(-3.6) – (-8.6)	Sulphide-bearing silty clay	1.62	75-78	7.9
(-8.6) – (-18.6)		1.60 – 1.63	78	14.75
(-18.6) – (-28.6)		1.63 – 1.65	75	10

Two CPTs were done for the location, which both show a general linear tendency over depth, see Appendix 2. They show that the clay contains more silt and have a thin sand layer between level 1.4 m and -3.6 m. The pore pressure drops slightly below level -3.6 m and increases again at level -13.6 m in borehole 08001. This aligns with the results from the pore pressure measurements. The most recent CPT test, in borehole CH5001, shows a constant Δu down to level -13.6 m and an increase again below that. The tests presented in Table 6.4 are evaluated to give our best estimate shear strength. Only the latest CPT is considered here. These are then compared with vane tests taken from five boreholes and fall cone tests taken from four boreholes.

Table 6.4 Number of tests used in evaluation of best estimated shear strength.

Test method	Number of tests	Number of boreholes
CRS test	7+7+3	3
DS test	4	1
CPT	-	1

6.2.2 Presentation of test data

The preconsolidation pressure is determined with linear regression for all CRS tests, see Appendix 2. The effective stress is also presented in this graph, which does not have a linear tendency over the entire depth. That is due to the shifting pore pressure.

Our best estimated shear strength is presented in Figure 6.7 and is a linear regression of the DS tests. The empirical f(σ_c')-line is higher than the measured values for the deeper measurements, which could origin from the CRS test procedure. The test result for the DS test at level 3.4 m did not show a distinct failure, which could indicate sample disturbance.

Figure 6.8 shows all the corrected vane tests and fall cone tests compared with our best estimated shear strength. The measurements are scattered on both sides of the trendline down to level 0 m. There are no values within the ±10%-margin below level -5 m; all values are below. The fall cone test at level -13.6 m and at +3 m is deleted in the further evaluation, as it is faulty.

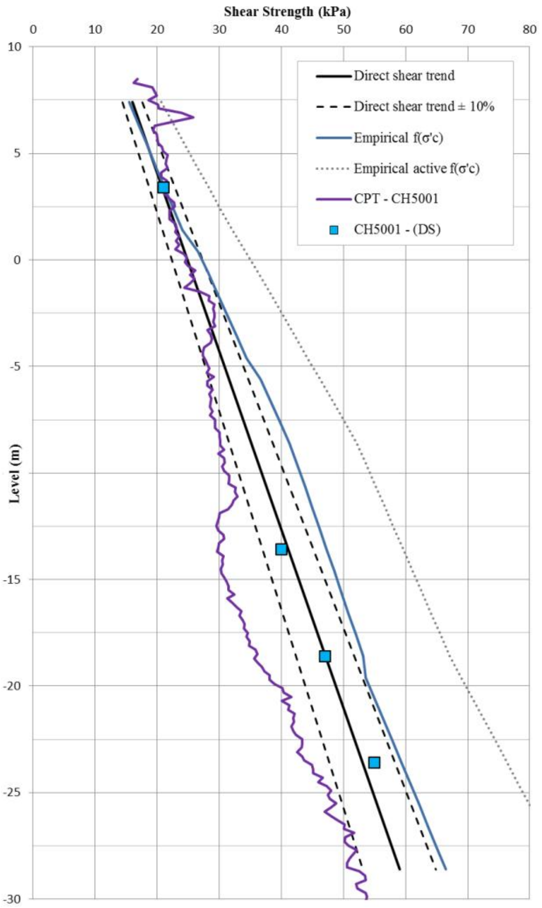


Figure 6.7 Best estimate shear strength from laboratory tests (for explanation of points and lines see figure 5.1).

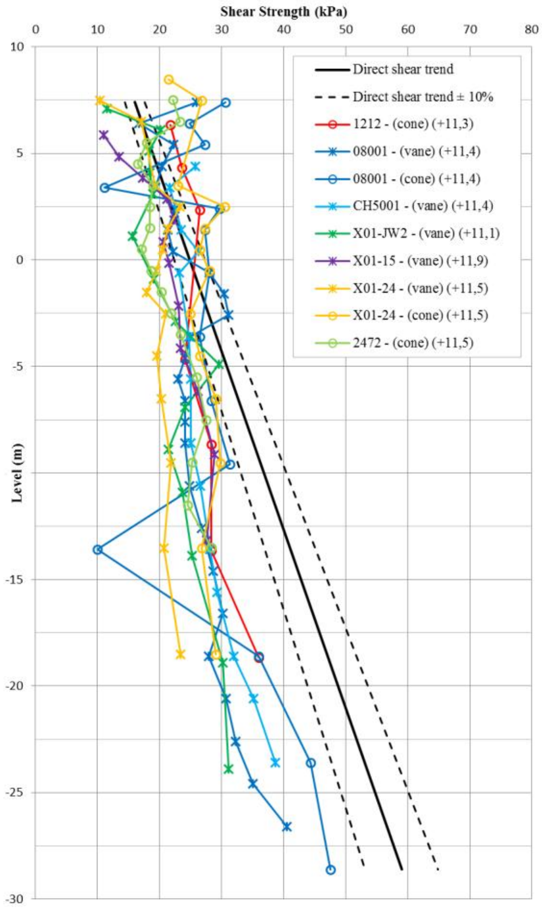


Figure 6.8 Corrected shear strength from vane tests and fall cone tests.

A linear trend for each test method is presented in Figure 6.9. The top five metres are not considered here, as they contain fillings and are not representative for the clay deeper down. The trendlines for both methods are within the ±10%-margin down to the level -1 m, at 12.5 m depth. At deeper levels than that, the fall cone tests have higher shear strength than the vane tests. The trend for vane tests shows a shear strength increase similar to our estimated trendline below level -11 m. The deepest value within the ±10%-margin is at level -5 m for vane tests and level -6.6 m for fall cone tests.

Figure 6.10 show that there are values for uncorrected vane tests of more than 10% higher than our best estimated trendline down to level -5 m. There are values on both side of the ±10%-margin down to that level. This corresponds to 16 m depth, and below that level, the vane tests are within or below the ±10%-margin. The fall cone test show values within or above the ±10%-margin down to the level -9.6 m, a depth of 21 m. Three of the values are below the margin below that and five are within. This gives a trend within our best estimated trendline for shear strength.

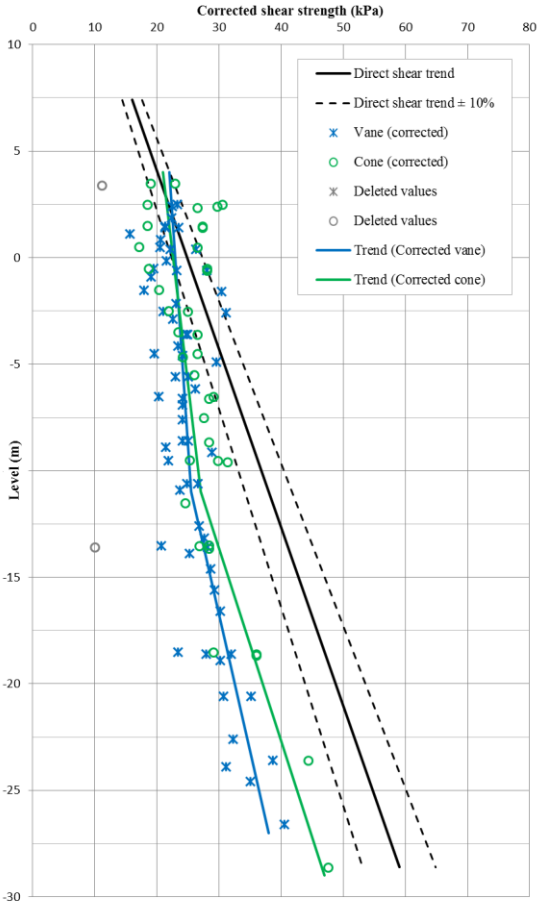


Figure 6.9 Corrected shear strength with trendlines.

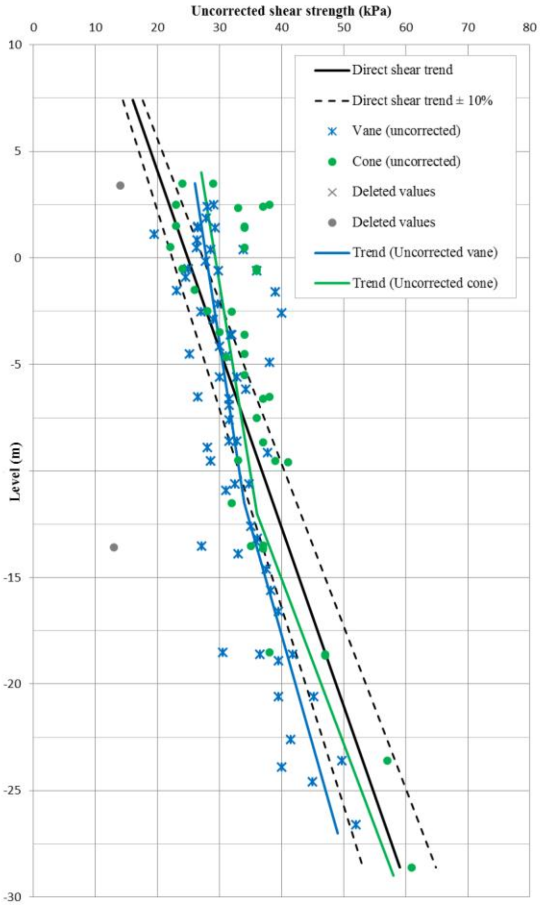


Figure 6.10 Uncorrected shear strength with trendlines.

6.3 Marieholm Tunnel

A road tunnel below Göta River is planned to be built in the northern part of Gothenburg city to connect the island Hisingen with mainland. The tunnel stretches 500 m and will be built approximately 600 m north of the existing Tingstad Tunnel. The waterfront is a flat area, which consists of clay with a thick layer of mixed filling on top.

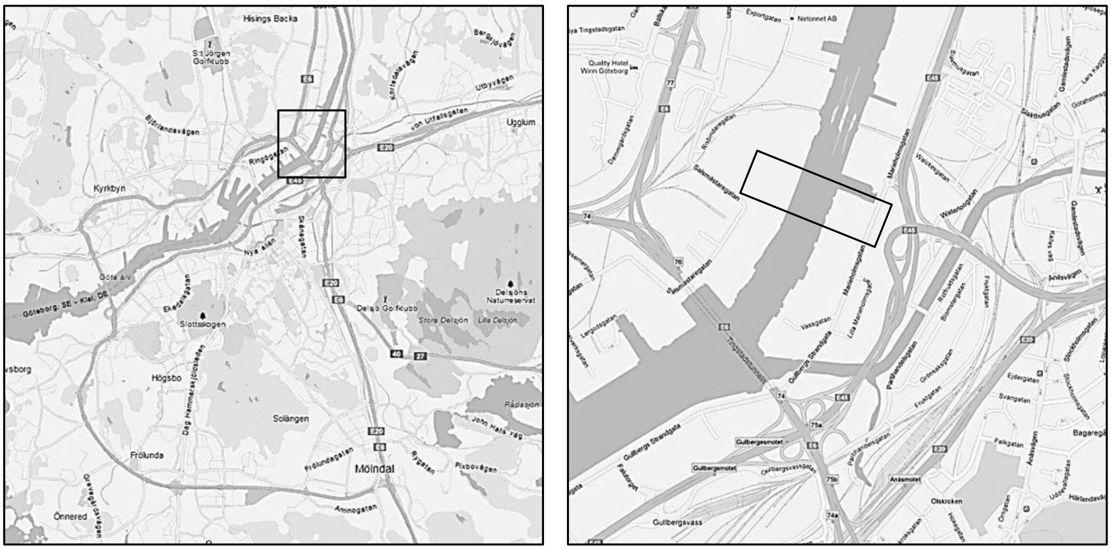


Figure 6.11 Map of Mariehom Tunnels positon (borehole plan is presented in appendix 3)

6.3.1 Site investigation

The measurements used in this report are taken from boreholes on land on both sides of Göta River, assuming the same loading situation from the filling. The boreholes are within an area of 650 × 150 m and have a maximum height difference of one metre, see Appendix 3. The pore pressure is set to hydrostatic and starts one metre below the soil surface. Piston sampling was done down to 60 m depth and shows clay with small lenses of sand. There are organic materials in the top metres and the clay is occasionally shell-bearing, see Table 6.5.

Table 6.5 Summary of soil layering and soil properties for Marieholm Tunnel.

Level (m)	Material	ρ (t/m^3)	w_L (%)	Δu (kPa/m)
12 – 10	Dry crust / Filling with organic material	No data		
10 – 4	Sulphide-bearing clay	1.54-1.60	70	Medium
4 – (-2)	Clay	1.55	65	
(-2) – (-23)	Sulphide-bearing clay	1.55-1.60	70 - 80	
(-23) – (-48)	Silty clay with sulphide layer	1.60-1.67	70 - 75	

The preconsolidation pressure is estimated from CRS tests performed for seven boreholes, four taken on Hisingen and three on mainland. The measurements are taken on samples from level + 9 m down to -48 m. The soil is overconsolidated down to level +3 m, which is at 9 m depth. OCR is decreasing below that level, with a minimum value of 1.3. Our best estimated shear strength is based on the data from the field and laboratory investigations presented in Table 6.6.

Table 6.6 Number of tests used in evaluation of the best estimated shear strength.

Investigation method	Number of samples	Number of boreholes
CPT	-	6
Active, undrained triaxial test	6	1
Passive, undrained triaxial test	4	1
CRS test	6+7+9+5+7+16+6	7
DS test	5+5+5+5+5	5

Vane tests are taken in four boreholes on each side of the river, while fall cone tests are taken in two boreholes on Hisingen and four boreholes on mainland. There are data from four vane tests on each side of the river, two fall cone tests from Hisingen and four fall cone tests from mainland. The CPT-results are presented down to 60 m depth, even though two tests are performed 10 m deeper. The CPT recordings were evaluated with respect to shear strength.

6.3.2 Presentation of test data

The preconsolidation pressure does not show any distinct differences between measurements from Hisingen and mainland. In the figure for the preconsolidation pressure in Appendix 3 values from Hisingen are presented as “H” and values from mainland are presented as “G”. We have chosen to draw a vertical line for the top three metres and use linear regression for deeper measurements.

The shear strength calculated from CPTs show linear increase over depth, with some larger scatter of the results below 20 m depth, see Appendix 3. The average is calculated for each recorded level down to the level -50 m, since there are recordings of undisturbed samples to that level. This is presented in Figure 6.12 with the laboratory tests from Table 6.6. The trendline for our best estimated shear strength is based on a linear regression. The empirical f(triax A) and f(triax P) and all DS tests except for the two deepest measurements from borehole 21015 are taken into consideration. These two tests show a lower shear strength than the other DS test at that depth and have clearly lower values than the empirical f(triax A).

The average CPT shows similar tendency over depth as the DS tests, while the trendline for empirical f(σ_c') does not. The active triaxial tests, presented with open triangles, are below the empirical active f(σ_c')-line. The same tendency is for the DS tests compared to the empirical f(σ_c'). The empirical f(σ_c') is more than 20% higher than our best estimated trendline. It has several distinct shifts in inclination, which is due to a shift in liquid limit at these depths.

The vane tests and fall cone tests are presented in Figure 6.13 with our best estimated shear strength trendline. The values are within the ±10%-margin down to the level -2.5 m, which is approximately at 15 m depth. Both test methods drop in shear strength below that, except for the fall cone test from borehole 11002. A majority of the test results from these boreholes are within the $\pm$10%-margin of our best estimated trendline. A linear regression for that borehole only gives a line parallel to our chosen trendline and approximately 3 kPa below it. This is in line with how the correction factor should correspond with other methods, and is therefore still evaluated.

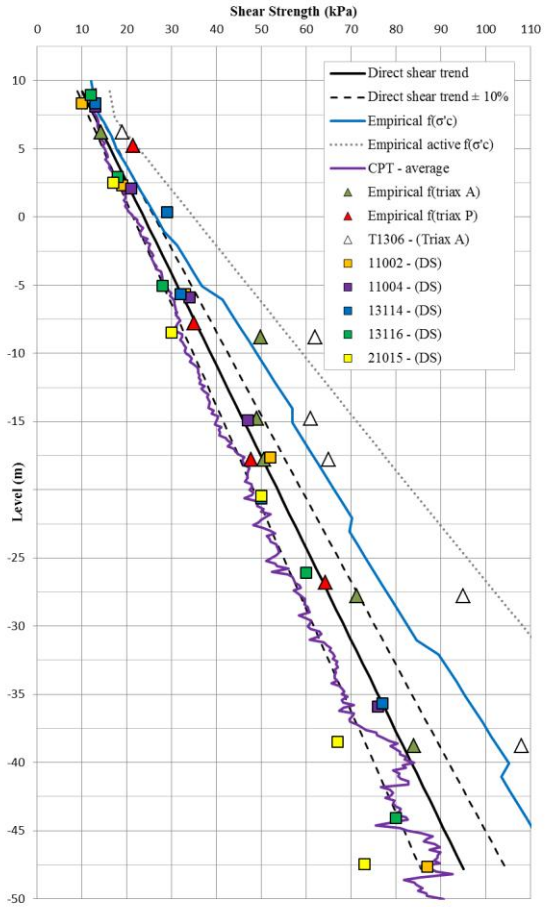


Figure 6.12 Best estimate shear strength from laboratory tests (for explanation of points and lines see figure 5.1).

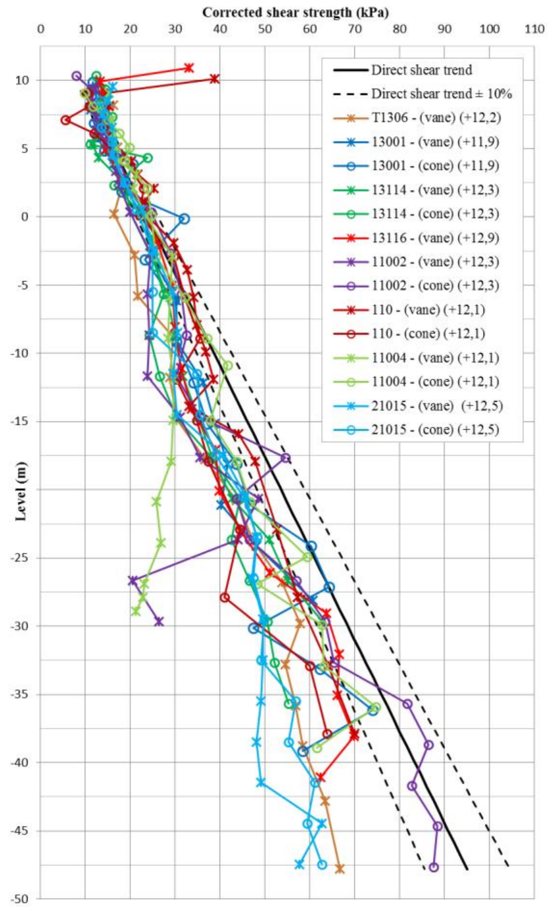


Figure 6.13 Corrected shear strength from vane tests and fall cone tests.

The test results from borehole 11004 shows higher shear strength from fall cone test than from vane test. The results from the vane test below level -17 m are unlikely for this borehole, as the shear strength decreases over depth. The same tendency is for the two deepest measurements for vane test from borehole 11002. We have therefore chosen to disregard these values in the further evaluation.

The chosen measurements below level -2.5 m are plotted in Figure 6.14 and Figure 6.15 with corrected and uncorrected shear strength. A trendline based on linear regression is presented for each test method. The linear regression was separated into four parts, as the tests did not have on linear trend over the entire depth.

The corrected values for fall cone tests are within the ±10%-margin down to level -2.5 m. The vane tests are within the margin down to -5 m, a depth of 17 m. The trend has very small shear strength increase below level -27 m, even though several tests are done.

The uncorrected values for both methods have an average larger than the best estimated trend +10% for shallow depths. A majority of the values are within the ±10%-margin below level -5 m. The fall cone tests show higher shear strength than the vane tests for these depths. This is due to the high test results from borehole 11002. If these were neglected, the linear trend below level -28 m would be equal to the trend for vane tests.

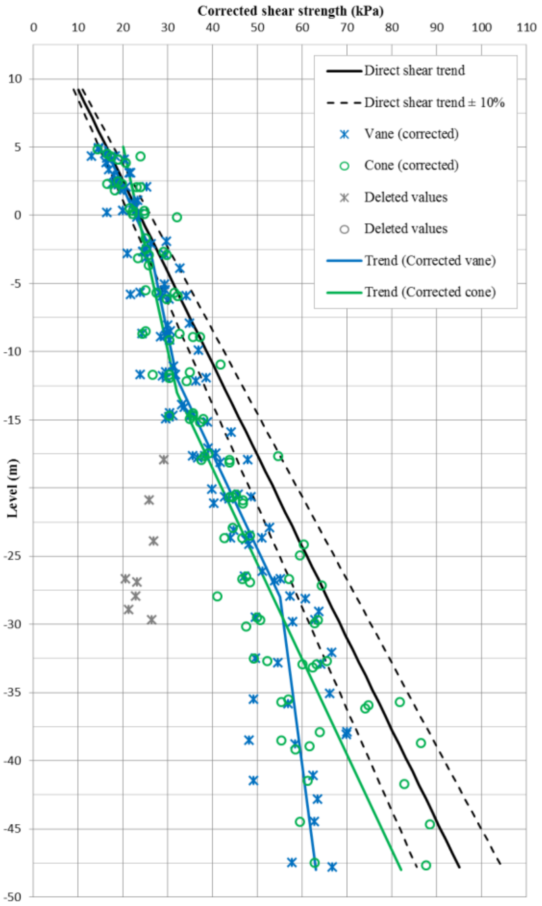


Figure 6.14 Corrected shear strength with trendlines.

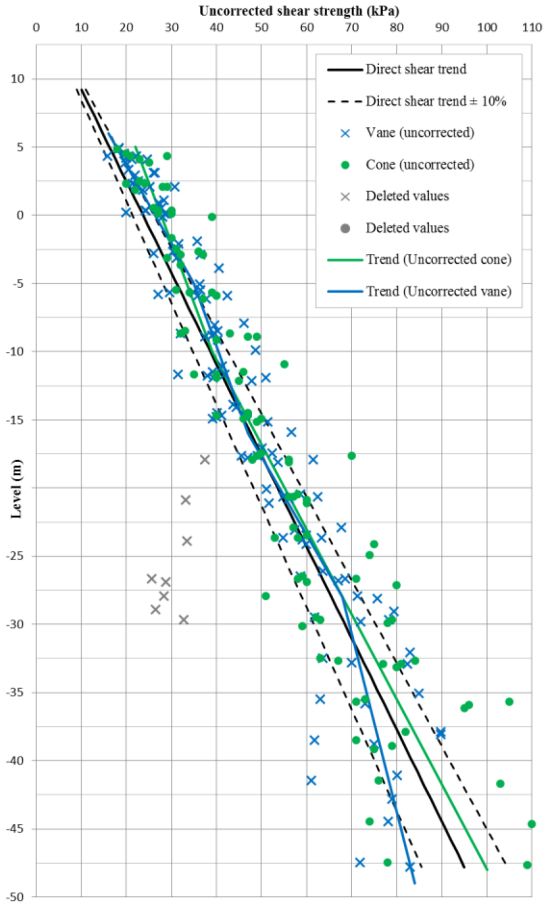


Figure 6.15 Uncorrected shear strength with trendlines.

6.4 Alelyckan

Alelyckan is located 3 km upstream from Göta Älv Bridge, just north of Gothenburg city. The tests from this location are all taken on the mainland, at the eastern side of Göta River. The investigations used in this chapter are taken from a master thesis written by Petersens in 2011.

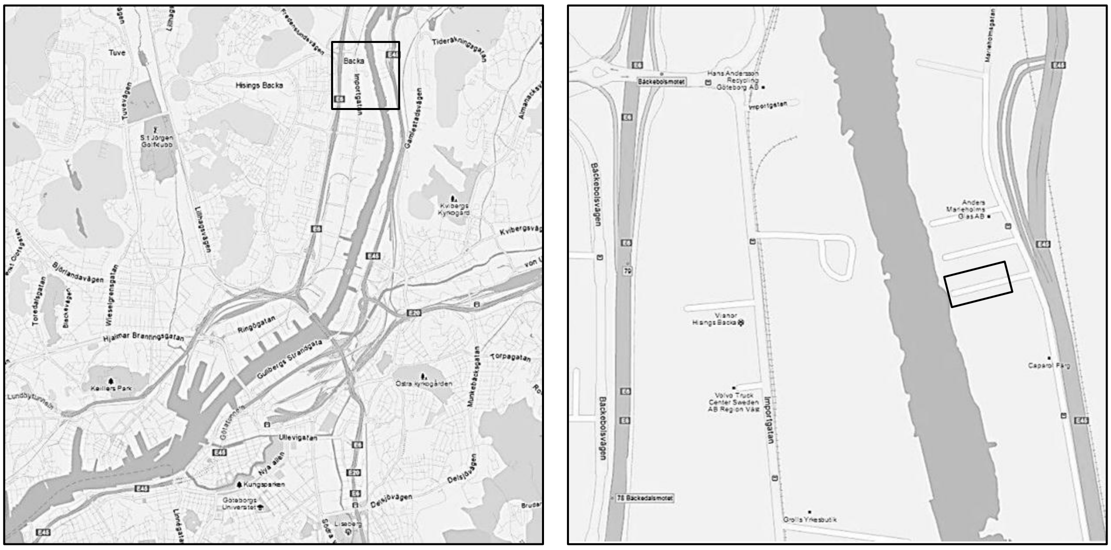


Figure 6.16 Map of Alelyckans position (borehole plan is presented in appendix 4)

6.4.1 Site investigation

The site is located in the harbour and is flat with a height difference of less than one metre. The boreholes taken into account for this report have a maximum difference in height of 0.3 m and are all on the mainland. Two boreholes are investigated with both vane tests and piston sampling, which have been used for fall cone tests in laboratory. These boreholes are located 70 m from each other and four CPTs are performed in a straight line between them, see Appendix 4. The measurements are done down to 30 m depth. The hydrostatic pore pressure is set to one metre below surface, at level +10.6 m.

Based on the soil samples, the layering consist of clay with parts of shells at all depths investigated. There are some layers of sulphide-bearings at several depths, which is more significant in the eastern borehole 100612. This borehole has the largest distance from Göta River. The CPT results show sand in the top 4-5 m of the soil. Below that is a layer of clay down to the level -10.4 m, where all four CPTs show silty clay. This is confirmed by the soil samples taken at level -18.4 m, see Table 6.7.

Laboratory investigations of the soil samples shows that the liquid limit does not vary much over depth. The clay is medium sensitive, with values varying between 15 and 24.

Table 6.7 Soil layering and summary of soil properties for Alelyckan.

Level (m)	Material	ρ (t/m^3)	w_L (%)
11.6 – 8.6	*No data*		
8.6 – 6.6	Silty sand containing plant remains	1.90	73
7.6 – 6.6	Somewhat shell-bearing clay	1.90	73
6.6 – (-10.4)	Somewhat shell-bearing clay	1.59 – 1.63	70 – 76
(-10.4) – (-18.4)	Sulphide-bearing clay	1.59 – 1.63	70 – 76

OCR is based on the five CRS tests done for the location. The OCR increases slightly with depth, from 1.3 to maximum 1.4. All the tests used in the evaluation of our best estimated trendline are presented in Table 6.8. The CPTs are evaluated based on a hydrostatic pore pressure starting one metre below surface.

Table 6.8 Number of tests used for evaluating best estimated shear strength.

Laboratory method	Number of tests	Number of boreholes
Active, undrained triaxial test	2	1
CRS test	5	1
DS test	4	1
CPT	-	5

6.4.2 Presentation of test data

Results from the evaluated CPTs are presented in Appendix 4. All five boreholes show the same tendency; a linear trend to level 0 m and a lower linear increase to level -11 m. Values below that have a larger scatter. We have chosen to take an average for each recorded level to use in the overall comparison of shear strength.

The preconsolidation pressure is determined from linear regression for the CRS tests, where all results follow a linear trend, see Appendix 4. These values are calculated with empirical relations to be comparable with direct shear strength, see empirical f(σ_c') in Figure 6.17.

Our best estimated trendline is a linear regression of the DS tests and drawn as a solid black line and is dashed where the CRS tests are extrapolated. The trendline is very similar to the empirical f(σ_c'). The two open triangles for active undrained triaxial test are somewhat in line with the empirical relation for active shear strength f(σ_c').

The average value of the shear strength from CPT is substantially lower than the other test methods. The given pore pressure is equal to the conditions at the Marieholm Tunnel, which is 2.5 km downstream.

Figure 6.18 shows our best estimated trendline with vane tests and fall cone tests from two boreholes. Only three measurements per vane test fall within the ±10%-margin, and this occurs at maximum 10 m depth.

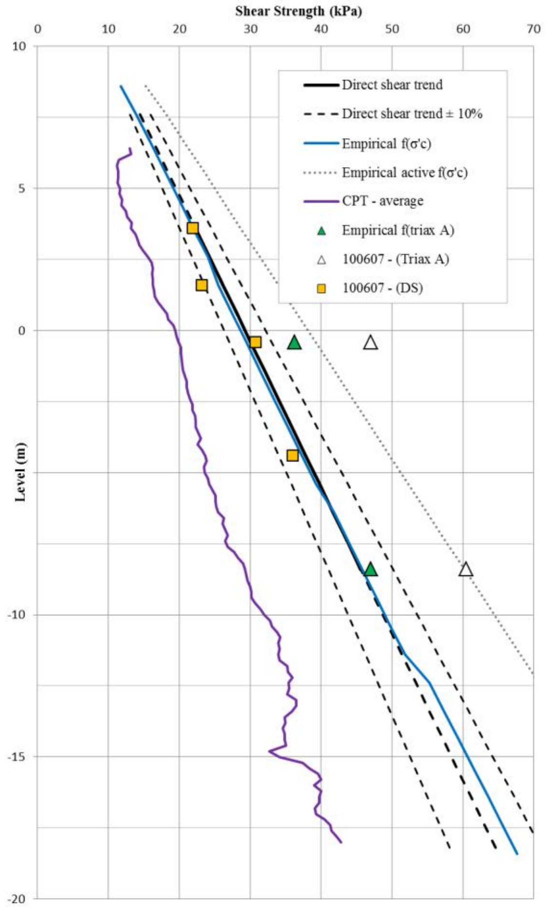


Figure 6.17 Best estimate shear strength from laboratory tests (for explanation of points and lines see figure 5.1).

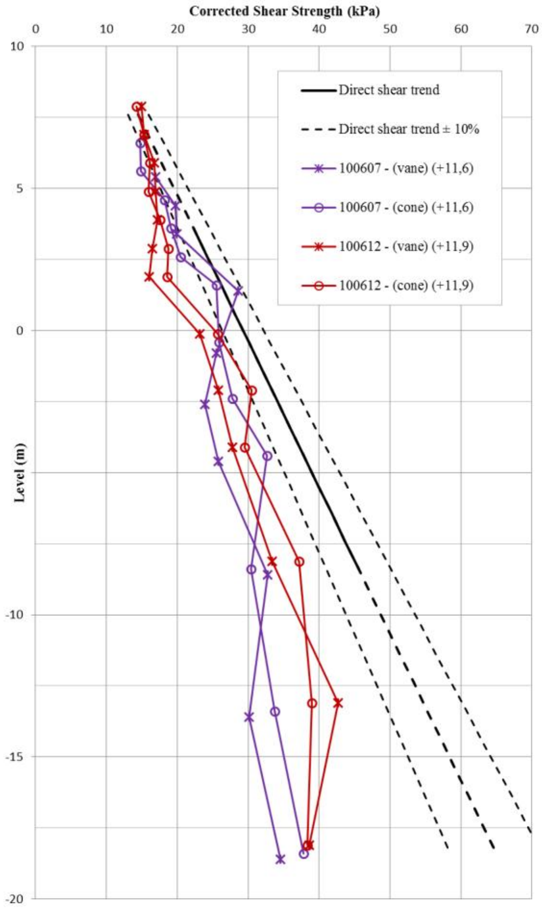


Figure 6.18 Corrected shear strength from vane tests and fall cone tests.

A trendline for corrected and uncorrected shear strength for each method is presented in Figure 6.19 and Figure 6.20, where each test result is weighted equally. We have chosen to include all test results, since no value stands out in either direction. The trendline for all vane tests is drawn from linear regression, while the fall cone tests are drawn with linear regression down to the level -5 m. The trendline below that is drawn from an estimated average.

The corrected vane tests are lower or equal to our best estimated shear strength -10%, with increasing difference with depth. The trendlines for both methods shows measurements of higher shear strength from fall cone tests than from vane tests.

The trendline for uncorrected shear strength from both tests aligns better with our best estimated shear strength for deeper levels. Here the trendline for vane tests is within the ±10%-margin down to the level -4 m, meaning a depth of 16 m. There are measurements larger than our best estimated trendline +10% for the vane tests down to 10 m depth. The deepest vane test measurement within the ±10%-margin is at level -13 m, a depth of 24 m.

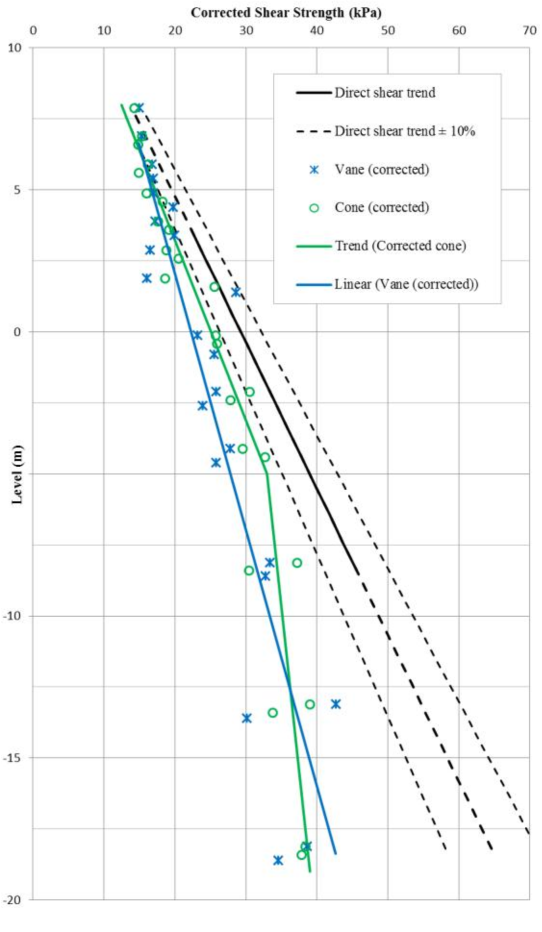


Figure 6.19 *Corrected shear strength with trendlines.*

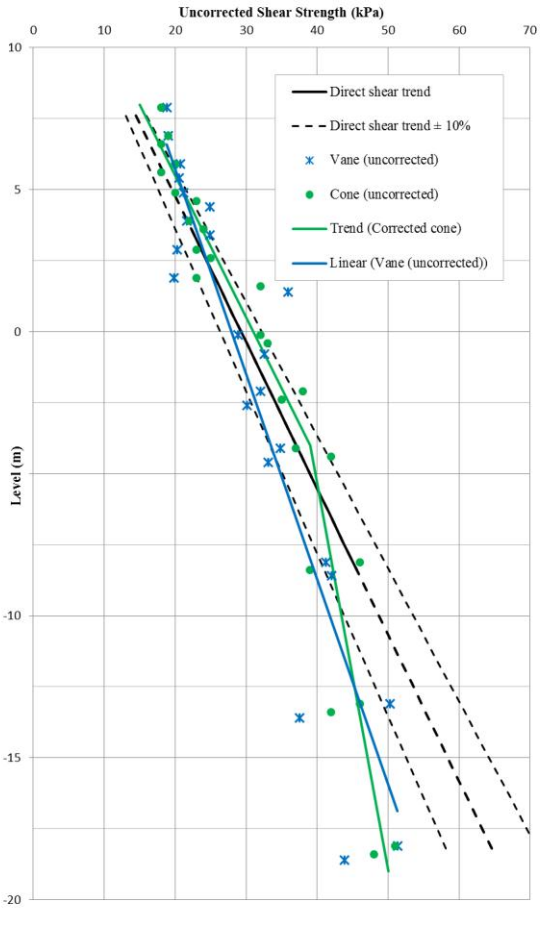


Figure 6.20 Uncorrected shear strength with trendlines.

6.5 E45 Agnesberg - Bohus

The location E45 Agnesberg - Bohusis a part of a stretch of the Highway E45 and is a part of the infrastructural project BanaVägiVäst, section Agnesberg – Bohus. The Göta River stretches in north-south direction, with road and railway on the eastern side. The road is parallel to the adjacent Göta River, with a railway track going in between. Several investigations were performed by three companies to investigate the total stability when both road and railway were expanded, see Appendix 5 for borehole plan.

Figure 6.21 Map of E45 Agnesberg - Bohus positon (borehole plan is presented in appendix 5)

6.5.1 Site investigation

The location is approximately 1.3 km north of Angered Bridge and the distance from the middle of the road to Göta River is approximately 80 m. It is a flat area located between the Göta River in the west and a mountainside on the east. The soil surface for the investigations has a maximum height difference of 0.6 m within an area of 650 × 100 m.

14 pore pressure measurements at three boreholes and at different depths generally show a small pore over pressure, with a water level starting just below ground surface, see Table 6.9. A majority of the boreholes show that the clay is more or less shell-bearing at all depths.

Table 6.9 Soil layering and summary of soil properties for the location.

Level (m)	Material	ρ (t/m^3)	w_L (%)	Δu (kPa/m)
0 – (-1)	Dy-bearing humus	No data		8.6
(-1) – (-6)	Gyttja-bearing clay	1.45 - 1.48	90	10.3
(-18) – (-6)	Sulphide-bearing clay (Sulphide clay)	1.50 - 1.55	85	
(-18) – (-25)		1.55 - 1.58	85 – 90	11.3
(-25) – (-30)		1.57 - 1.61	90	9.9
(-30) – (-40)		1.60 - 1.63	80 – 85	10.7

The depth of the gyttja-bearing clay varies ±5 m within the location, where boreholes close to the river shows a thicker layer. Two independent companies have defined the layering from the piston sampling. The soil below level -6 m was identified as sulphide-bearing clay by one company and sulphide clay by the other. It is less likely to be sulphide clay, as it has not been detected in other areas along the riverbed. The calculations are therefore performed based on sulphide-bearing clay.

The soil is overconsolidated down to level -8 m. Below that, the OCR varies between 1.4 and 1.3, with generally decreasing values over depth. The laboratory tests, presented in Table 6.10, were taken down to 40 m depth. There were vane tests performed in eight boreholes and fall cone tests in nine boreholes, which are made to the same depth as the laboratory tests.

Table 6.10 Number of tests used for evaluating best estimated shear strength.

Laboratory method	Total number of tests	Number of boreholes
Active, undrained triaxial test	2+2	2
Passive, undrained triaxial test	3	1
DS test	1+4+4+4	4
CRS test	4+7+7+7+3+3+10	7

6.5.2 Presentation of test data

The CRS tests are presented in Appendix 5, with a linear regression for values below level -5.5 m. This is used in calculations with empirical relations, both for direct and active shear strength, and presented in Figure 6.22. These lines are comparable with active triaxial tests (open triangles) and empirically calculated triaxial tests. The empirical method corresponds well with the triaxial tests below level -24 m.

All DS tests are lower than what the empirical relation indicate, but the empirical f(triax P) shows similar values to DS tests. We chose to place our best estimated shear strength in line with the DS tests, as there are 12 individual tests showing the same tendency.

An external committee has analysed the shear strength results as a part of the BanaVägiVäst-project. They present, based on empirical relations, that the normalised shear strength with respect to preconsolidation pressure should be

$$\frac{\tau_{fu}}{\sigma'_c} \approx 0.25 - 0.27 \text{ for } w_L = 70 - 80\%$$

where w_L is the measured liquid limit. If the ratio is calculated based on results from vane tests and fall cone tests for this location, the value is lower;

$$\frac{\tau_{fu}}{\sigma'_c} \approx 0.20 - 0.22$$

which corresponds to a liquid limit of 50%. Their analysis of the passive triaxial tests showed a K_{0NC}-value of 0.53. If this is calculated with empirical relations, it will correspond to a liquid limit of 50%. A calibration of the CPT-tests based on the DS test results show values that are equivalent to the same liquid limit. The external investigation concluded that the empirical relations are not applicable below 15-20 m depth for this location. They also saw a distinct difference in shear strength dependent on the distance from the river. The test results presented in this report are all taken from the location close to the riverbed.

The corrected vane tests and corrected fall cone tests are within our best estimate trendline or are maximum 4 kPa lower than the ±10%-margin, see Figure 6.23. The values here are corrected with the measured liquid limit presented in Table 6.9. Some values stands out as substantially lower or higher and are therefore deleted in the further evaluation.

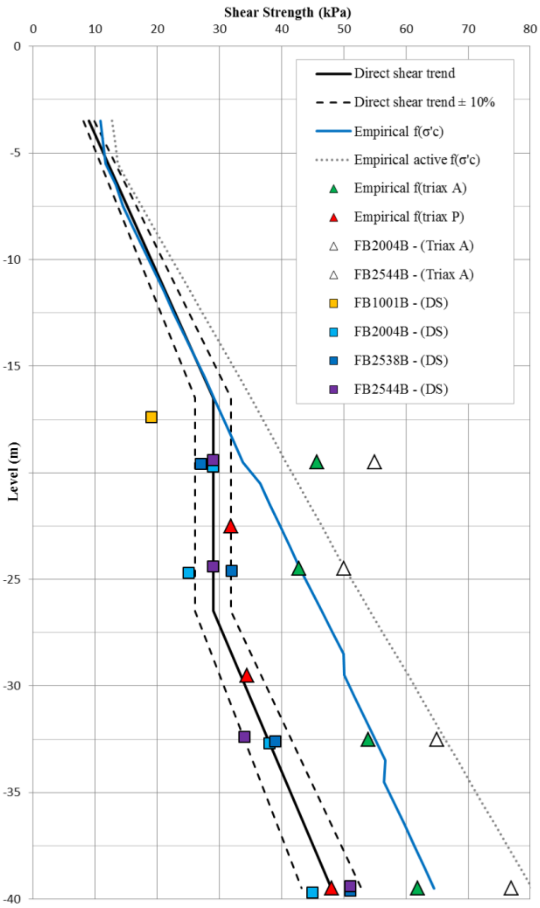


Figure 6.22 Best estimate shear strength from laboratory tests (for explanation of points and lines see figure 5.1).

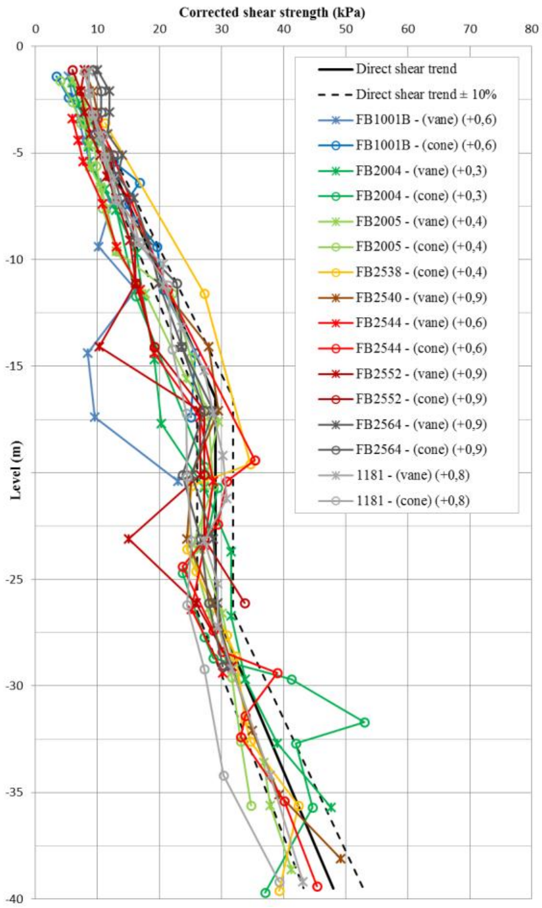


Figure 6.23 Corrected shear strength for vane tests and fall cone tests.

Figure 6.24 show the trendlines for corrected test results with our best estimated trendline. The trendlines are within the ±10%-margin for all depth, except for the vane test down to level -12 m. The fall cone tests have a larger scatter of results than the vane test, but show a very similar trend.

The uncorrected values are presented in Figure 6.25. A majority of the results from both test methods are larger than the best estimated shear strength +10%. The deepest specific value from vane test within the ±10%-margin is at level -20.5 m. The corresponding value for the fall cone tests is the deepest measurement done for the location.

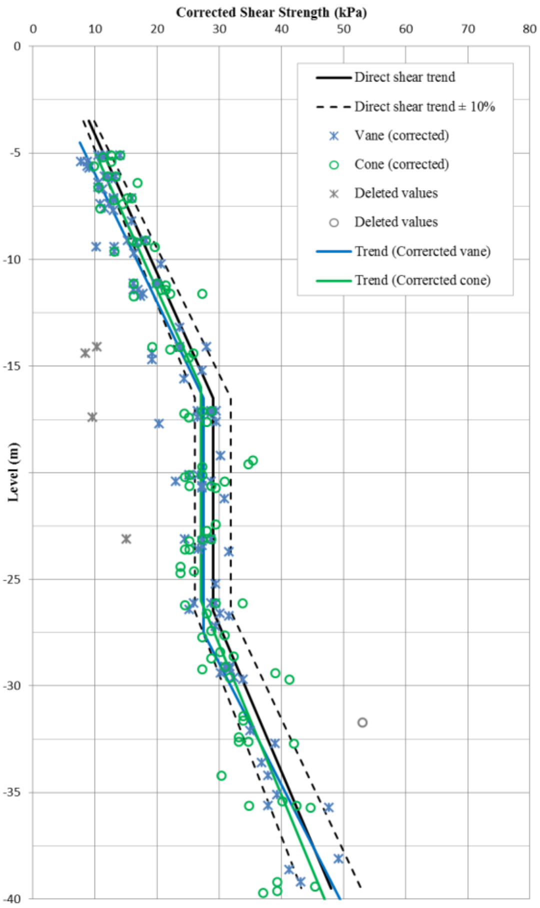


Figure 6.24 Corrected shear strength with trendlines.

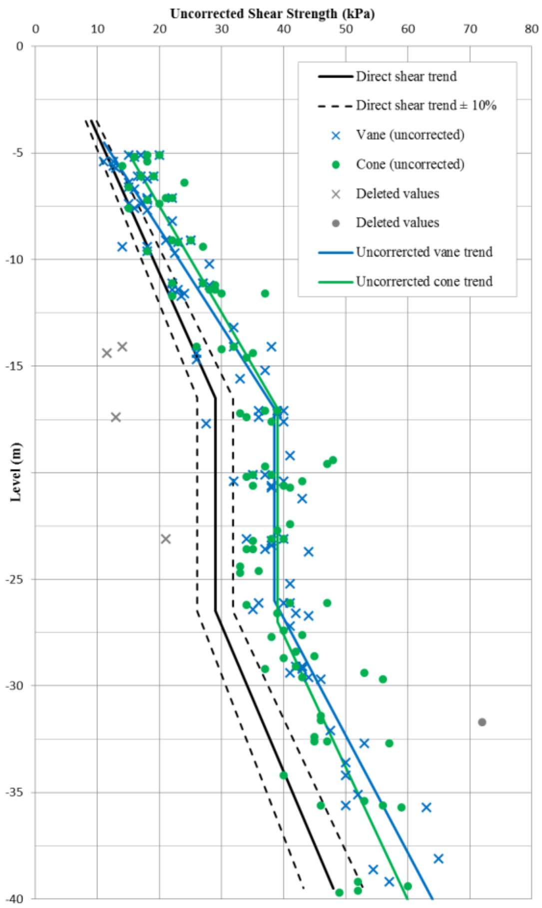


Figure 6.25 Uncorrected shear strength with trendlines.

6.6 Brodalsbäcken

Brodalsbäcken is another stretch of the Highway E45 which is also a part of the infrastructural project BanaVägiVäst. The road and railroad runs parallel to Göta River. There is a gorge with a creek running parallel to the road for 300 m, between the road and Göta River. An investigation was done to confirm the local stability when the road was to be expanded to the east.

Figure 6.26 Map of Brodalsbäckens position (borehole plan is presented in appendix 6)

The distance from the edge of the gorge to the road varies along this stretch, with the edge of the gorge close to the road. A typical cross-section with the gorge and road is presented in Figure 6.27, located at borehole 27102.

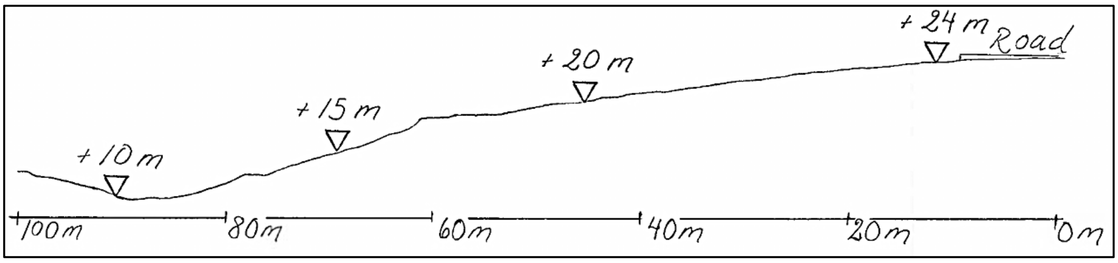


Figure 6.27 Cross-section of the area left of the road, at road section 27/160 m.

6.6.1 Site investigation

The data is taken from investigations performed on top of the gorge, with a maximum height difference of 3 m. The data is collected from boreholes within an area of 200 × 100 m. The top layer was investigated with a helical auger, which shows clay with plant remains. CPT was performed below that down to 41 m depth. Both pore pressure and cone tip resistance indicate clay with thin sand layers at the levels 7 m, 3.5 m and -12 m. The pore pressure is hydrostatic and is approximated to begin two metres below ground surface. The layering is presented in Table 6.11.

Table 6.11 Soil layering and summary of soil properties for Brodalsbäcken.

Level (m)	Material	ρ (t/m^3)	w_L (%)	Δu (kPa/m)
25 – 23	Dry crust with plant remains	*No data*		
23 – 19	Clay	1.55	70	Low/Medium
19 – 6	Clay	1.50 – 1.60	65	High
6 – 0	Sulphide-bearing clay	1.60	65	Medium
Below 0	Sulphide-bearing clay	1.55-1.70	75	

There is a layer of highly sensitive clay between level 19 m and 6 m. This was measured in borehole 27201, located on the present Highway E45. A piston sampling was also done 50 m west of this borehole, towards the gorge. The borehole, 27204, show high sensitive clay for a less thick layer; level 18 m down to 8 m. Also the deepest measurement in this borehole, at level -13 m, shows a high sensitivity. Measurements at higher levels and for corresponding levels in borehole 27201, show a medium sensitivity to low sensitivity.

The soil is overconsolidated down to five metres depth and below that OCR varies between 1.45 and 1.55. Piston sampling was done and the laboratory tests performed on these are presented in Table 6.12. These are compared with data from vane test from six boreholes and fall cone tests in three boreholes, at a maximum depth of 40 m. Two of the vane tests, borehole 27107 and 27108 are performed at a later occasion by a different company.

Table 6.12 Number of tests used for evaluation of best estimated shear strength, all for borehole 27108.

Laboratory method	Number of samples
Active, undrained triaxial test	4
Passive, undrained triaxial test	1
CRS test	4
DS test	3

There are pore pressure measurements from one borehole, which are installed at approximately 40 m depth. They show a pore pressure equal to a hydrostatic pressure starting 1-2 metres below surface. This is simplified to a ground water level starting two metres below soil surface.

6.6.2 Presentation of test data

The preconsolidation pressure is based on a linear regression for four CRS tests, see Appendix 6. The top metres are assumed to have a constant value, while the line below level 1 m is extrapolated. These values are used in empirical relation for both active and direct shear strength, see Figure 6.28. The empirical f(σ_c') line is steep at shallow depths due to the high OCR, while the change in inclination at level 0 m is due to a shift in liquid limit.

Our best estimated direct shear strength is presented with a solid black line within the interval where there are measured values from CRS tests and is dashed when extrapolated. The trendline is a linear regression of all DS tests and empirically calculated active and passive undrained triaxial tests.

The corrected shear strength from vane tests and fall cone tests are presented in Figure 6.29. The older tests follow a trend down to the level 7 m, while two boreholes with newly performed vane tests measures higher shear strength. Worth mentioning is that these new vane tests are performed in research purpose. The results for borehole 27107 below level +6.6 m show a decrease in shear strength, which is not reasonable and these values are therefore deleted in the further evaluation. This is even though the protocol for each recorded level show a distinct failure. The two closest boreholes, 27102 with vane test and 27204 with fall cone test, have increasing shear strength over depth.

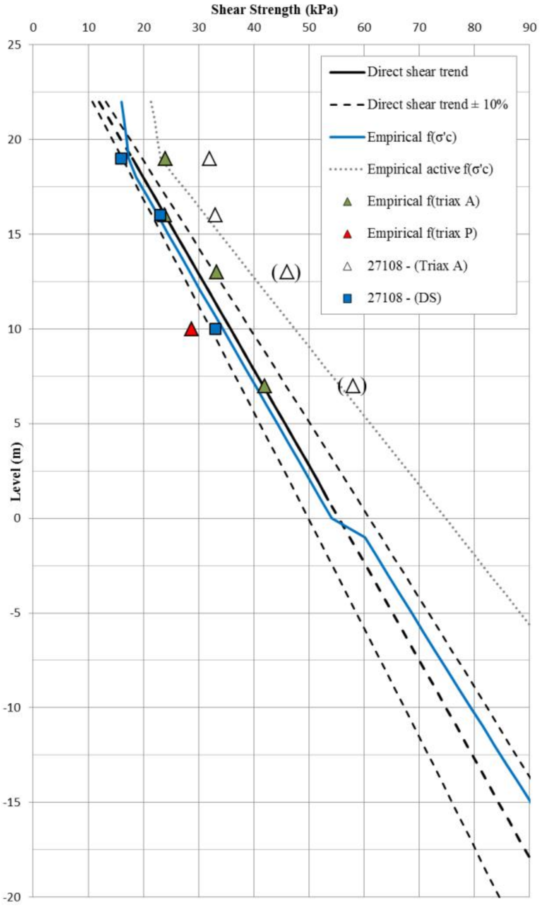


Figure 6.28 Best estimate shear strength from laboratory tests (for explanation of points and lines see figure 5.1).

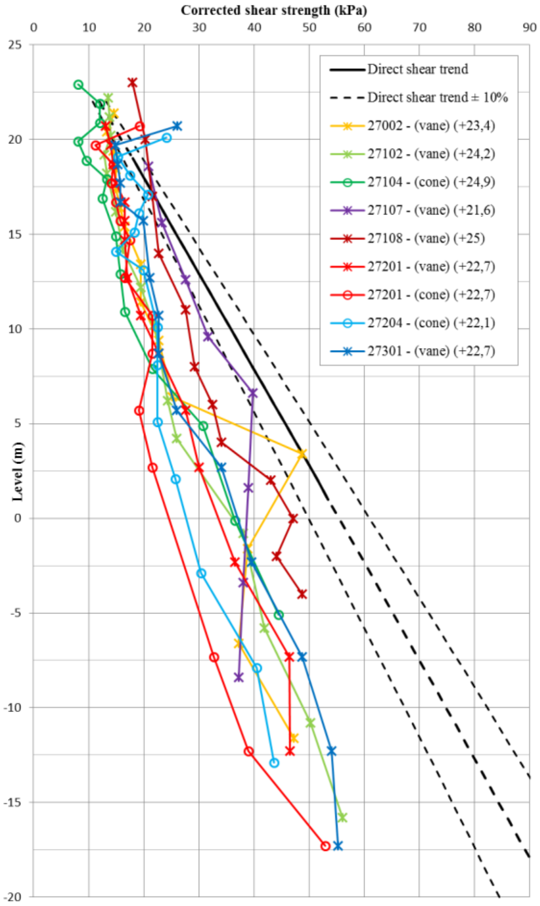


Figure 6.29 Corrected shear strength from vane tests and fall cone tests.

Figure 6.30 shows two linear regressions for the corrected shear strength; one for fall cone tests and one for vane tests. The trendline for both corrected vane and fall cone tests are substantially lower than our best estimate trendline. Only a few vane tests and two fall cone tests are within the margin of the trendline ±10%.

The same thing is presented in Figure 6.31, but with uncorrected values. The trend for uncorrected vane test is almost the same as our best estimated shear strength -10%. The uncorrected values from borehole 27107 show shear strength values higher than our best estimated shear strength +10%. The fall cone tests show lower values, with the deepest measured value within the ±10%-margin at level 16 m, a depth of 9 m.

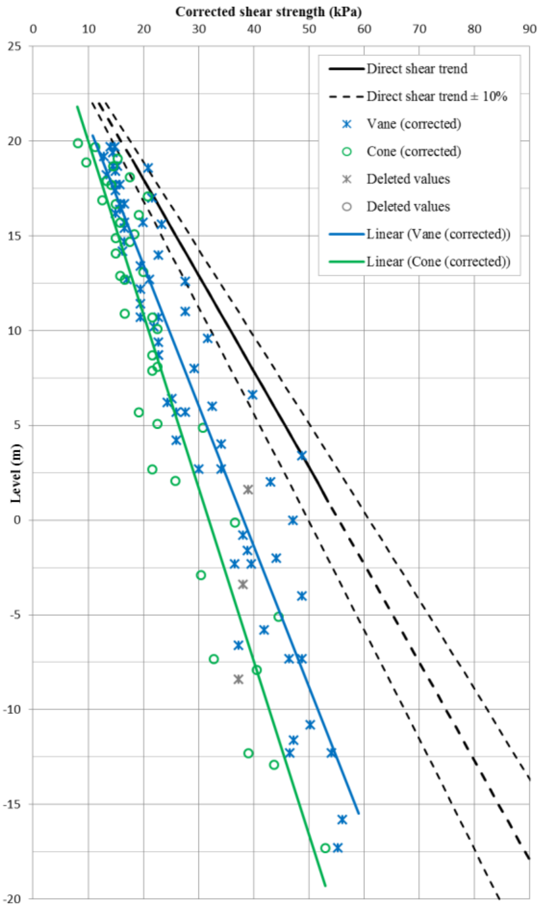


Figure 6.30 Corrected shear strength with linear regressions.

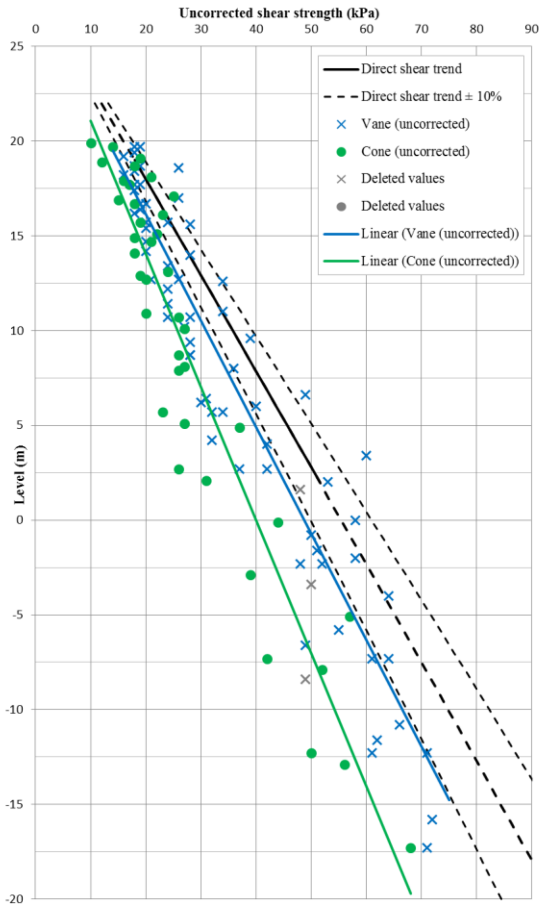


Figure 6.31 Uncorrected shear strength with linear regressions.

6.7 Kvibergsbäcken

The creek Kvibergsbäcken is 2.5 km east of Göta River in the suburban area Utby. The creek runs in a riverbed from north to south, ending up in the river Säveån. There are houses on top of the riverbed on both sides of the creek, with a road closest to the creek on the western side. The stability needed to be determined, where the northern part of Kvibergsbäcken is presented in this report.

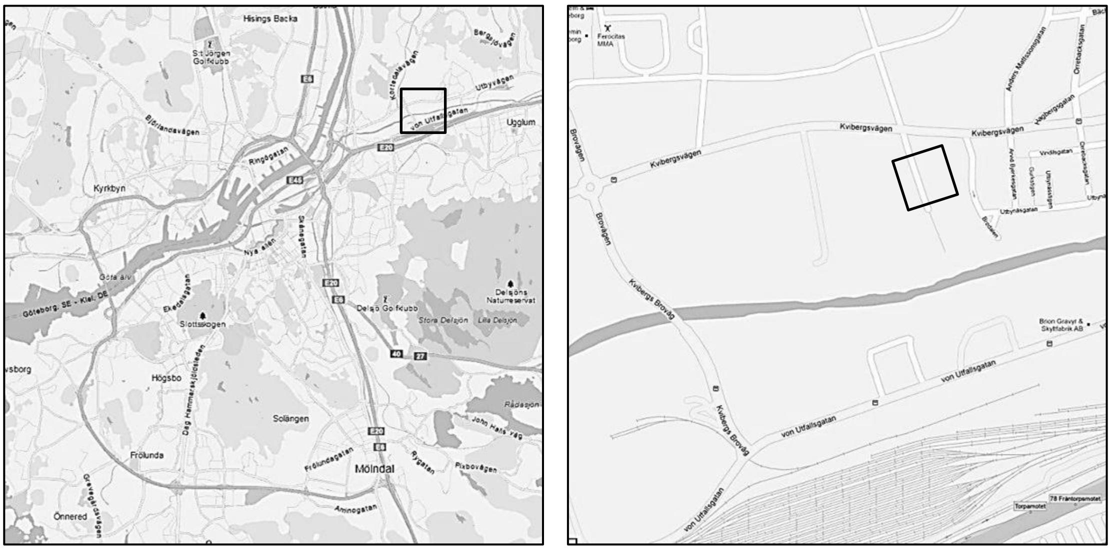


Figure 6.32 Map of Kvibergsbäckens position (borehole plan is presented in appendix 7)

6.7.1 Site investigation

The measurements are performed on land on the western side of the creek, see Appendix 7, with a maximum height difference of 6.5 m. A cross-section from the middle of the location is presented in Figure 6.33. The area inclines from level +20.8 m in the north to level +18.3 m in the south, while the bottom of the riverbed inclines from level +12.1 m down to level +11.4 m. The considered area is 120 x 120 m.

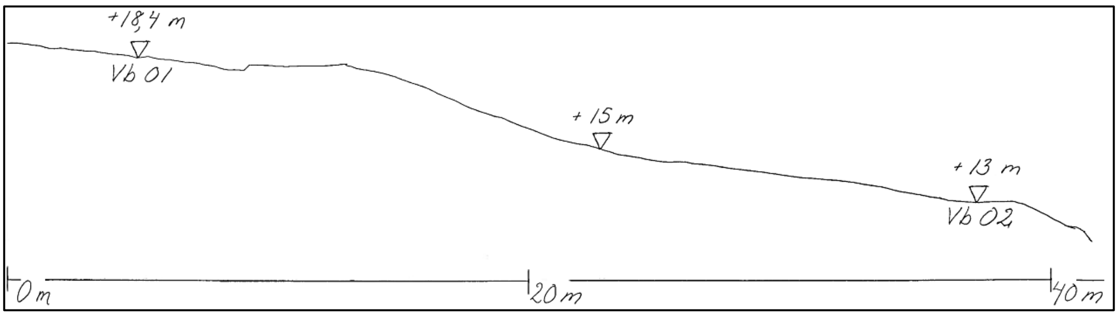


Figure 6.33 Cross-section 185 from Kvibergsbäcken location.

The pore pressure is hydrostatic from two metres below ground, level +17.5 m. The soil samples are taken down to level -7.5 m, which corresponds to a depth of 20 m. The top layer is three metres thick with a dry crust, followed by clay. Measurements show that the dry crust occasionally contains fill with bricks, silty sand and humus. The clay is somewhat shell-bearing down to the level 10 m, Table 6.13.

Table 6.13 Summary of soil layering and soil properties from borehole at the top of the riverbed.

Level (m)	Material	ρ (t/m^3)	w_L (%)
20 – 17	Mixed fill or dry crust	*No data*	
17 – 15	Clay containing plant remains	1.55 – 1.60	65 – 72
15 – 9	Somewhat shell-bearing, somewhat sulphide-bearing clay	1.55 – 1.60	65 – 72
9 – 0	Somewhat sulphide-bearing clay	1.60	75
0 – (-5)	Somewhat sulphide-bearing clay	1.62 – 1.65	70 – 72

The sensitivity is measured in borehole 12 and 13, with low sensitivity at three depths in borehole 13. All other results show a medium sensitivity. The clay is slightly overconsolidated with OCR of 1.3 at level +17.5 m and decreasing to 1.17 at level -5.5 m.

There are laboratory investigations performed for borehole 12, see Table 6.14. Vane tests are performed in six boreholes and fall cone tests in two boreholes. One borehole for fall cone tests is at the top of the riverbed, at level +19.5 m. The other borehole for fall cone tests is further down in the gorge, at level +13.9 m.

Table 6.14 Number of tests used for evaluating best estimated shear strength for borehole 12.

Investigation method	Number of samples
CRS test	2
DS test	5

6.7.2 Presentation of test data

The CRS tests are evaluated with linear regression to estimate a preconsolidation pressure over the entire depth, see Appendix 7. These values are used with empirical relations to be comparable with direct shear strength. The empirical line is presented in Figure 6.34 with the DS results. It is not perfectly linear, as it varies with OCR and liquid limit. Our best estimated trendline for the shear strength is a linear regression of the DS tests. The trendline is presented as a black solid line where CRS tests are interpolated, and is dashed when the CRS tests are extrapolated.

Our best estimated trendline is presented with fall cone tests and vane tests in Figure 6.35. The location has a big variation in elevation, but shows a linear trend. This indicates that this part of the creek is eroded from a flat landscape.

The valley has steep slopes, with dry crust on the first 3 metres of depth. As the elevation varies for the boreholes, the level of which the dry crust is penetrated will also vary. Borehole 01, 05 and 12 show constant shear strength at shallow depths, which could be a result of the dry crust not being completely penetrated. The values at larger depths are within the ±10%-margin. Borehole 13 shows an increasing diversity between fall cone tests and vane tests from level 8 m and down. Borehole 12 shows the same tendency from level 10 m. The vane test from that borehole has a distinct linear trend, while the fall cone tests increase 6 kPa over 10 m depth. Such a low increase is unrealistic in low or medium sensitive clay. The same results are for borehole 01.

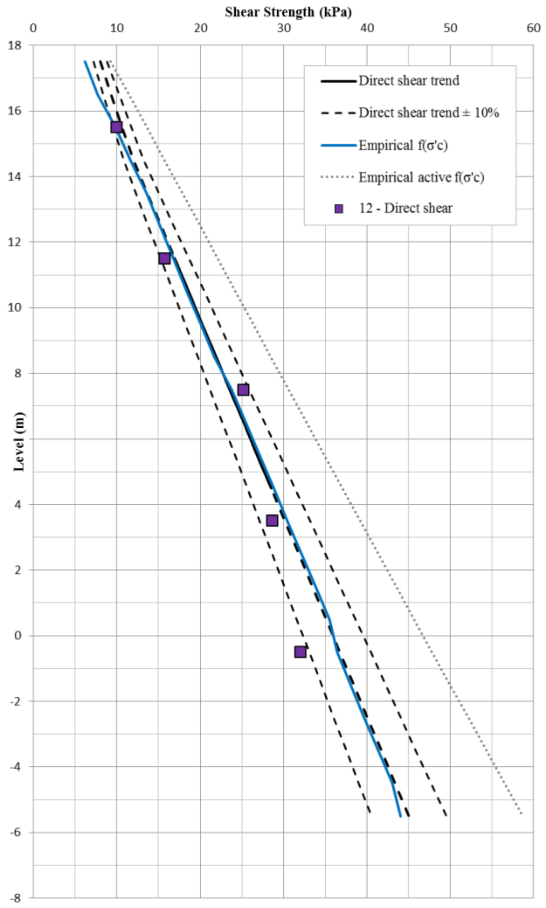


Figure 6.34 Best estimate shear strength from laboratory tests (for explanation of points and lines see figure 5.1).

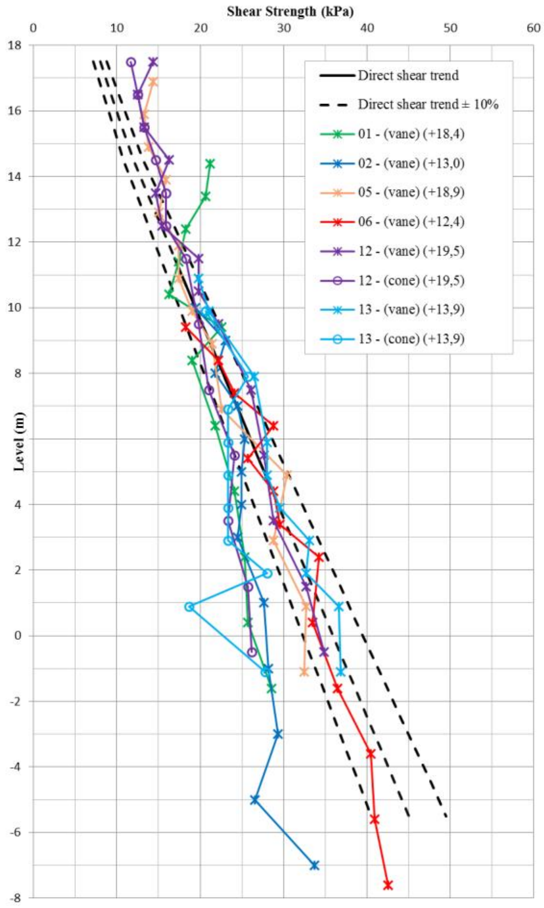


Figure 6.35 Corrected shear strength from vane tests and fall cone tests.

The top five metres are often exposed to disturbance due to a dry crust or filling and are therefore not representative for the clay below. We have therefore chosen to weight all measurements below level 15 m equally for each test method in a linear regression. The obtained trendline for the vane tests is within the ±10%-margin, see Figure 6.36. The corresponding trendline for fall cone tests has a much lower inclination, since all values between level 5 m and level 13 m are outside the ±10%-margin.

The same procedure is done for uncorrected values, see Figure 6.37. A majority of the measurements from both methods have shear strength values larger than our best estimated shear strength +10%. Both methods have values within the ±10%-margin from level 4 m, a depth of 15.5 m, and deeper down.

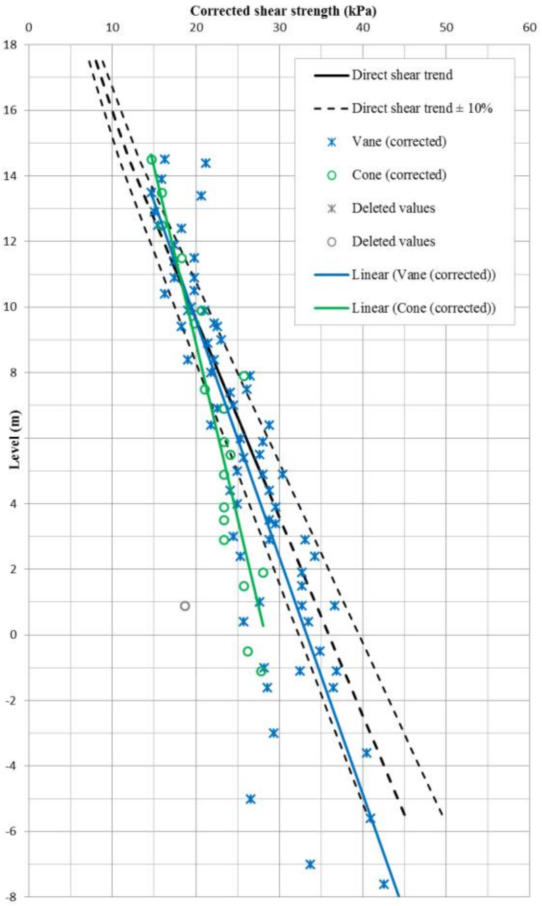


Figure 6.36 Corrected shear strength with trendlines.

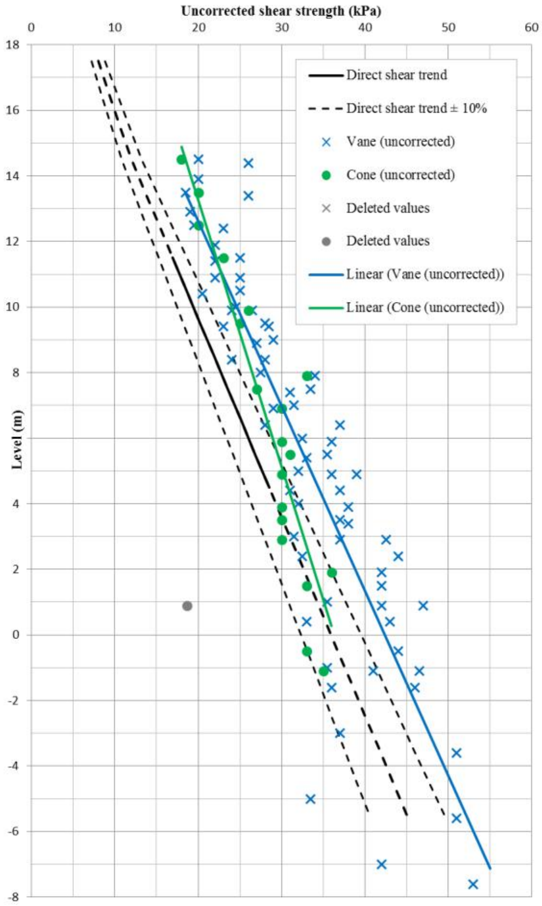


Figure 6.37 Uncorrected shear strength with trendlines.

7 Analysis

When evaluating and analysing data from the locations studied, it is clear that the corrected undrained shear strength from the vane and the fall cone tests do not always follow the best estimated shear strength through the whole profile. The best estimated shear strength is mainly based on DS tests and empirical calculated triaxial tests. The empirical relation from preconsolidation pressure is used as guidance. For the locations in general, the corrected values are correlating with, or are even higher than the best estimated shear trend in the top of the soil profile. For greater depths in the soil, the corrected vane and fall cone test results have a tendency to be too low. The deviation is therefore more distinct at locations where testing has been done to great depths. Figure 7.1 show how the trends for vane and fall cone tests vary with depth. It is an overview that shows whether the vane and fall cone test trend is lower, within or larger than the best estimated trendline ±10%.

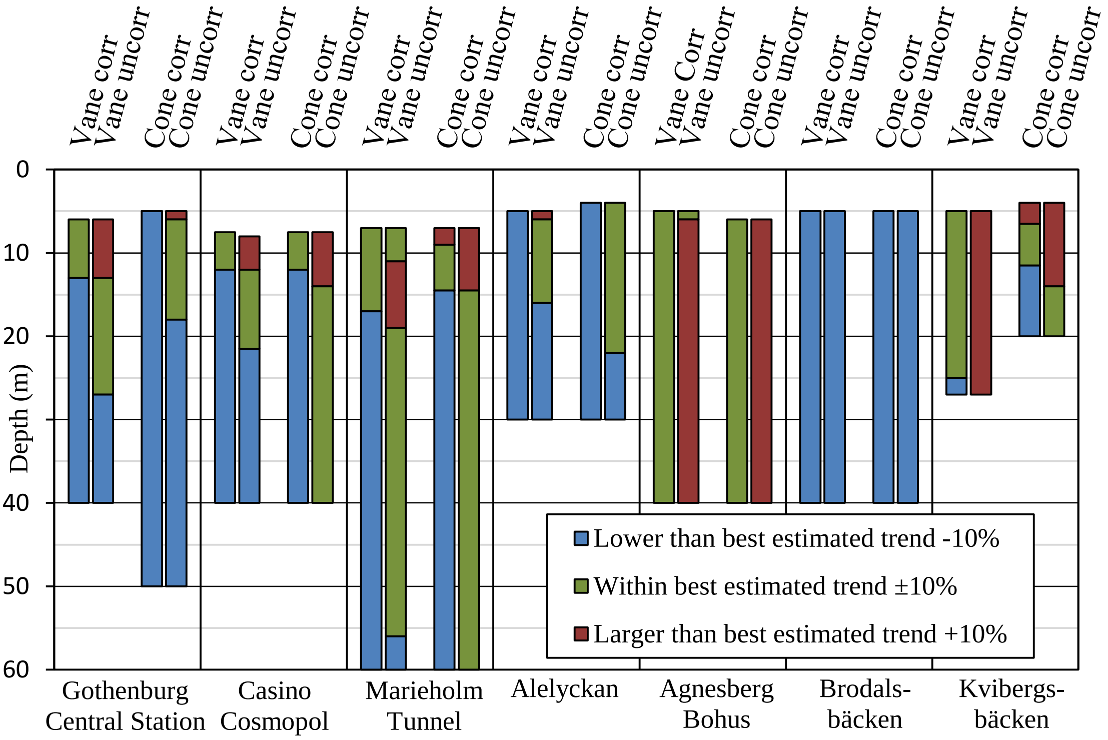


Figure 7.1 Comparison of vane and fall cone test results vs. best estimated trend

The figure shows that “E45 Agnesberg - Bohus” and “Brodalsbäcken” are significantly different from the other locations. As described in chapter 6, the location at “E45 Agnesberg – Bohus” is an unusual case for the soil behaviour by Göta River. Many experts have been involved before us and discussed what shear strength to determine for the clay. Our chosen best estimated shear strength is based on these statements and on discussions with our supervisors. Our chosen trend does look strange, but the direct shear tests are too many to reject as faulty. We think that this location is an exception and not representative for the western part of Sweden and should have less weight in the comparisons between locations.

The other location that stands out is “Brodalsbäcken”, which has low values through the whole profile. The reasons for the low values for could be the 10-13 m thick layer of high sensitive clay in the soil profile. The clay is then easily disturbed and can result in lower shear strength measurements with the vane test. The low measured shear strength from fall cone tests could also suffer from disturbance. The laboratory tests of which the best estimated trend is evaluated from, have a consolidation phase, which simulates the in situ conditions better than the fall cone test.

In the upper part of the soil, the trends for corrected vane and fall cone are close to or within the 10% deviation of the best estimated shear strength. This is valid for approximately the top 15 m from ground surface. When comparing this to the uncorrected trends for vane and fall cone tests it is clear that that the correction is required for the top metres of the soil, since trends for the uncorrected values are repeatedly over the best estimated shear trend.

In the lower part of the soil profile, depths larger than approximately 15 m, the trend for vane and fall cone tests have a tendency to deviate increasingly over depth. It is more or less in proportion to the best estimated shear trend for deep levels. For the location Kvibergsbäcken it is difficult to see this trend, because the boreholes are not deep enough. For the rest of the five locations, the deviance can be compared. At for example 30 m depth from ground surface, the trends are 20-40% lower than the best estimated shear strength, most of them over 30%. When comparing this to the correction factor at the same levels it is visible that the correction factor can be rejected at deep levels. Usually the liquid limit at deep levels is 70-80% which results in μ= 0.80-0.76, this is a reduction of 20-24%.

When the old method of correction for vane and fall cone tests was used, the reduction of shear strength was applied when the liquid limit exceeded 80%. This makes a significant difference in the western part of Sweden since the liquid limit seldom exceeds this level, especially at large depths. The only location studied in this report that has a liquid limit higher than 80% is “E45 Agnesberg – Bohus”. When changing the method of correction it was said that the old method with carefully chosen values would give approximately the same result as the new method with statistical mean value. The difference between no correction and 24% does make a difference and it could be that clays in this range of liquid limit are more exposed to this change.

The empirical relation for direct shear strength shows good correlation with the direct shear tests and our best estimated trendline for a majority of the locations. Generally, the test results have slightly lower values than the empirically calculated values. The locations that had more than 10% deviation from our best estimated trendline were “Casino Cosmopol”, “Marieholm Tunnel” and “E45 Agnesberg – Bohus”. The locations presented in this report show no clear relation between empirical relations and the results from vane tests and fall cone tests.

When viewing all the data from vane and fall cone tests for each location, it can be concluded that the test methods have good precision, especially in the top of the profile. There are some diverging values but most of them are in connection with the trend for the test. This indicates that the test procedure itself is good and the problem is how the data is handled.

Another reason for the low shear strength values is that the vane tests might not be performed according to the Swedish regulations. The protocols from vane tests are often not presented with the results, so the performance of the test procedure is hard to evaluate afterwards. An example of this is “Brodalsbäcken” location, where vane tests were performed by CTH in two of the boreholes. These test results performed for research purpose are 25-50% higher than the other test results, even though the test procedure should be identical.

8 Discussion

As mentioned in Chapter 2, the initial correction factor was based on a number of landslides, a couple of fullsize load tests and direct shear tests. In addition, the fall cone test was also calibrated with respect to loading tests on piles. When the correction factor was revised in 1984, these experiences were still considered together with new experiences. The tests were performed at relatively shallow depths, compared to the depths that are presented in this report. Back-calculated landslides and failures, as well as the full-scale loading tests have seldom slip surfaces deeper than 15 m. The boreholes considered in this report are down to 60 m of depth, and the correction method has thus been extrapolated for 45 m.

We have chosen to include the fall cone tests, as the cone shear strength is corrected in the same way as for vane tests. The clay is often stiffer at greater depths, which requires a greater weight to perform the fall cone test as recommended. The change of weight can make a distinct shift in the linear trend for shear strength for the borehole.

The empirical relations based on CRS tests have been presented down to 60 m depth. The friction between the ring and the soil sample is noticeable in comparison with triaxial tests below 30 m depth. This gives an overestimation of the shear strength based CRS tests for deep depths. Also, the values from CRS tests are extrapolated for three of the locations; “Alelyckan”, “Brodalsbäcken” and “Kvibergsbäcken”. This increases the uncertainty of our best estimated shear strength at these depths.

What we have seen from the four locations with CPT data is that the shear strength evaluated from CPT test results are less than or equal to our best estimated shear strength -10%. It does not seem as the absolute values are comparable with other methods, but the shear strength increase with depth could be usable for identifying a trend.

Our locations vary in size and the number of tests performed within it. Three of the locations have a great number of vane and fall cone tests performed; “Gothenburg Central Station”, “Marieholm Tunnel” and “E45 Agnesberg – Bohus”. These show a good precision between vane tests and fall cone tests. The other four locations have a larger scatter of the measurements. The trendlines for them are evaluated from linear regressions, since it was hard to detect a trend just from the measurements. This can give a misleading trendline, as more measurements could help identifying specific changes in shear strength increase.

We chose to add two empirical relations together, to be able to compare active triaxial tests with direct simple shear tests. The relation between these points and the empirical relation lines remains the same, as they are founded on the same parameters. It might be discussed if this is a correct way to use active triaxial tests.

Some values for vane tests and fall cone tests have been removed from the evaluations with all tests. We have mostly chosen to remove values that have extremely low values, and very few values that are extremely high. This was our way to avoid the conflict between adjusting data and improving data in favour of the thesis.

The test procedure itself for the vane test is an aspect of the result. There are Swedish recommendations of how to perform the vane test, but it is not always possible to validate that these are followed. The dry crust, fillings and sand layers need to be pre-drilled with helical auger or likewise before the vane is mounted. In that way, it is less risk for stones, gravel or shells are pushed in front of the vane and remould the clay before the test is done. There is also a risk that firm clay gets stuck on the vane, resulting in the same thing. The remoulded shear strength is lower than undisturbed shear strength, meaning that value will be underestimated.

Another aspect of the test procedure is the use of casing for the rod. The common practice in Sweden is to not use the casing. This means that the soil gives a horizontal pressure on the rod. In theory, the torque for the rod and the torque for the vane are separated with a clutch. We inspected a typical clutch for the vane test, and noted that it does have a friction, which will affect the result.

A measurement at 35 m depth for Casino Cosmopol had a torque just for the rod of 48% of the maximum torque, when the borehole was pre-drilled 2 m. This is a representative value for the method, and it is not so strange if this would affect the final calculated shear strength.

At greater depths or in firm clays, the steel itself can be twisted during the rotation. When the clay is approaching failure, this “built in” torque can release and make the soil go to failure. The shear strength could then be underestimated, as this twist from the rods is not taken in to account in the measuring.

9 Conclusion

The present correction factor gives a good estimation of shear strength from vane tests and fall cone tests for clays at shallow depths for a majority of the locations. The vane tests align well with direct shear tests and empirically calculated triaxial tests down to 12 – 25 m depth. The corresponding depth for fall cone tests is 12 – 14 m. The corrected shear strength from both test methods gives an underestimation of the shear strength for larger depths.

The major uncertainties for this conclusion are how the soil properties vary within each locations and how the tests have been performed on site. The test performance includes both test procedure and test equipment.

These seven locations might not be representative for the entire western part of Sweden, but should be seen as guidance for the soil behaviour in the region. The empirical relations are a good complement to performed tests, but they cannot be used alone due to local variations.

10 Recommendations for further studies

As mentioned in the report, the corrected shear strength from vane and fall cone tests gives lower values than other methods for larger depths. Further investigations should be performed in order to revise or remove the correction factor below 15 m.

When the depth increase, the length of the rod does too, and there could be a risk of affecting the test result. If the rod is twisted too much it might cause a dynamic force when it releases. This is one source of error which could be interesting to investigate. Another source connected to the torque is how correct it is to assume that the sleeve friction is constant since this is a big part of the total developed torque.

We know for a fact that there is a deviation of the test result depending on different causes. To see how much data varies when the same test is performed several times, a small location could be selected for testing. Several tests should be performed, transported and analysed in the same way. This could be done for both vane and fall cone tests to ensure that the tests have a good precision. If this is the case then it might be a good idea to review the regulations for the testing procedure to make sure that all data is of good quality.

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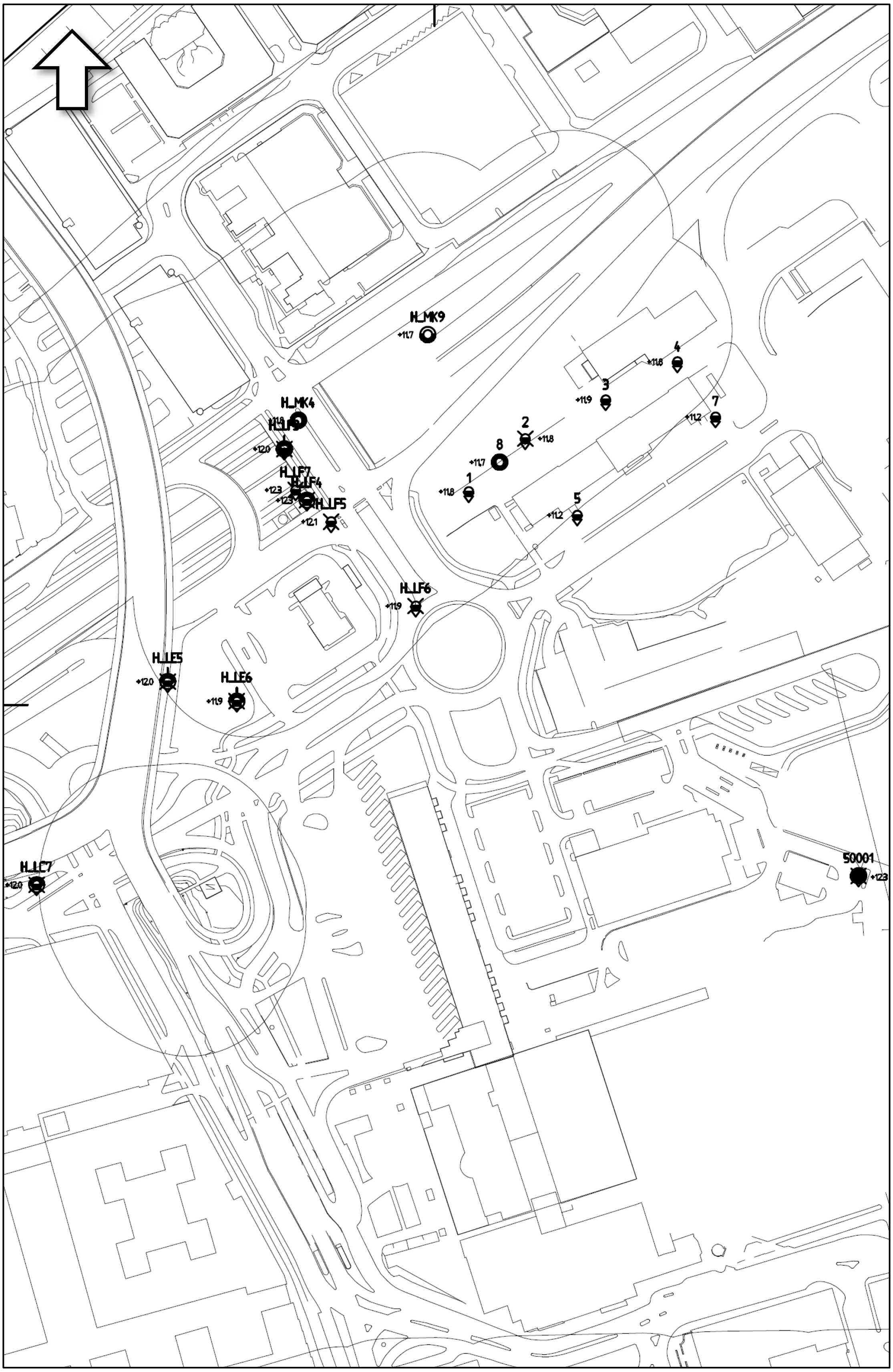
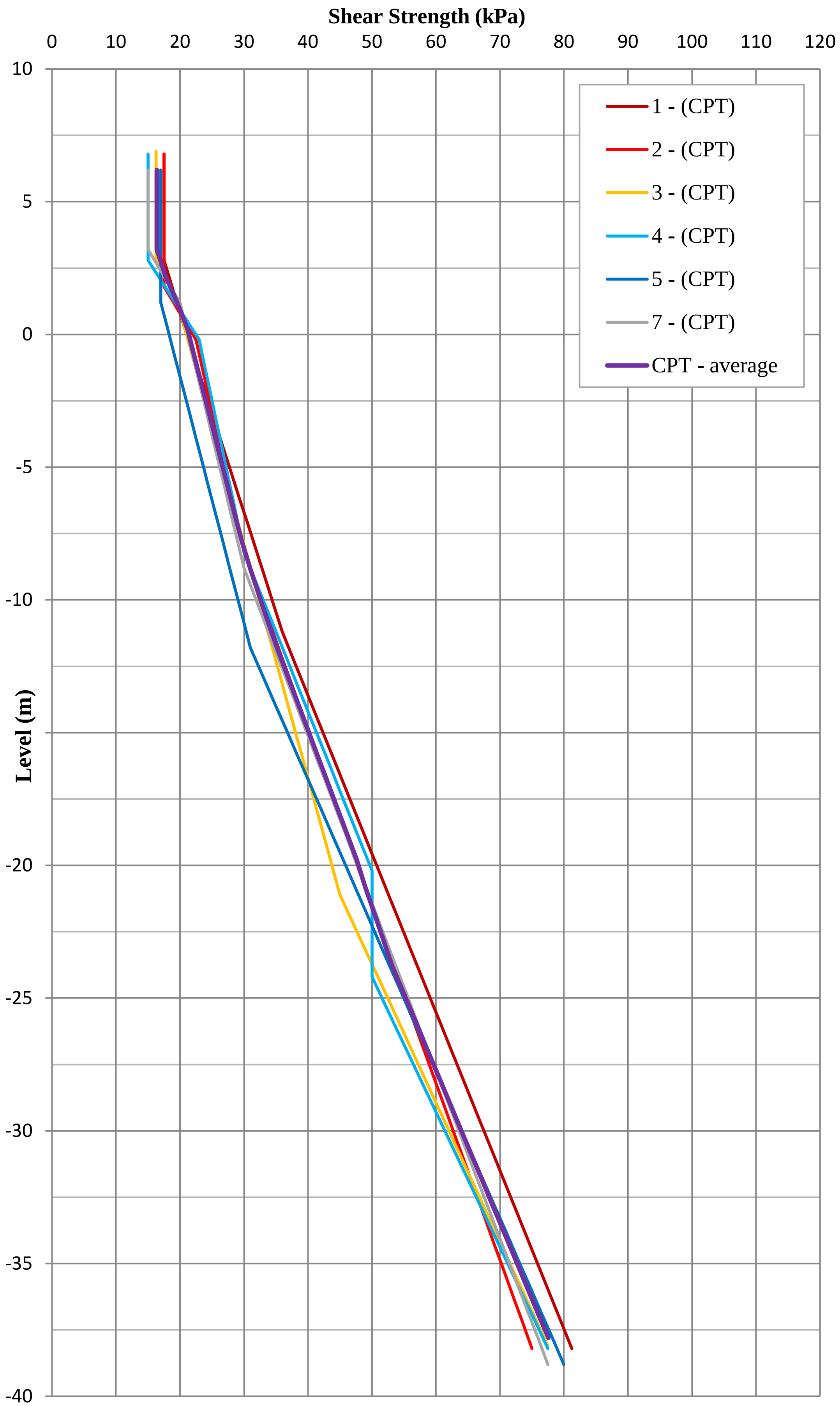
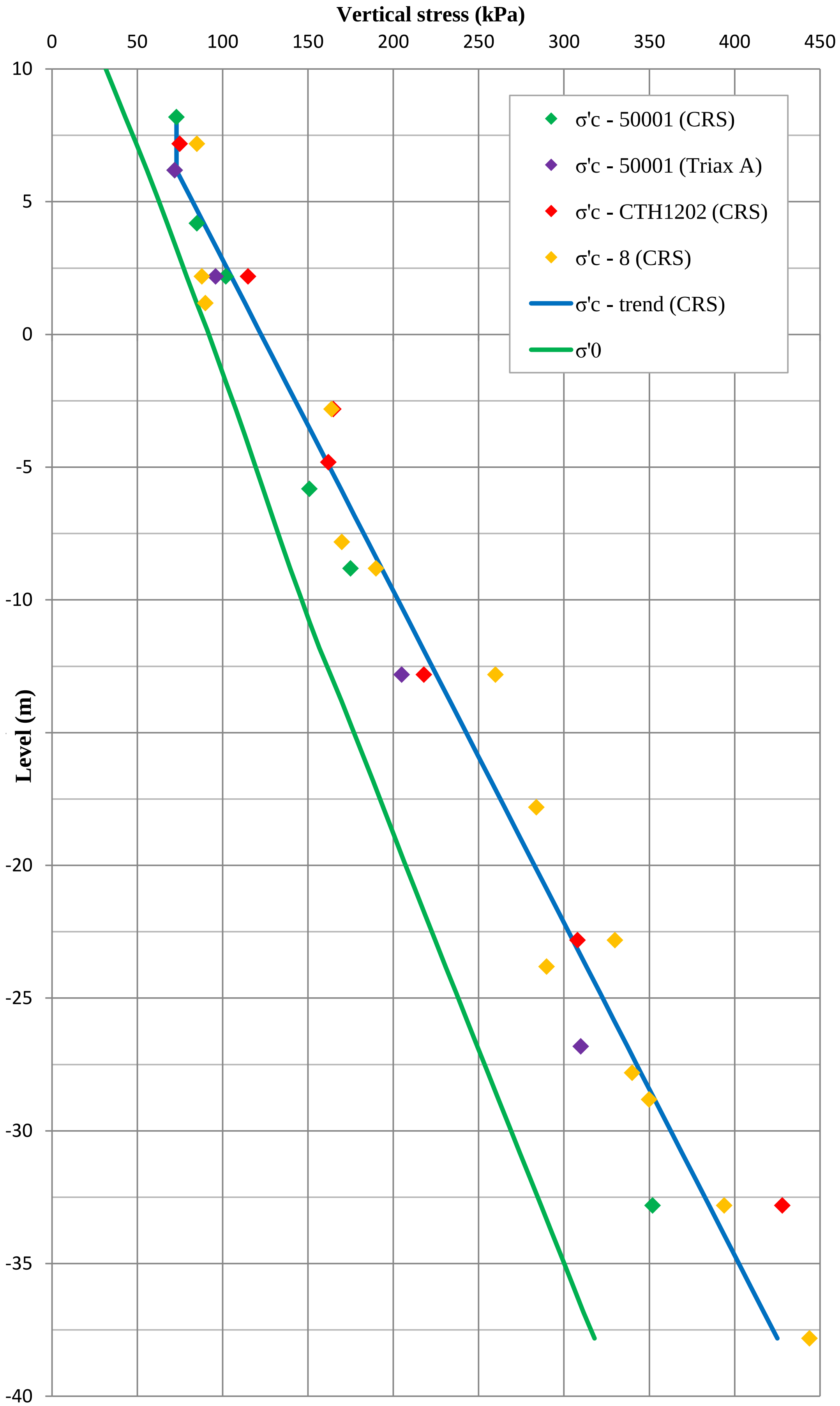
12 Appendix

- Appendix 1 - Gothenburg Central Station
- Appendix 2 - Casino Cosmopol
- Appendix 3 - Marieholm Tunnel
- Appendix 4 - Alelyckan
- Appendix 5 - E45 Agnesberg - Bohus
- Appendix 6 - Brodalsbäcken
- Appendix 7 - Kvibergsbäcken

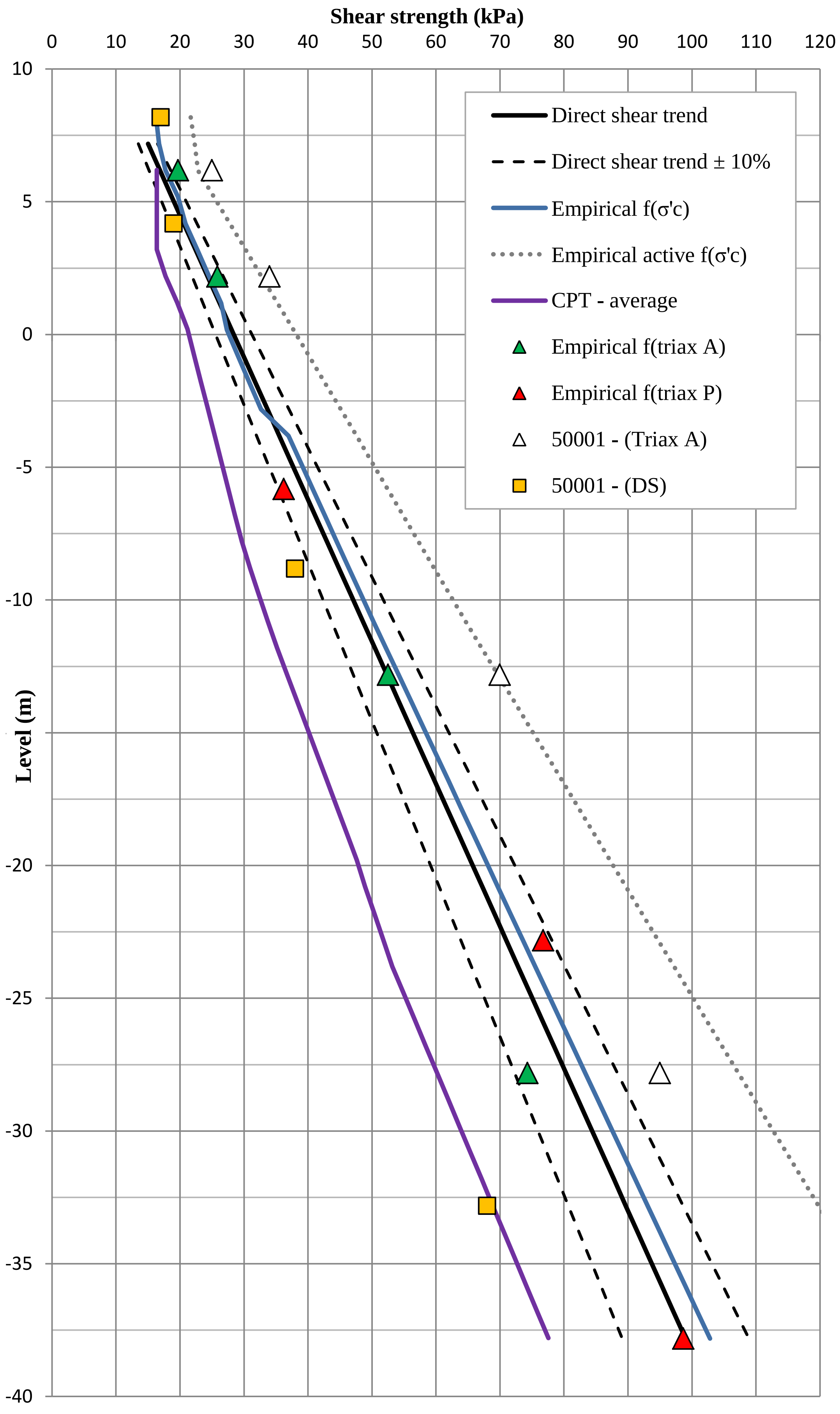
Borehole plan

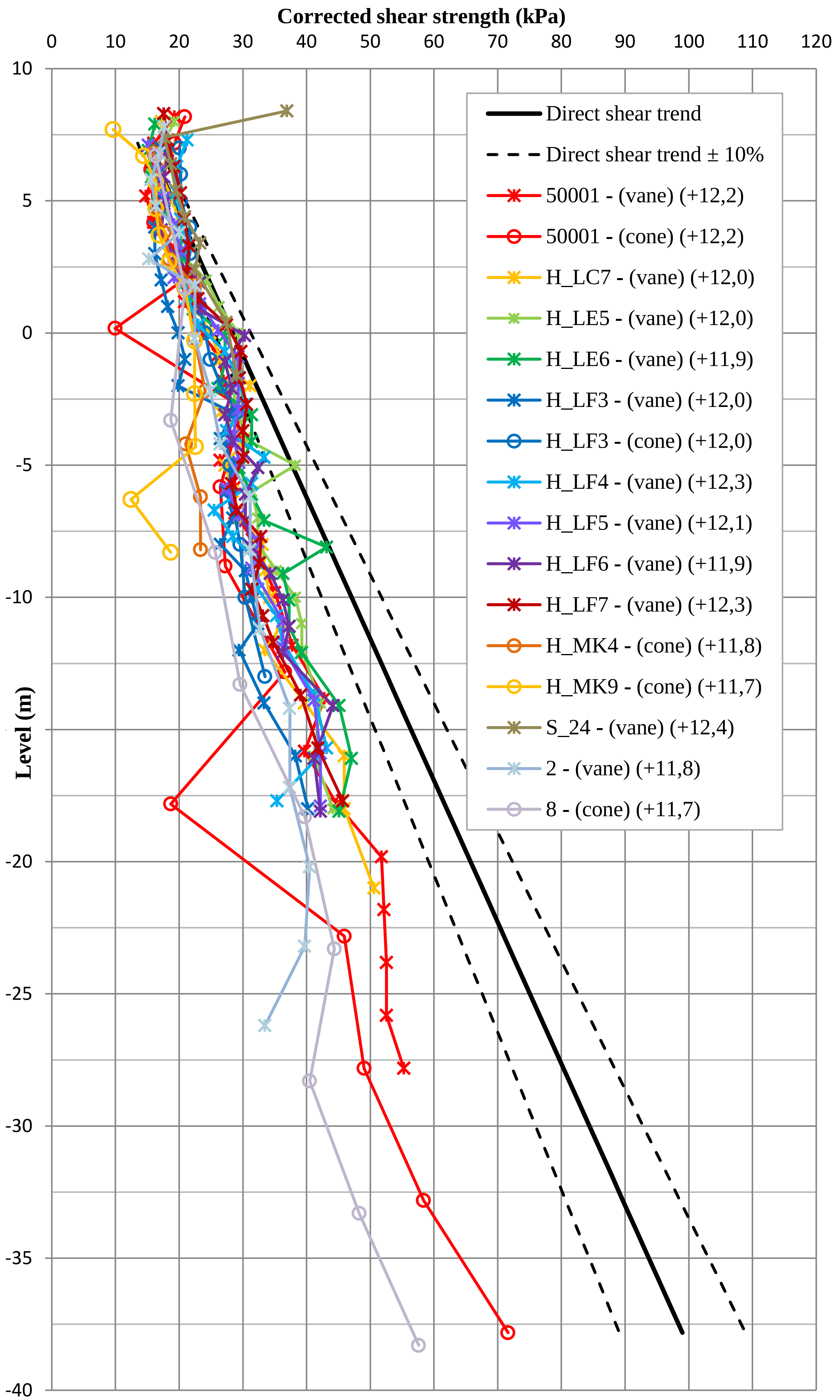


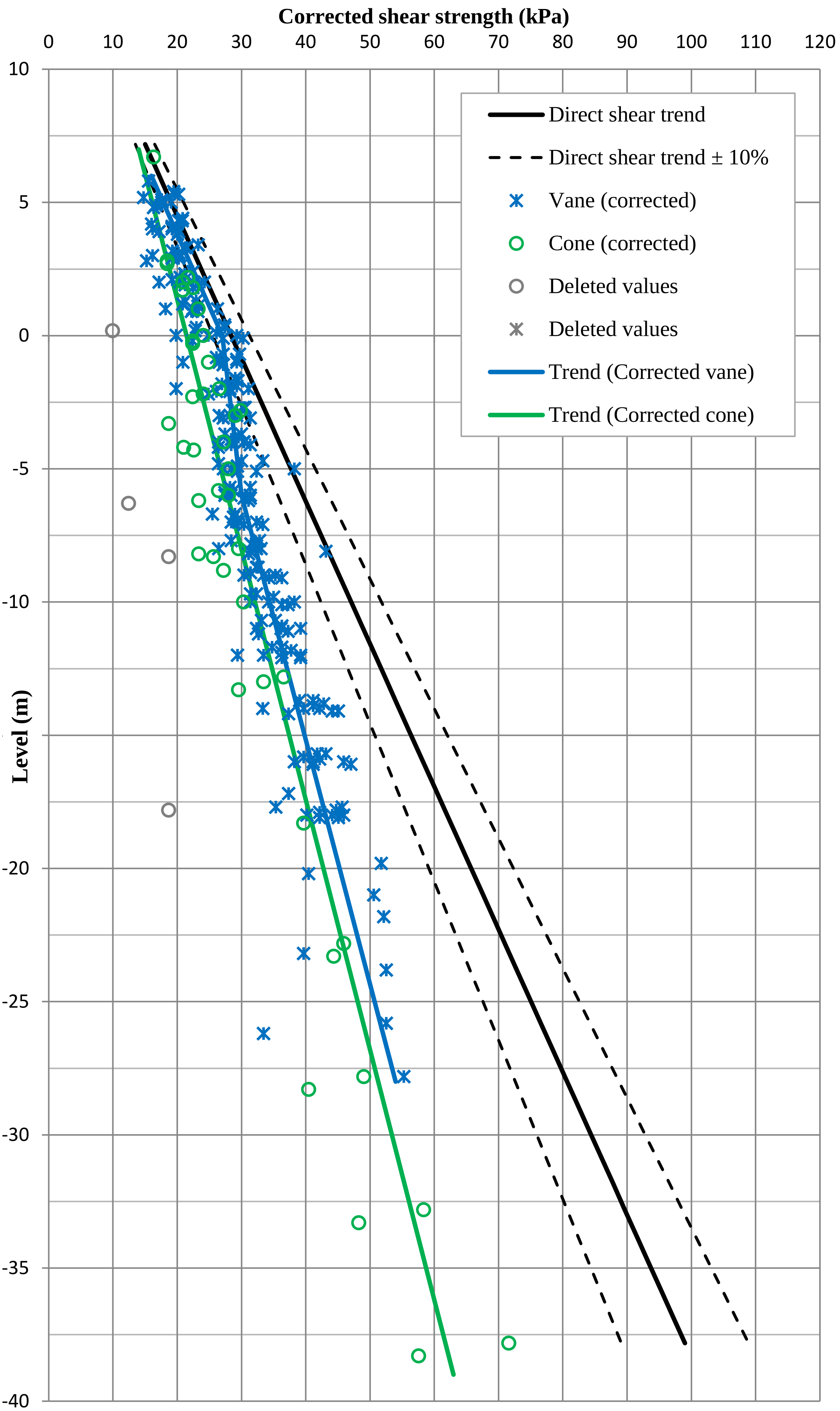

Shear Strength (kPa)
0
10
20
30
40
50
60
70
80
90
100
110
120
Level (m)
10
5
0
-5
-10
-20
-25
-30
-35
-40
1 - (CPT)
2 - (CPT)
3 - (CPT)
4 - (CPT)
5 - (CPT)
7 - (CPT)
CPT - average



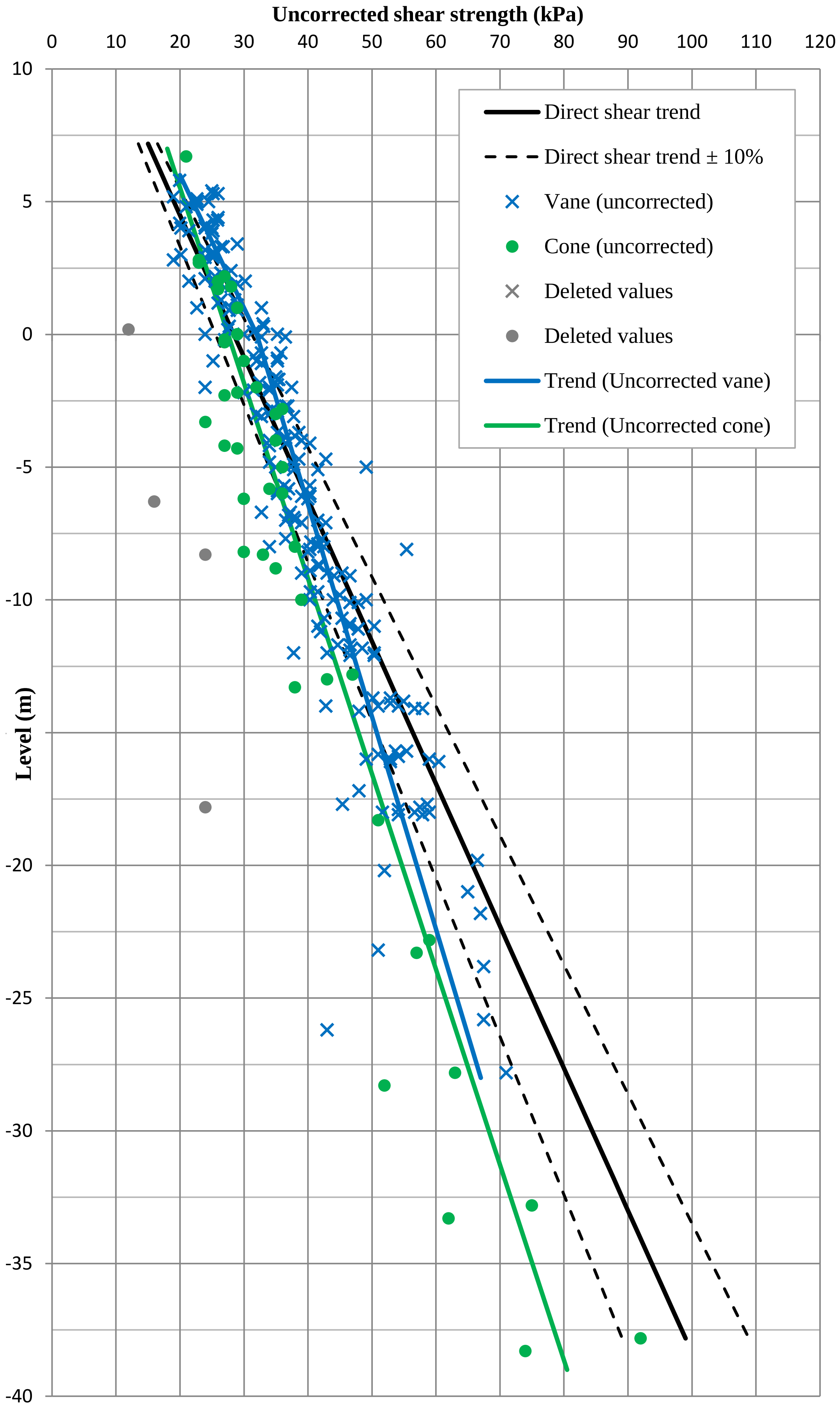
Vertical stress (kPa)
0
50
100
150
200
250
300
350
400
450
10
5
0
-5
-10
-20
-25
-30
-35
-40
Level (m)
σ'c - 50001 (CRS)
σ'c - 50001 (Triax A)
σ'c - CTH1202 (CRS)
σ'c - 8 (CRS)
σ'c - trend (CRS)
σ'0


Shear strength (kPa)
0
10
20
30
40
50
60
70
80
90
100
110
120
Level (m)
10
5
0
-5
-10
-20
-25
-30
-35
-40
Direct shear trend
Direct shear trend ± 10%
Empirical f(σ'c)
Empirical active f(σ'c)
CPT - average
Empirical f(triax A)
Empirical f(triax P)
50001 - (Triax A)
50001 - (DS)

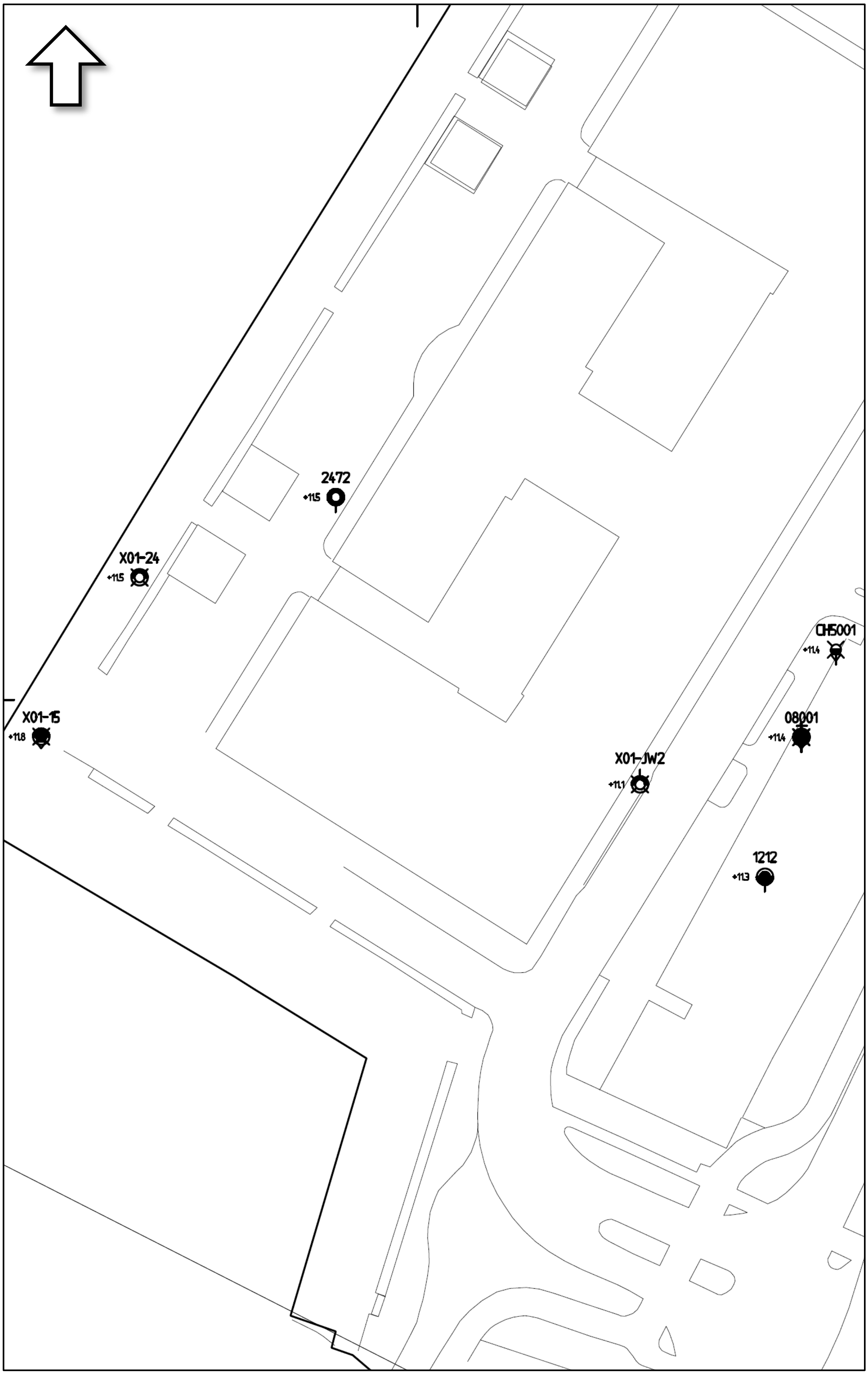

Corrected shear strength (kPa)
0 10 20 30 40 50 60 70 80 90 100 110 120
Level (m)
10 5 0 -5 -10 -15 -20 -25 -30 -35 -40
Direct shear trend
Direct shear trend ± 10%
50001 - (vane) (+12,2)
50001 - (cone) (+12,2)
H_LC7 - (vane) (+12,0)
H_LE5 - (vane) (+12,0)
H_LE6 - (vane) (+11,9)
H_LF3 - (vane) (+12,0)
H_LF3 - (cone) (+12,0)
H_LF4 - (vane) (+12,3)
H_LF5 - (vane) (+12,1)
H_LF6 - (vane) (+11,9)
H_LF7 - (vane) (+12,3)
H_MK4 - (cone) (+11,8)
H_MK9 - (cone) (+11,7)
S_24 - (vane) (+12,4)
2 - (vane) (+11,8)
8 - (cone) (+11,7)

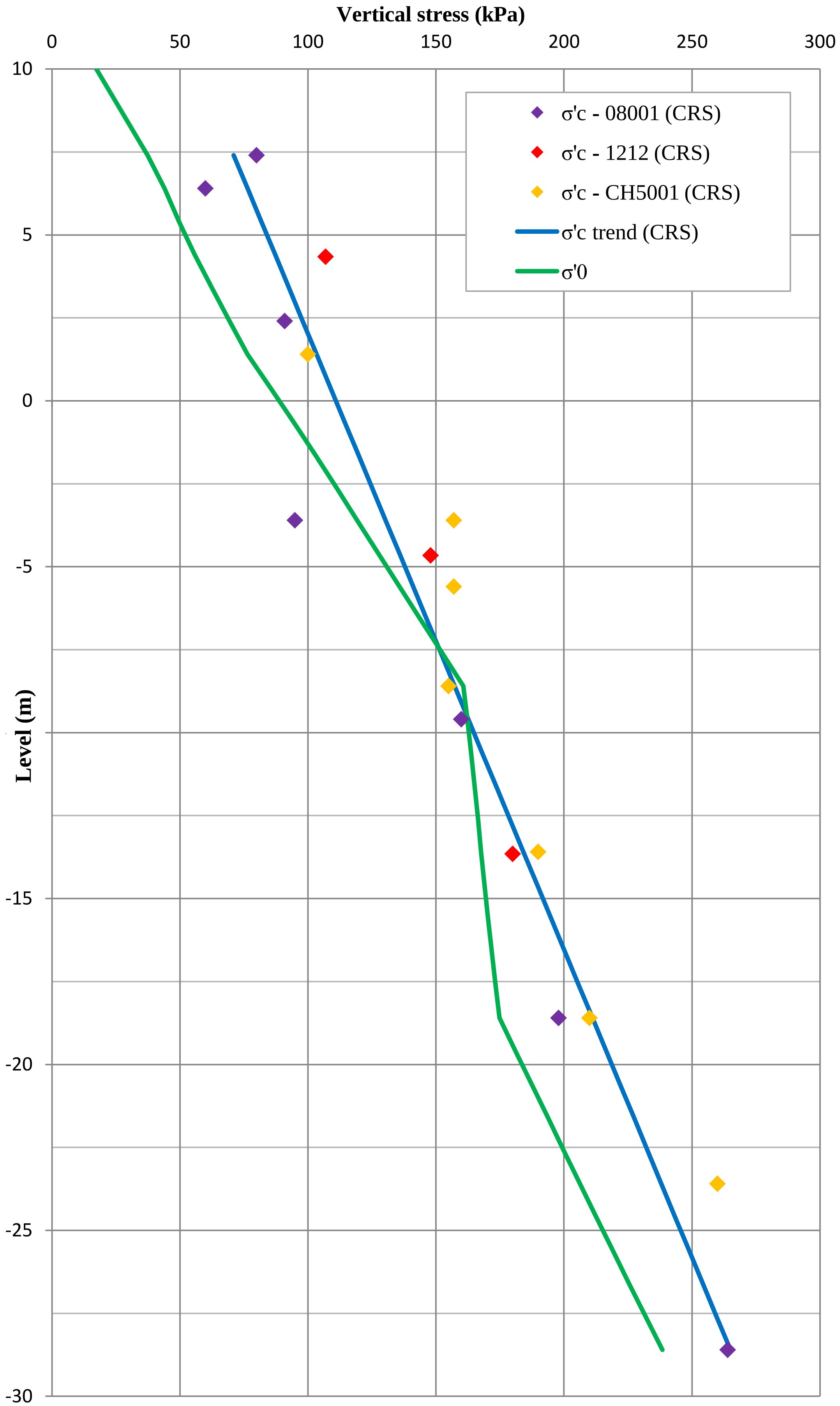


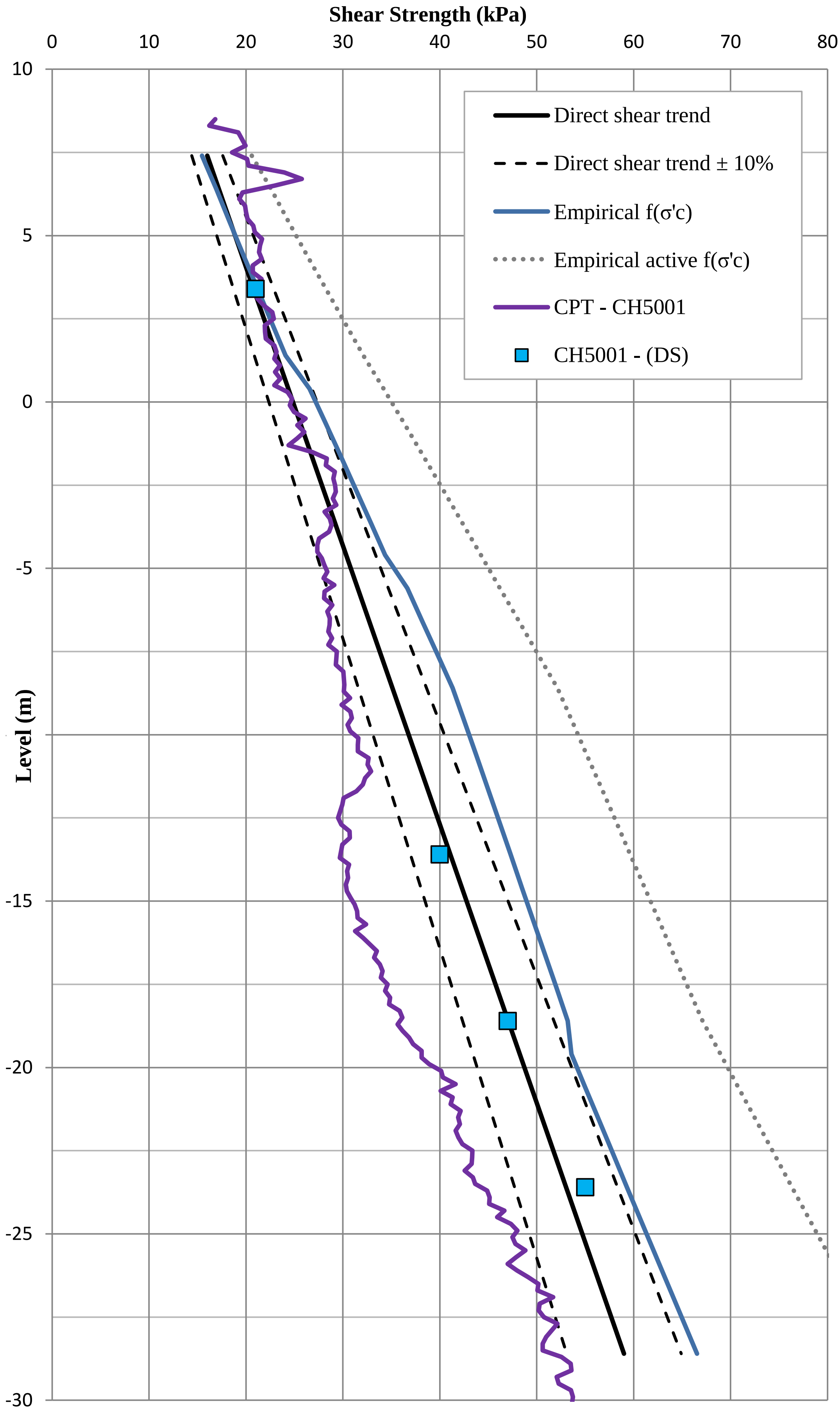
Corrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
90
100
110
120
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
Level (m)
Direct shear trend
Direct shear trend ± 10%
Vane (corrected)
Cone (corrected)
Deleted values
Deleted values
Trend (Corrected vane)
Trend (Corrected cone)


Uncorrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
90
100
110
120
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
Level (m)
Direct shear trend
Direct shear trend ± 10%
Vane (uncorrected)
Cone (uncorrected)
Deleted values
Deleted values
Trend (Uncorrected vane)
Trend (Uncorrected cone)

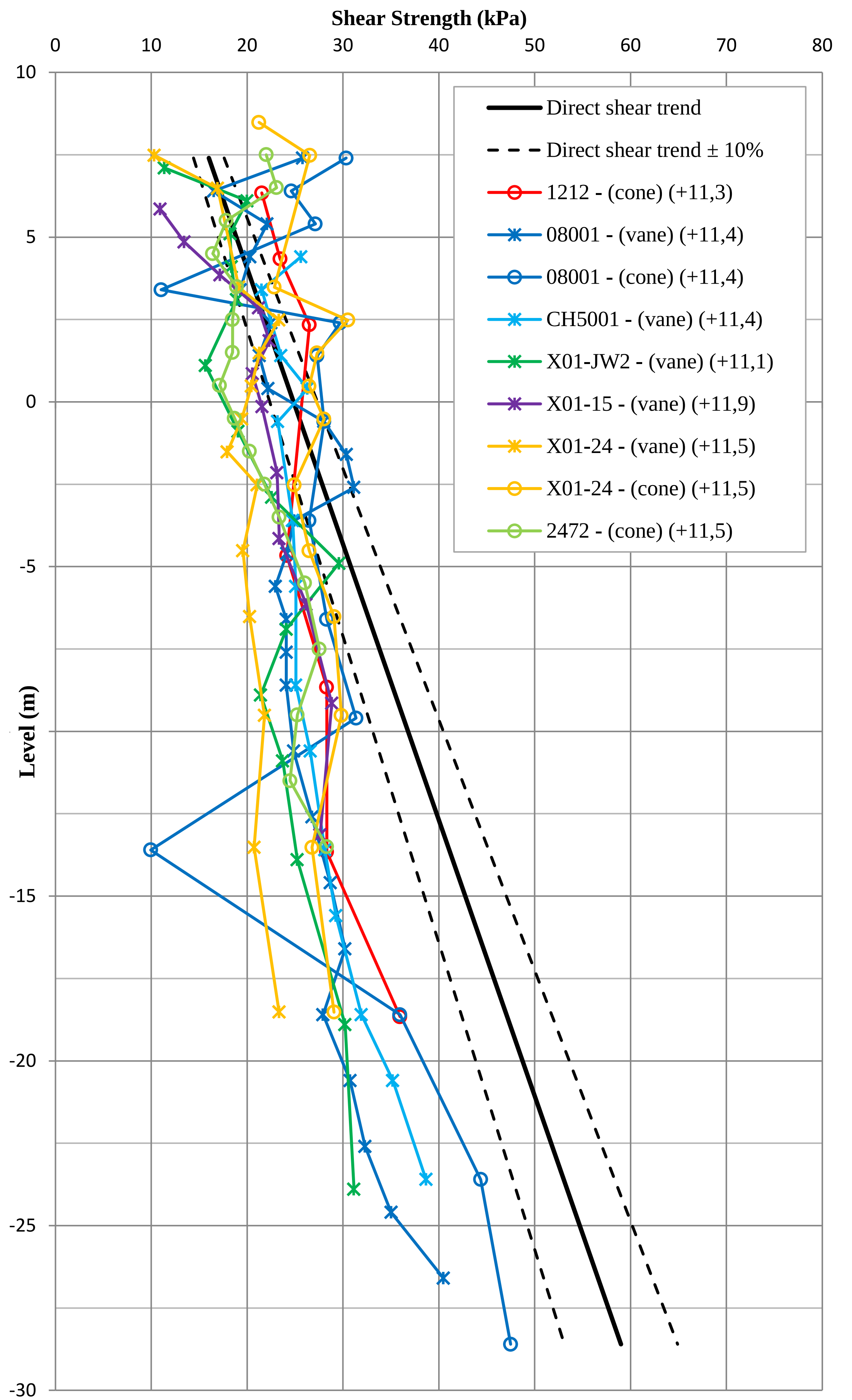
Borehole plan



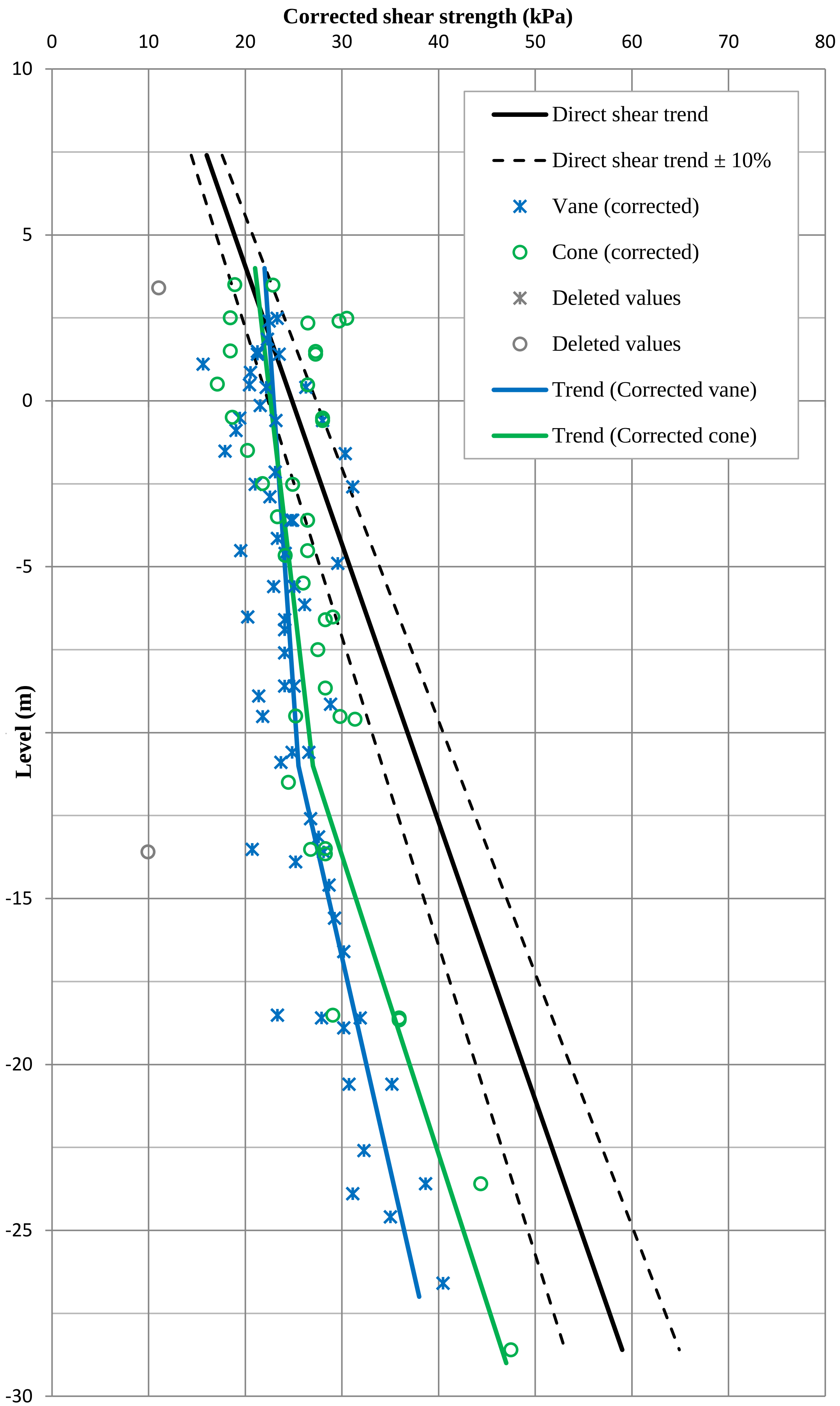

Vertical stress (kPa)
0
50
100
150
200
250
300
10
5
0
-5
-15
-20
-25
-30
Level (m)
σ'c - 08001 (CRS)
σ'c - 1212 (CRS)
σ'c - CH5001 (CRS)
σ'c trend (CRS)
σ'0

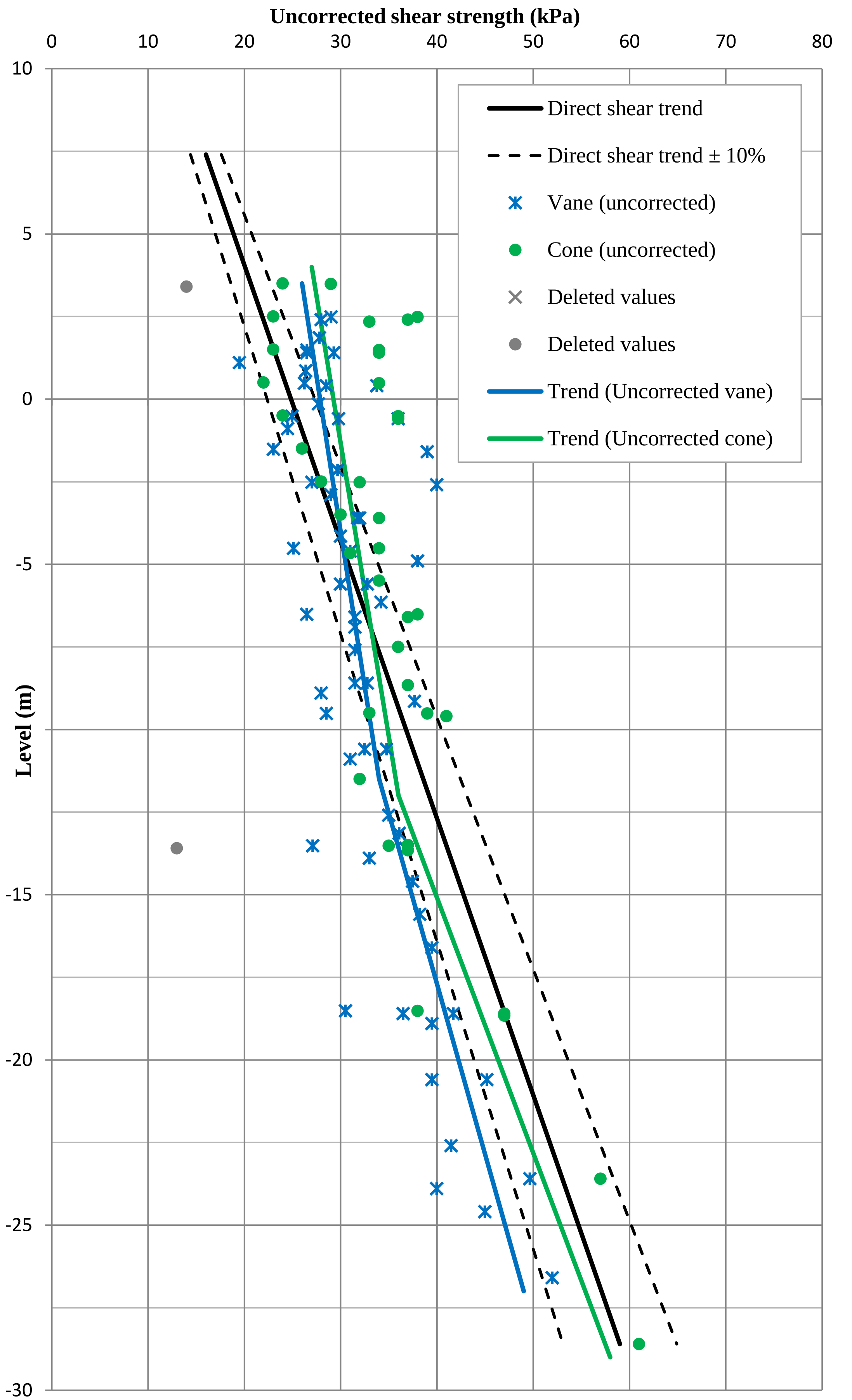


Shear Strength (kPa)
0
10
20
30
40
50
60
70
80
10
5
0
-5
-15
-20
-25
-30
Level (m)
Direct shear trend
Direct shear trend ± 10%
Empirical f(σ'c)
Empirical active f(σ'c)
CPT - CH5001
CH5001 - (DS)

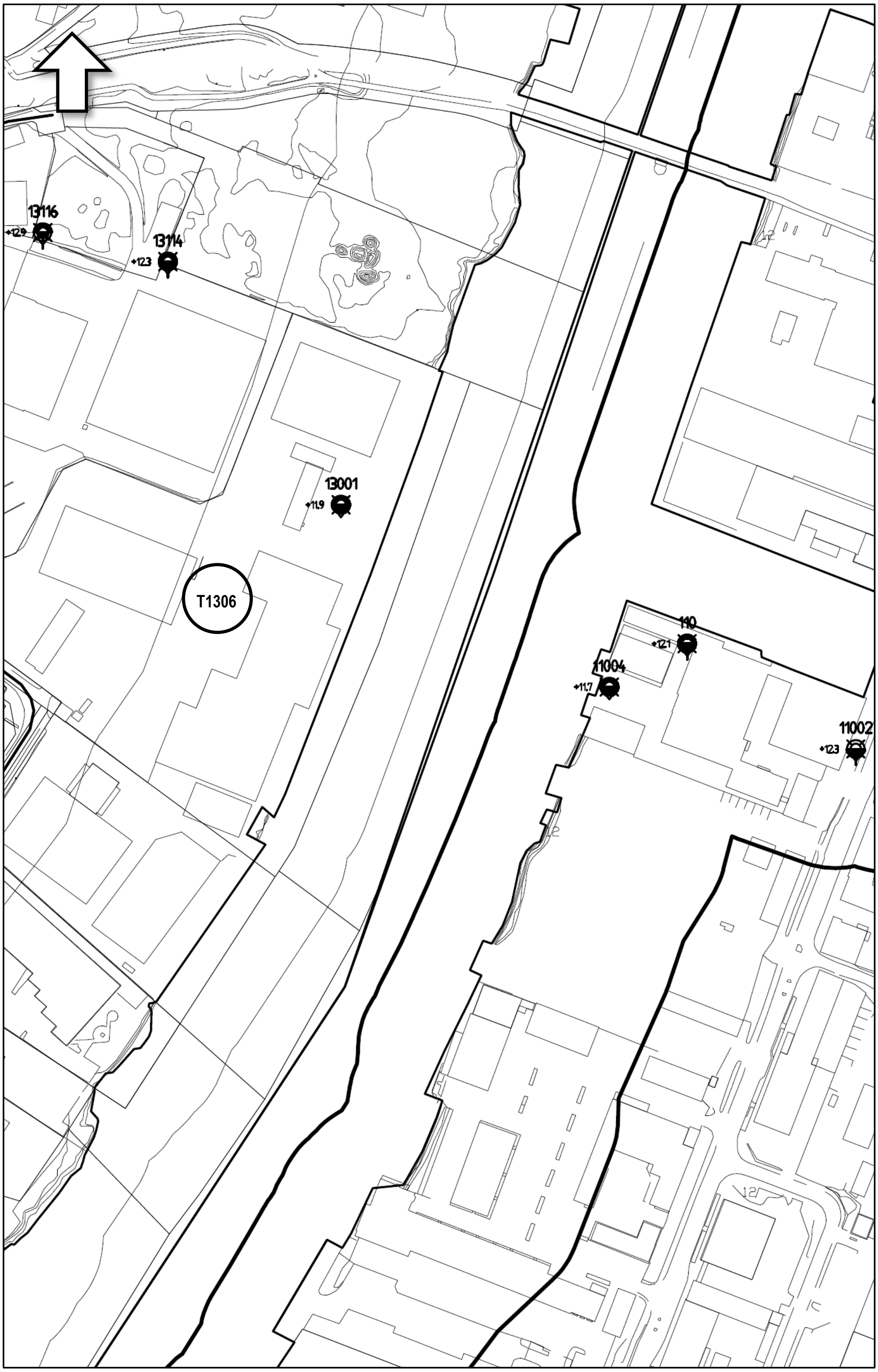


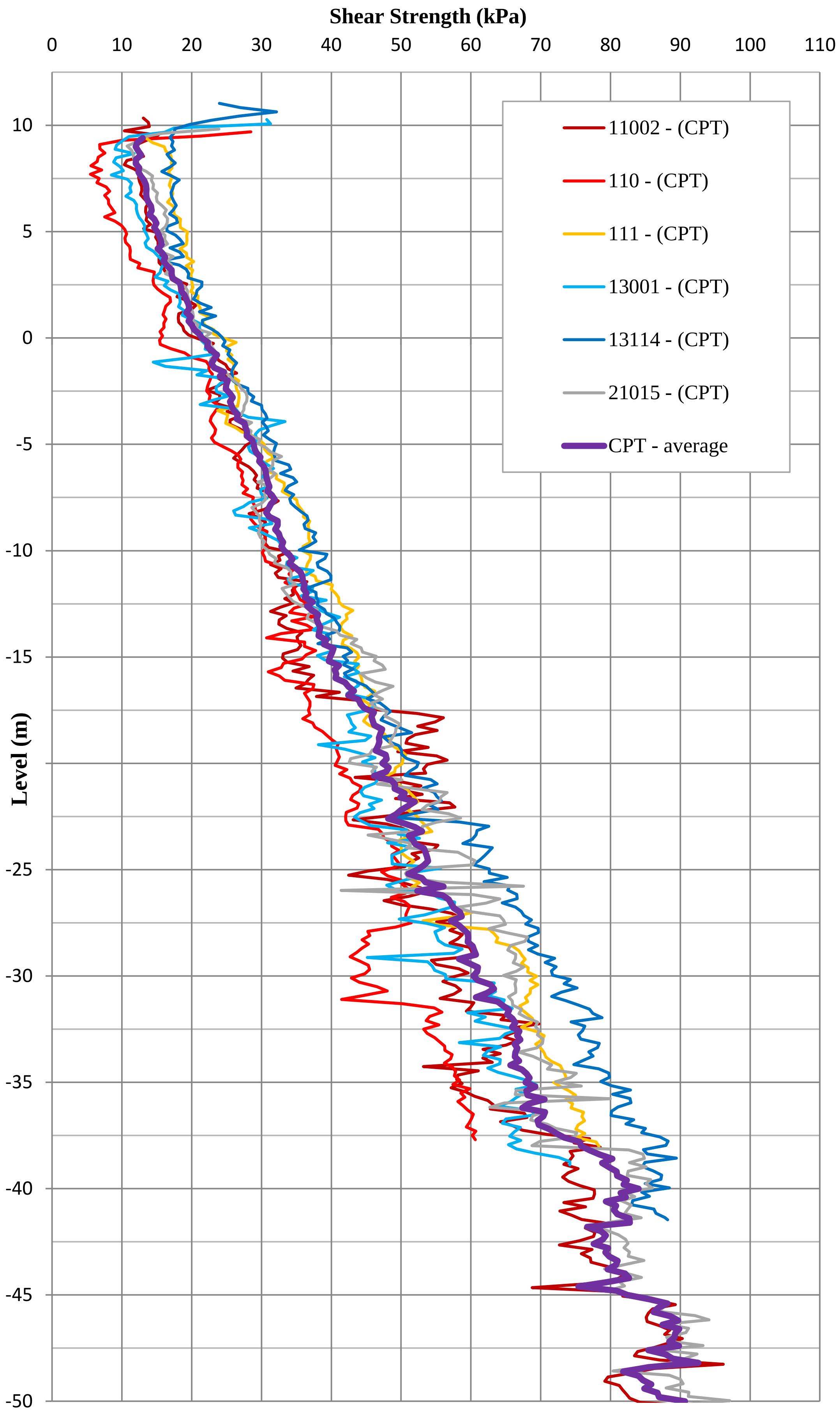
Shear Strength (kPa)
0
10
20
30
40
50
60
70
80
Level (m)
10
5
0
-5
-10
-15
-20
-25
-30
Direct shear trend
Direct shear trend ± 10%
1212 - (cone) (+11,3)
08001 - (vane) (+11,4)
08001 - (cone) (+11,4)
CH5001 - (vane) (+11,4)
X01-JW2 - (vane) (+11,1)
X01-15 - (vane) (+11,9)
X01-24 - (vane) (+11,5)
X01-24 - (cone) (+11,5)
2472 - (cone) (+11,5)


Corrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
10
5
0
-5
-15
-20
-25
-30
Level (m)
Direct shear trend
Direct shear trend ± 10%
Vane (corrected)
Cone (corrected)
Deleted values
Deleted values
Trend (Corrected vane)
Trend (Corrected cone)

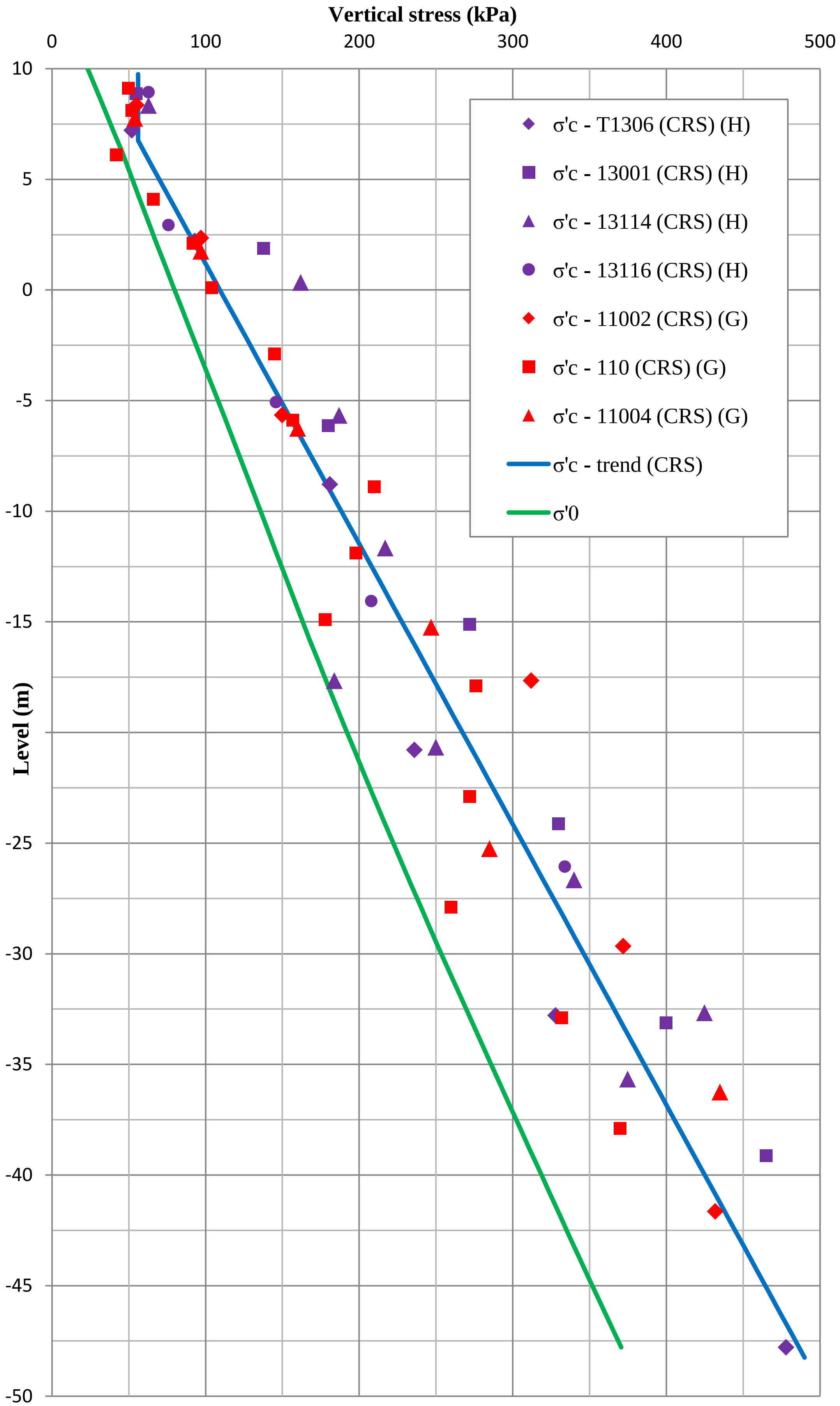

Uncorrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
10
5
0
-5
-15
-20
-25
-30
Level (m)
Direct shear trend
Direct shear trend ± 10%
Vane (uncorrected)
Cone (uncorrected)
Deleted values
Deleted values
Trend (Uncorrected vane)
Trend (Uncorrected cone)

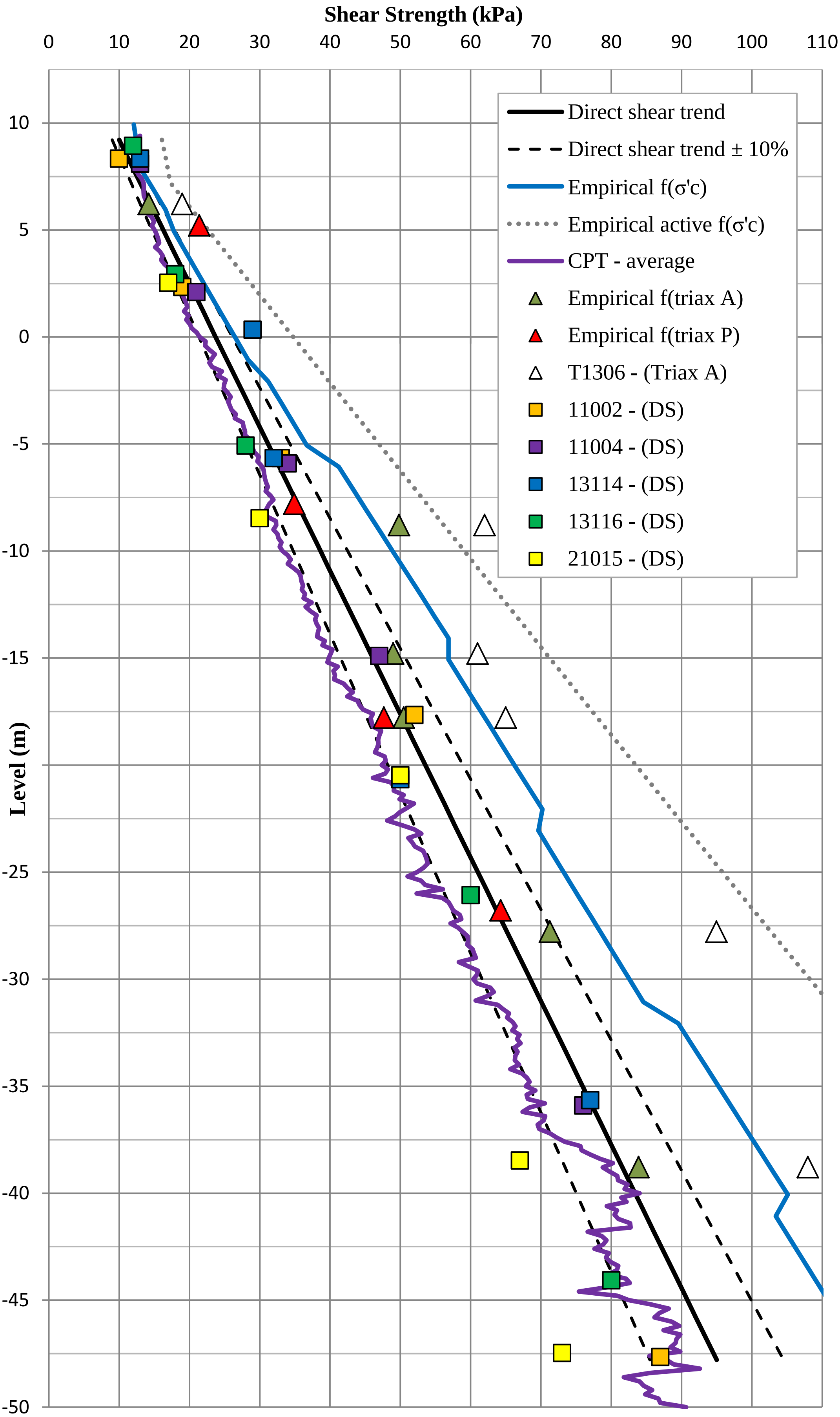
Borehole plan

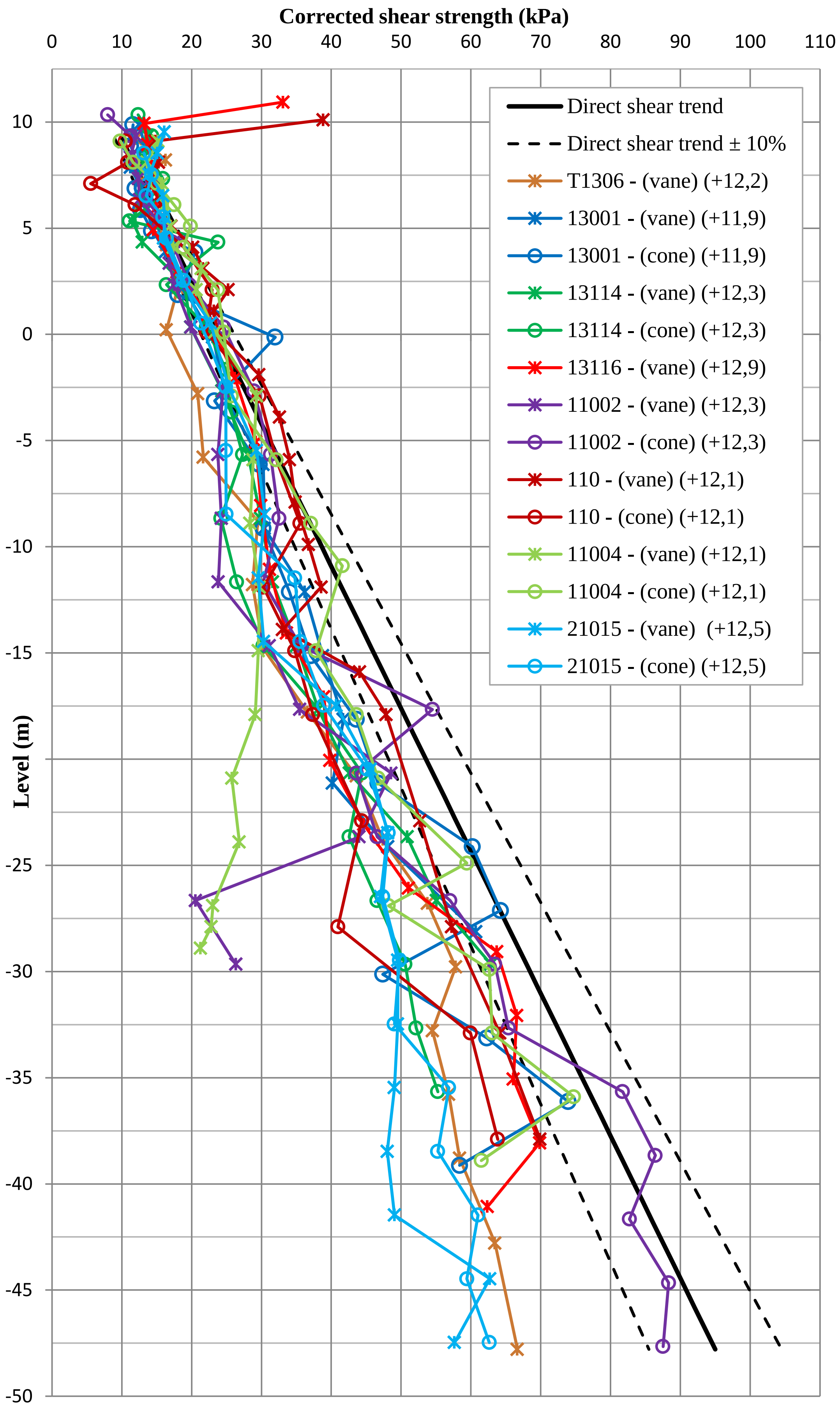




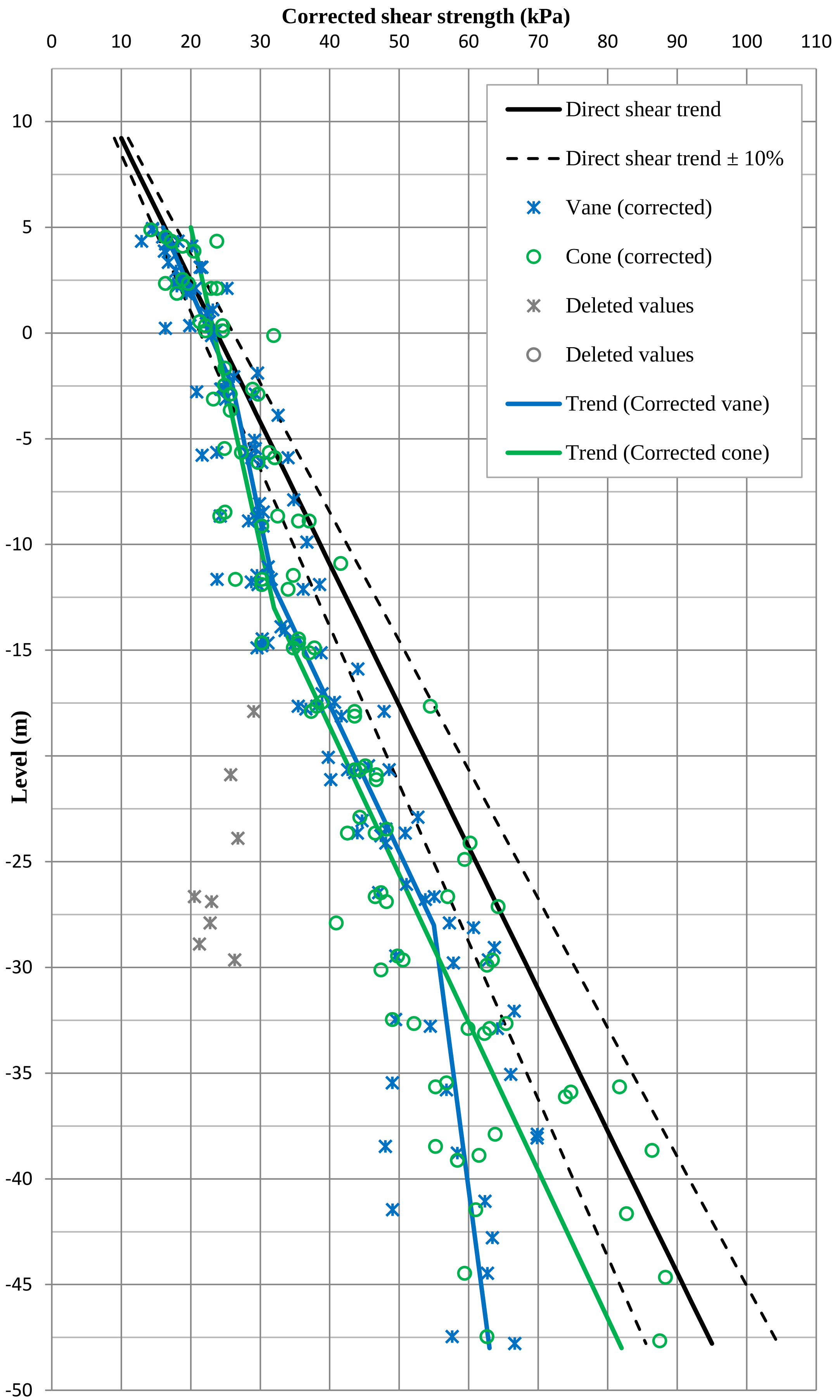
Shear Strength (kPa)
0
10
20
30
40
50
60
70
80
90
100
110
Level (m)
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
-50
11002 - (CPT)
110 - (CPT)
111 - (CPT)
13001 - (CPT)
13114 - (CPT)
21015 - (CPT)
CPT - average

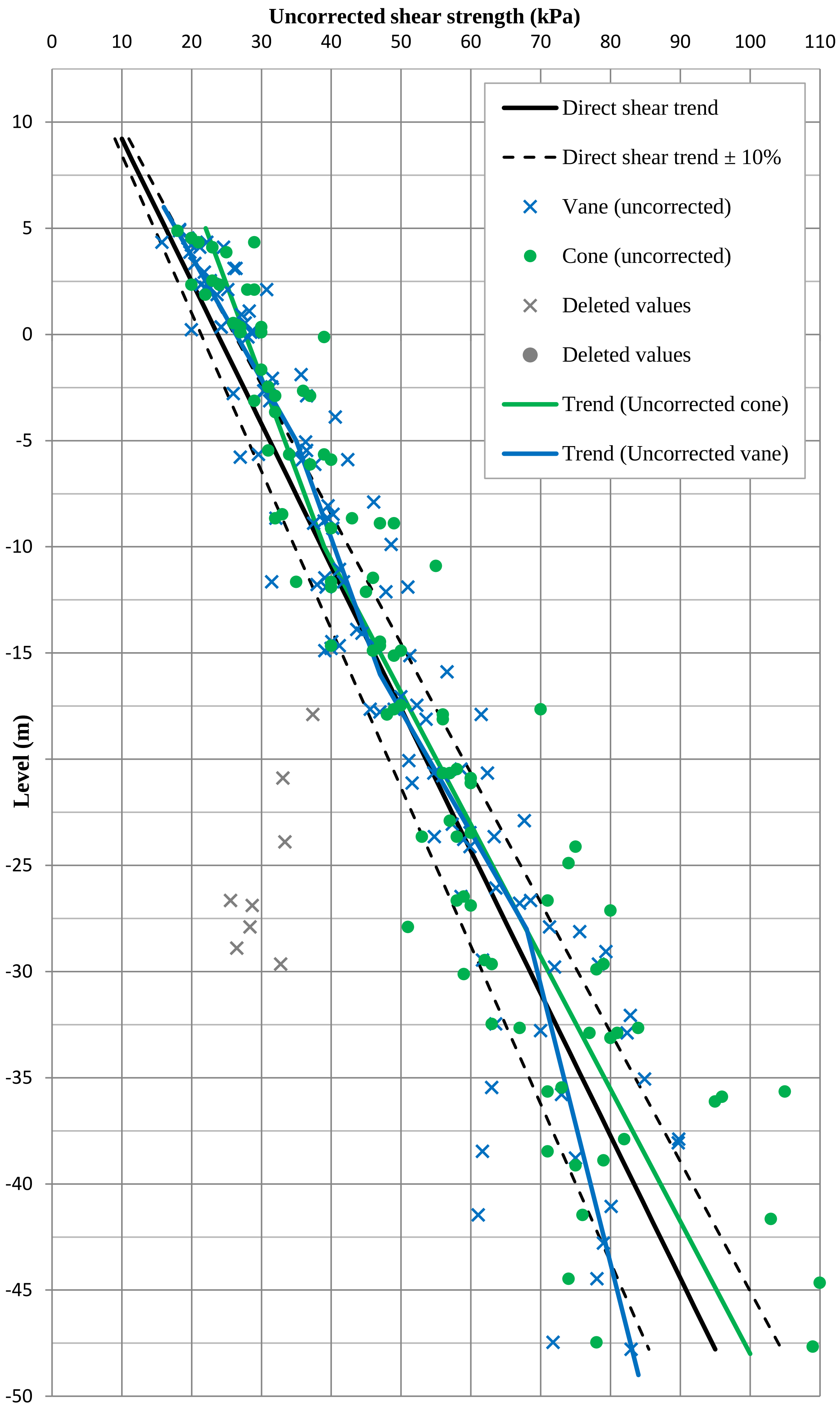

Vertical stress (kPa)
0
100
200
300
400
500
10
5
0
-5
-10
-15
-25
-30
-35
-40
-45
-50
Level (m)
σ'c - T1306 (CRS) (H)
σ'c - 13001 (CRS) (H)
σ'c - 13114 (CRS) (H)
σ'c - 13116 (CRS) (H)
σ'c - 11002 (CRS) (G)
σ'c - 110 (CRS) (G)
σ'c - 11004 (CRS) (G)
σ'c - trend (CRS)
σ'0


Shear Strength (kPa)
0
10
20
30
40
50
60
70
80
90
100
110
Level (m)
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
-50
Direct shear trend
Direct shear trend ± 10%
Empirical f(σ'c)
Empirical active f(σ'c)
CPT - average
Empirical f(triax A)
Empirical f(triax P)
T1306 - (Triax A)
11002 - (DS)
11004 - (DS)
13114 - (DS)
13116 - (DS)
21015 - (DS)

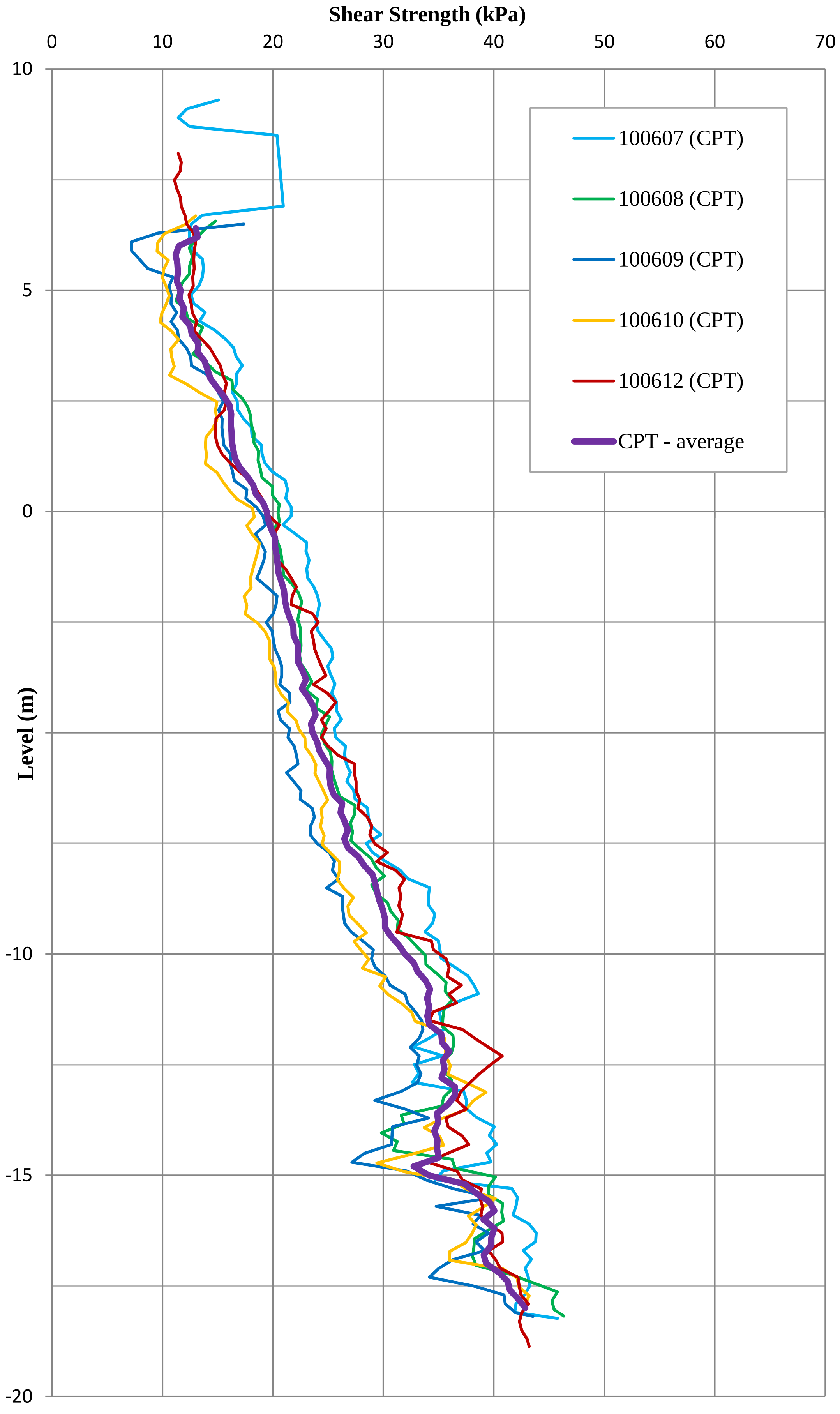


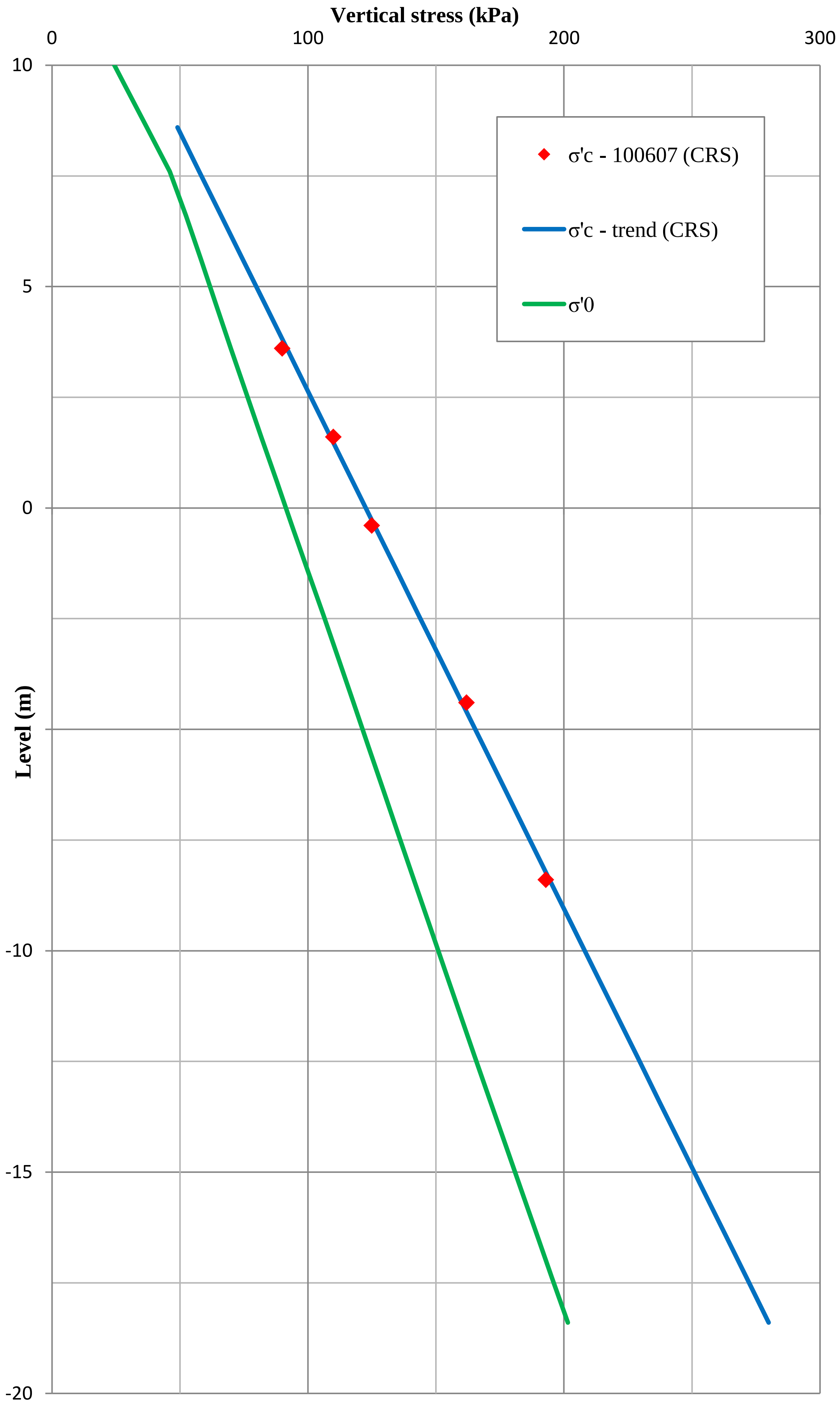
Corrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
90
100
110
Level (m)
10
5
0
-5
-10
-15
-25
-30
-35
-40
-45
-50
Direct shear trend
Direct shear trend ± 10%
T1306 - (vane) (+12,2)
13001 - (vane) (+11,9)
13001 - (cone) (+11,9)
13114 - (vane) (+12,3)
13114 - (cone) (+12,3)
13116 - (vane) (+12,9)
11002 - (vane) (+12,3)
11002 - (cone) (+12,3)
110 - (vane) (+12,1)
110 - (cone) (+12,1)
11004 - (vane) (+12,1)
11004 - (cone) (+12,1)
21015 - (vane) (+12,5)
21015 - (cone) (+12,5)

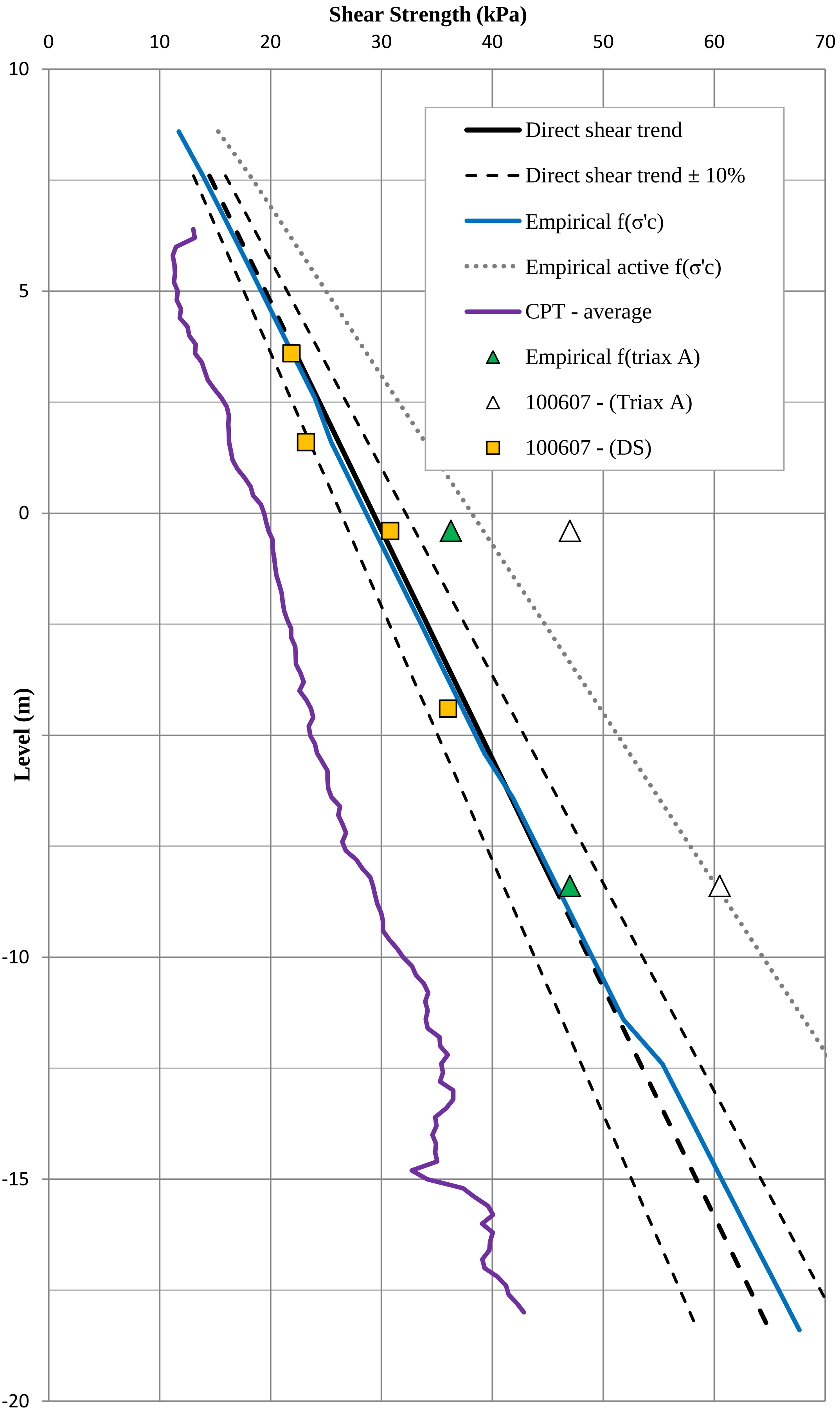

Corrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
90
100
110
Level (m)
10
5
0
-5
-10
-15
-25
-30
-35
-40
-45
-50
Direct shear trend
Direct shear trend ± 10%
Vane (corrected)
Cone (corrected)
Deleted values
Deleted values
Trend (Corrected vane)
Trend (Corrected cone)

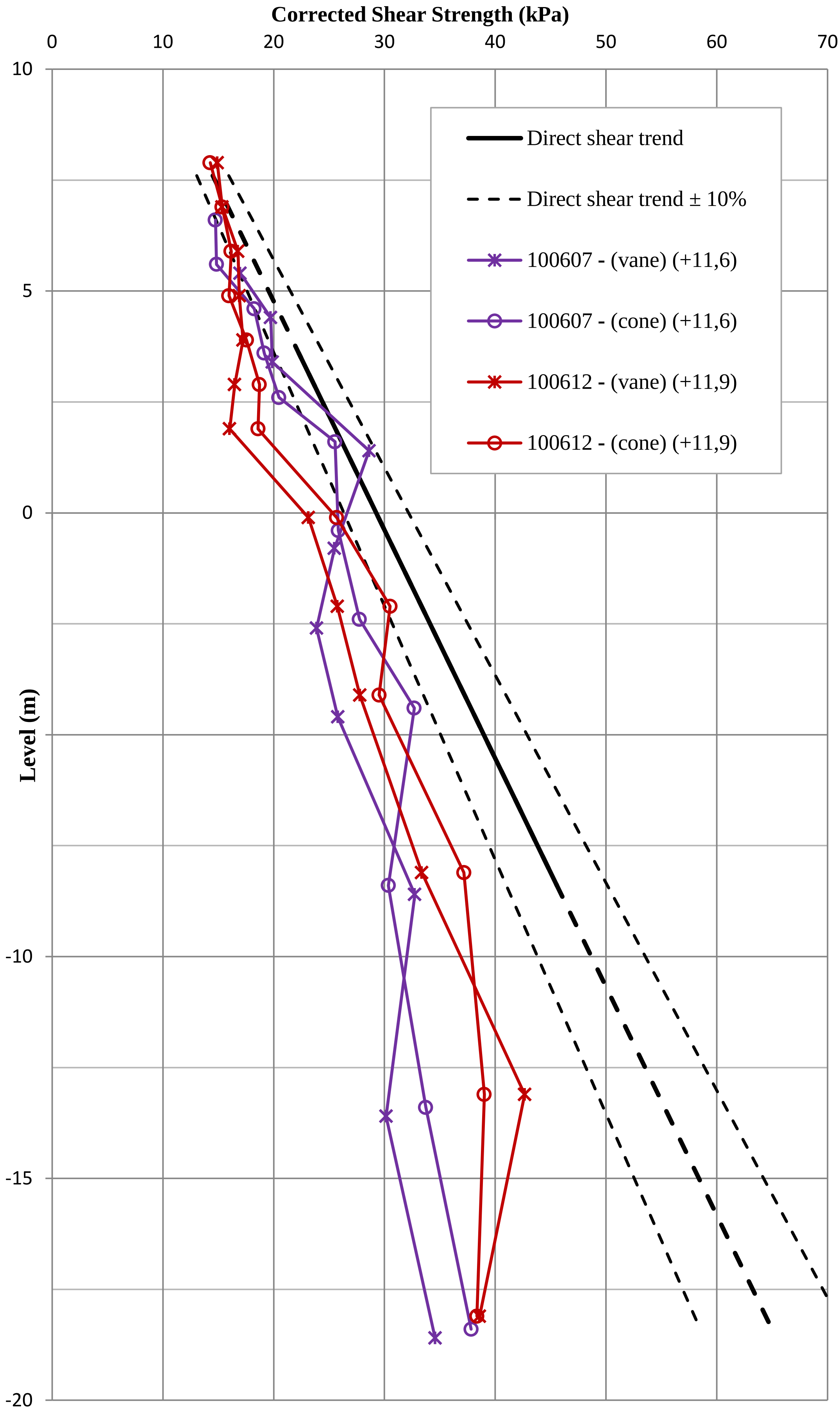

Uncorrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
90
100
110
Level (m)
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
-50
Direct shear trend
Direct shear trend ± 10%
Vane (uncorrected)
Cone (uncorrected)
Deleted values
Deleted values
Trend (Uncorrected cone)
Trend (Uncorrected vane)

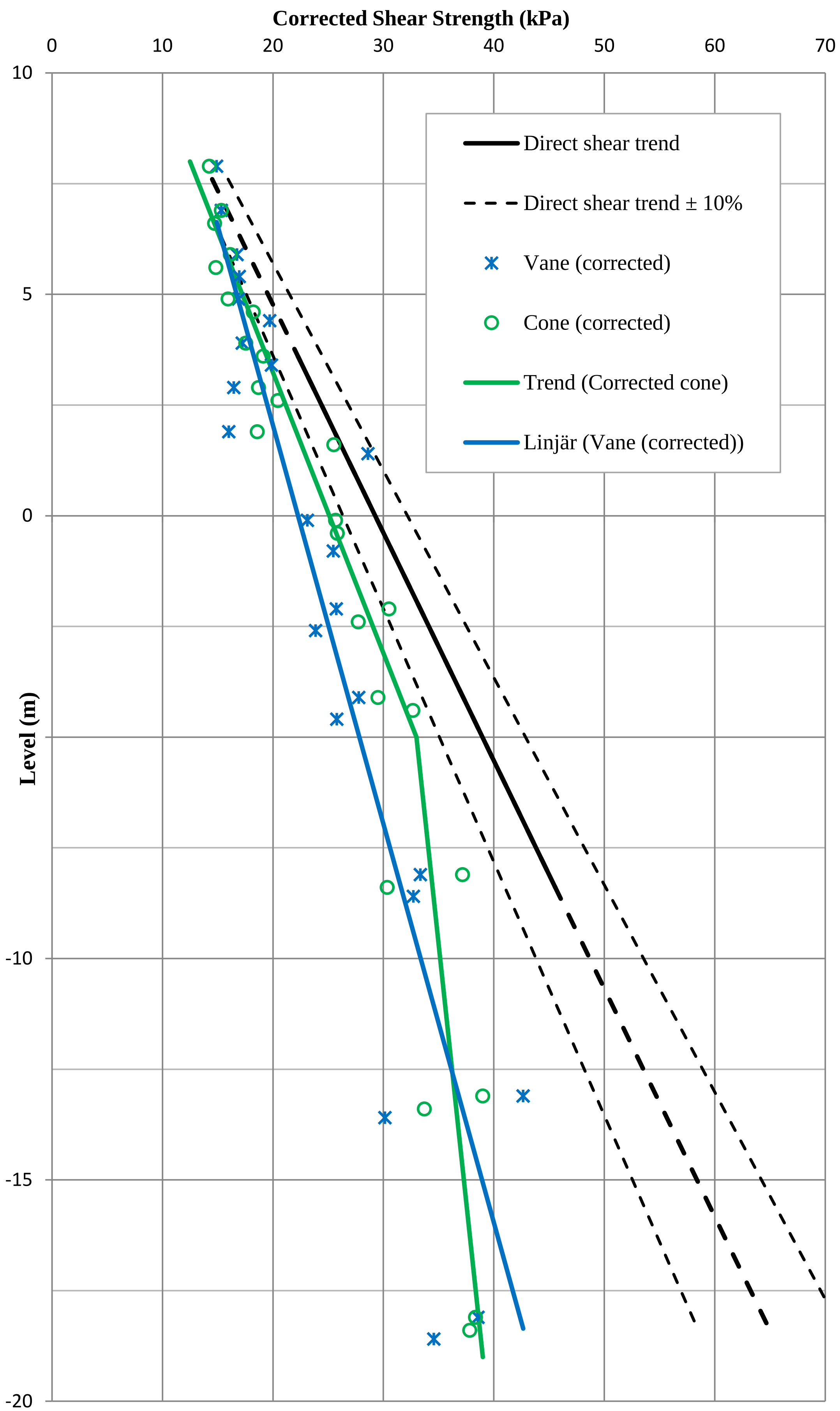
Borehole plan

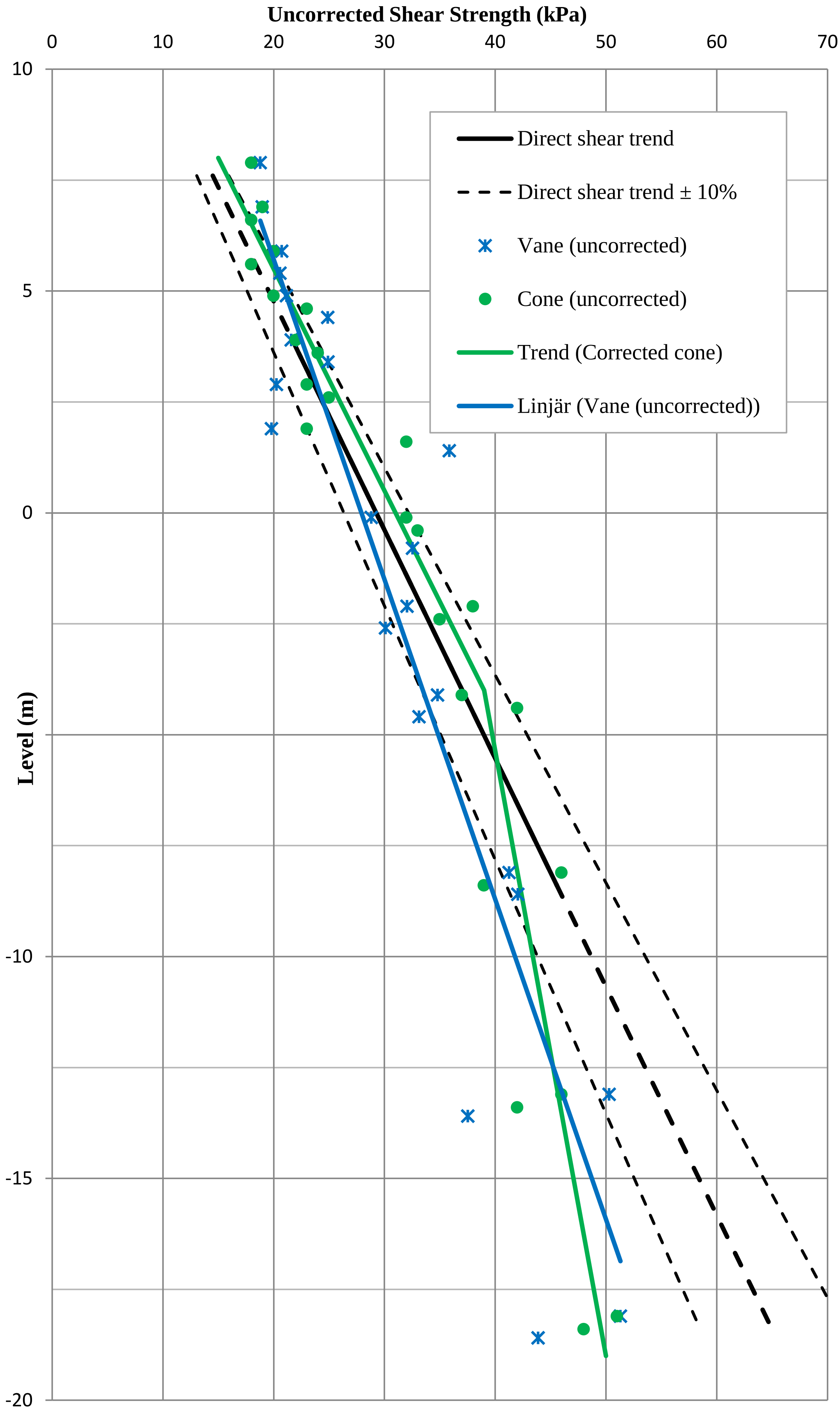

Shear Strength (kPa)
0
10
20
30
40
50
60
70
10
5
0
-5
-10
-15
-20
Level (m)
100607 (CPT)
100608 (CPT)
100609 (CPT)
100610 (CPT)
100612 (CPT)
CPT - average


Vertical stress (kPa)
0
100
200
300
10
5
0
-10
-15
-20
Level (m)
σ'c - 100607 (CRS)
σ'c - trend (CRS)
σ'0

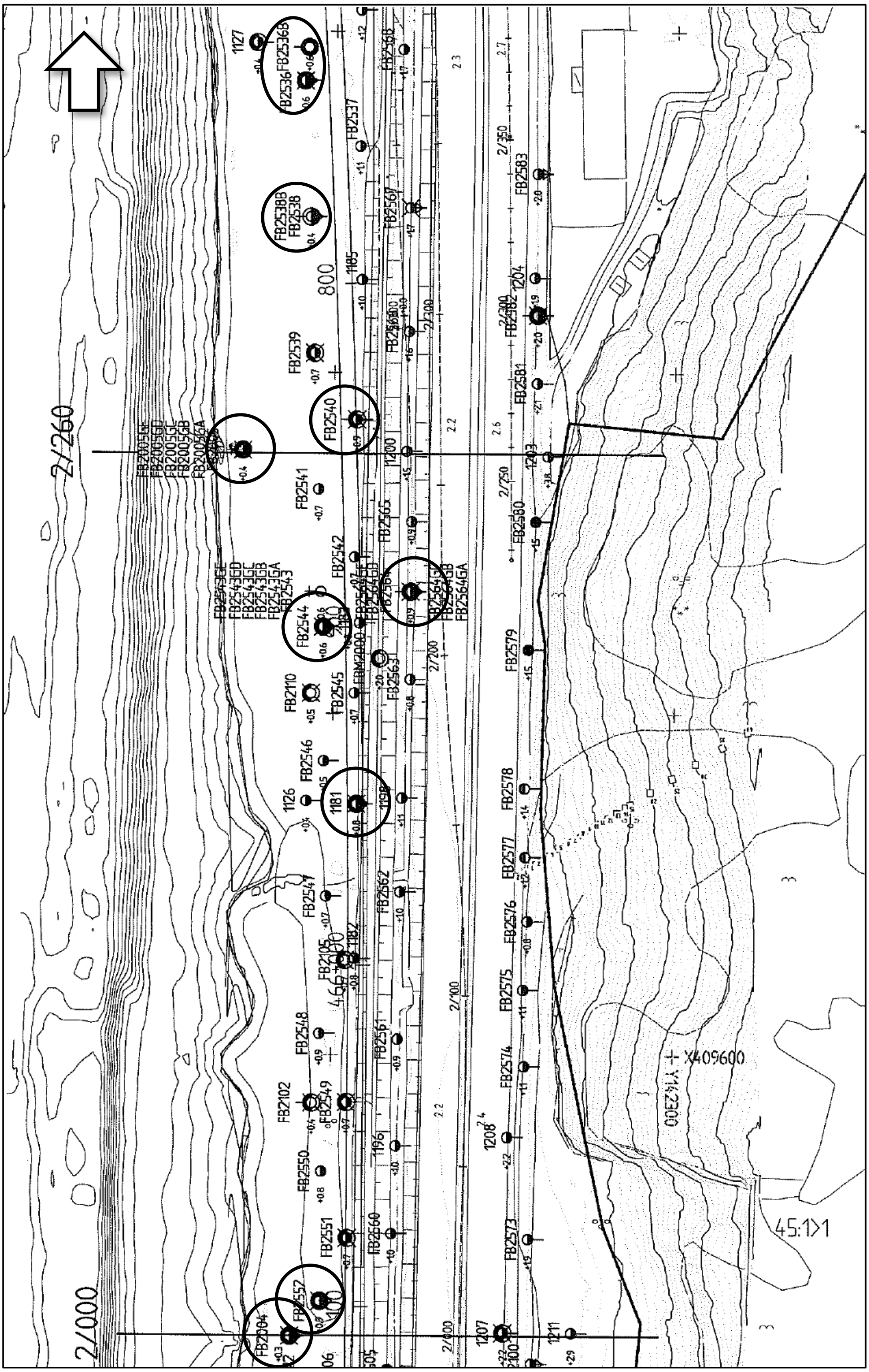

Shear Strength (kPa)
0
10
20
30
40
50
60
70
10
5
0
-5
-10
-15
-20
Level (m)
Direct shear trend
Direct shear trend ± 10%
Empirical f(σ'c)
Empirical active f(σ'c)
CPT - average
Empirical f(triax A)
100607 - (Triax A)
100607 - (DS)


Corrected Shear Strength (kPa)
0
10
20
30
40
50
60
70
10
5
0
-5
-10
-15
-20
Level (m)
Direct shear trend
Direct shear trend ± 10%
100607 - (vane) (+11,6)
100607 - (cone) (+11,6)
100612 - (vane) (+11,9)
100612 - (cone) (+11,9)

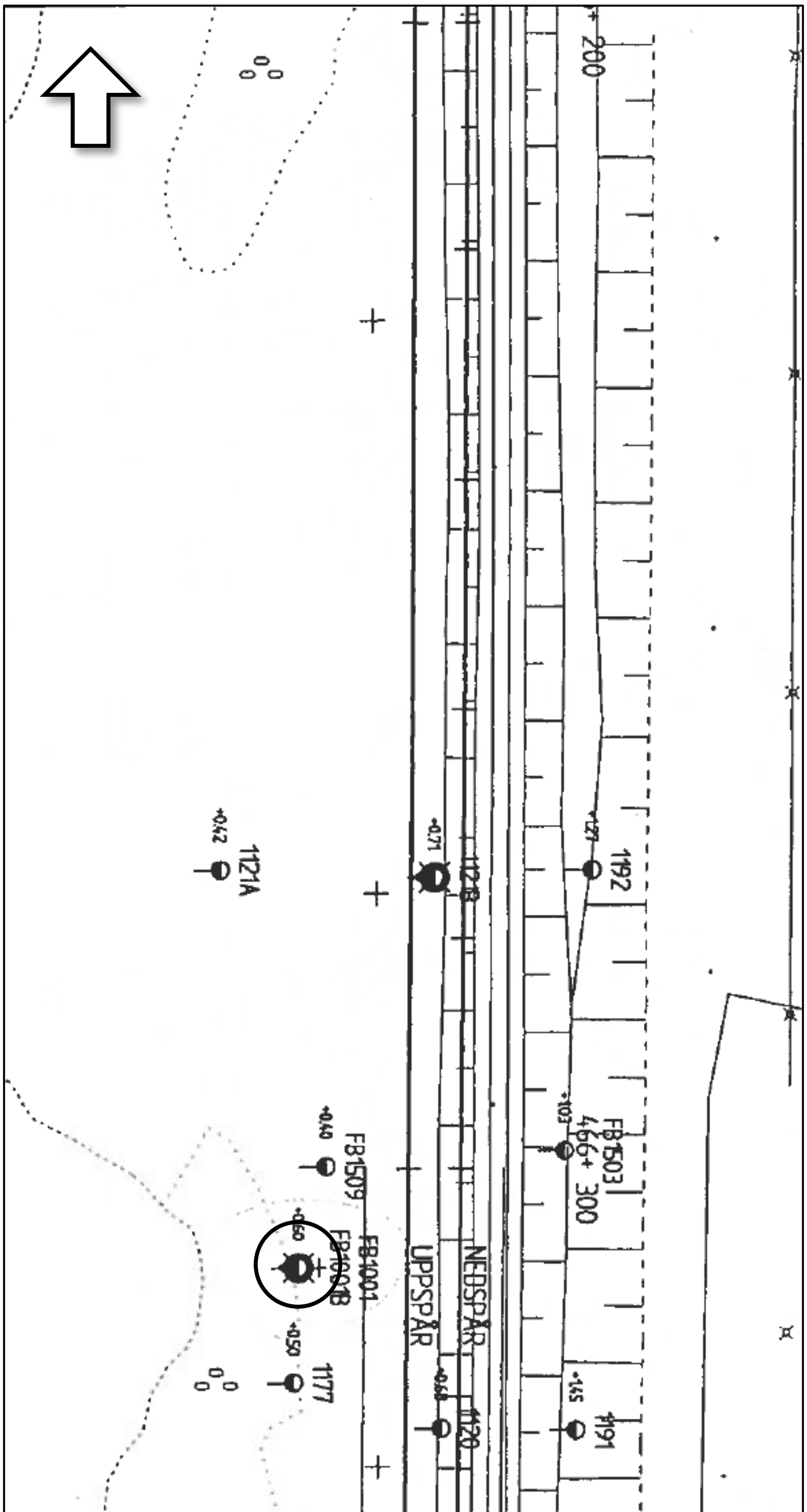

Corrected Shear Strength (kPa)
0
10
20
30
40
50
60
70
10
5
0
-5
-10
-15
-20
Level (m)
Direct shear trend
Direct shear trend ± 10%
Vane (corrected)
Cone (corrected)
Trend (Corrected cone)
Linjär (Vane (corrected))

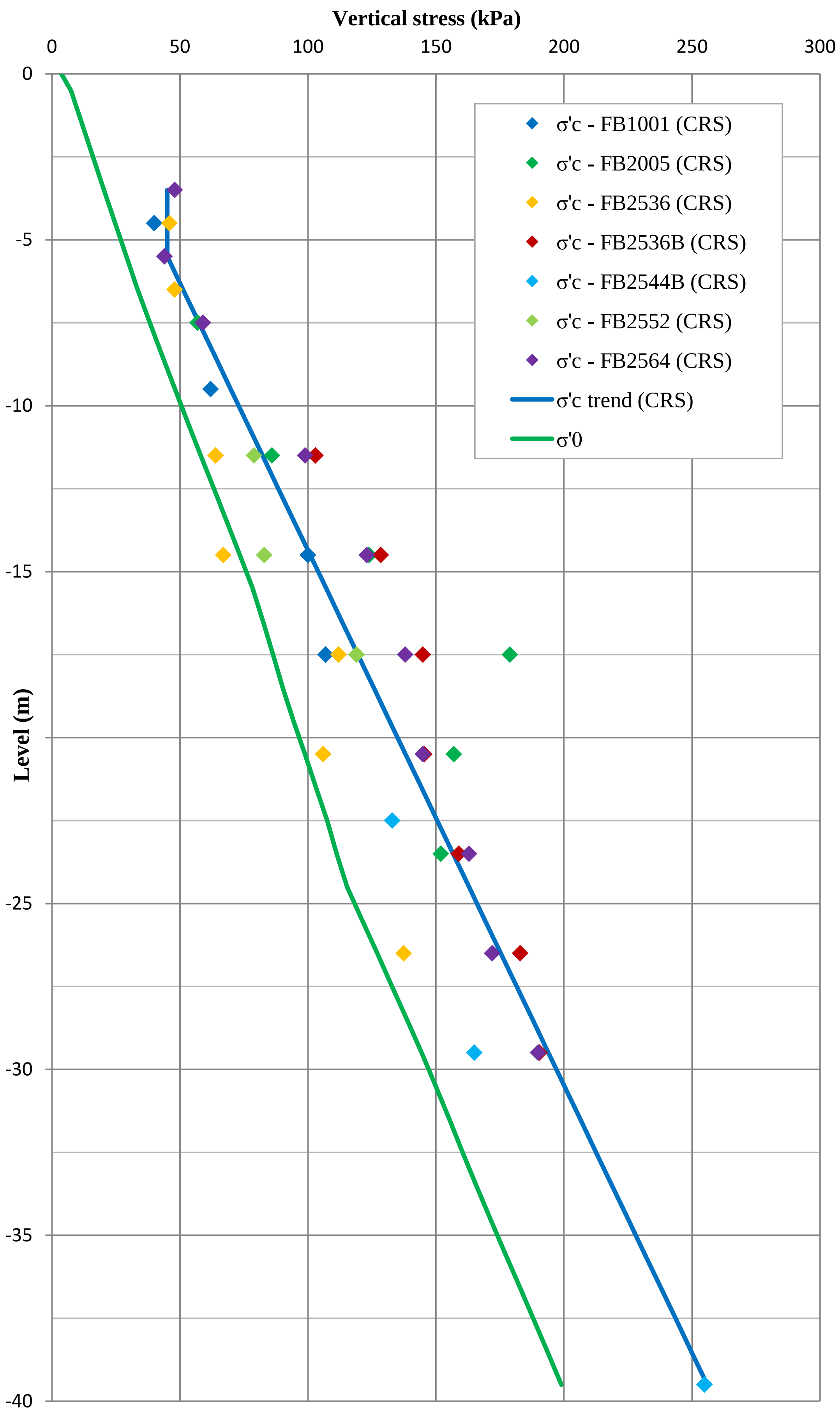

Uncorrected Shear Strength (kPa)
0
10
20
30
40
50
60
70
10
5
0
-5
-10
-15
-20
Level (m)
Direct shear trend
Direct shear trend ± 10%
Vane (uncorrected)
Cone (uncorrected)
Trend (Corrected cone)
Linjär (Vane (uncorrected))

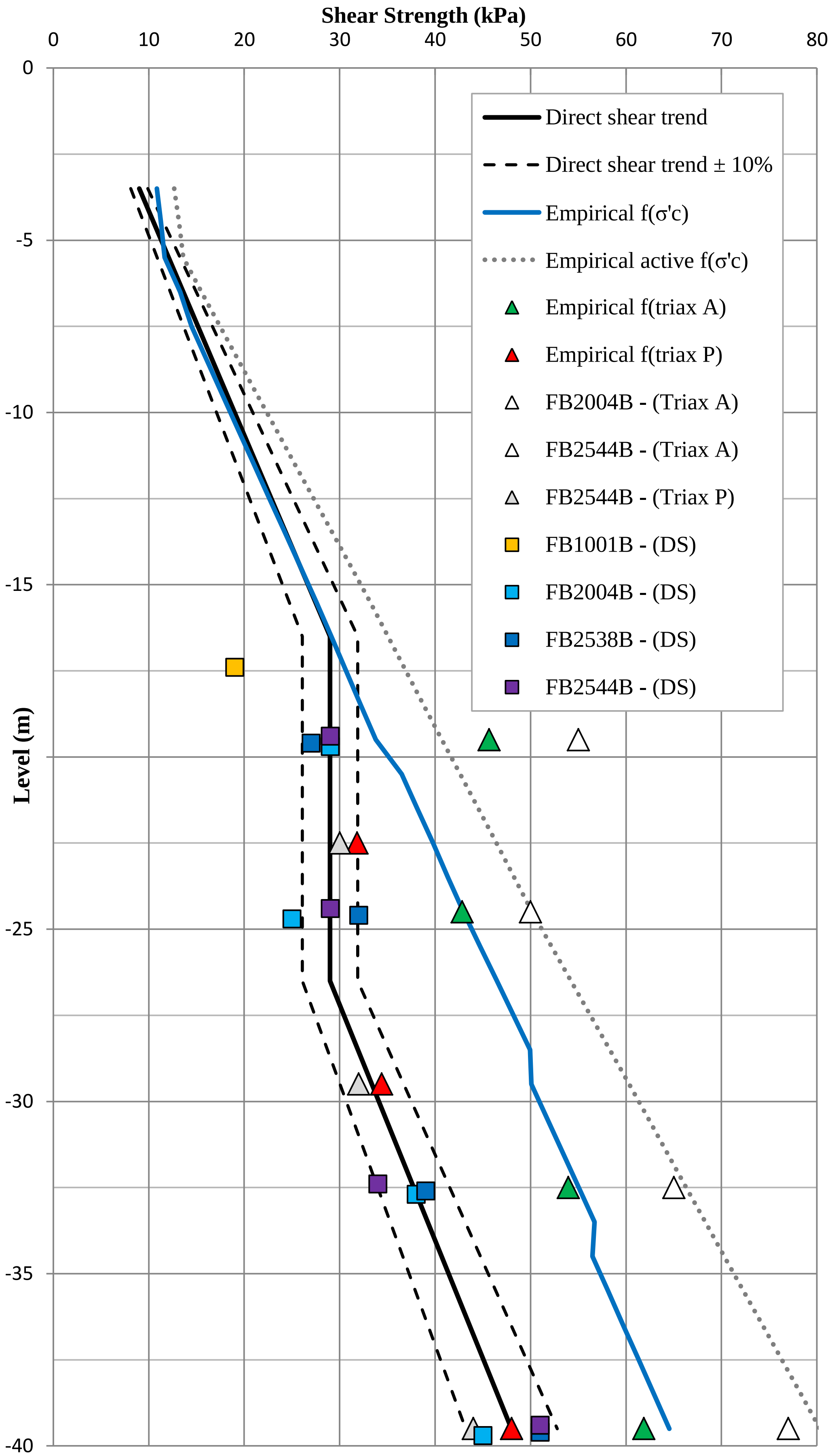
Borehole plan
465+720 – 466+120

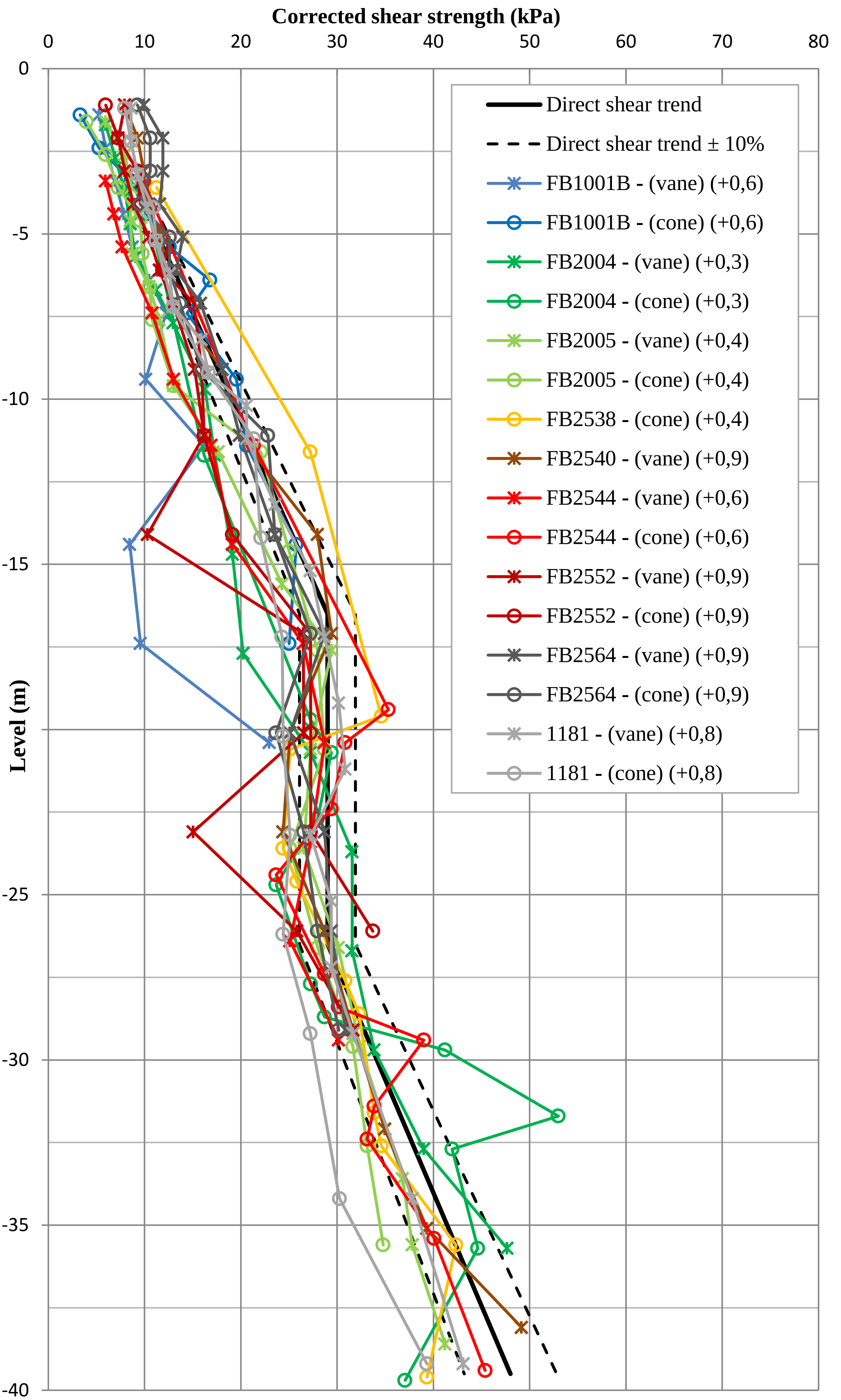


Borehole plan
466+200 – 466+350

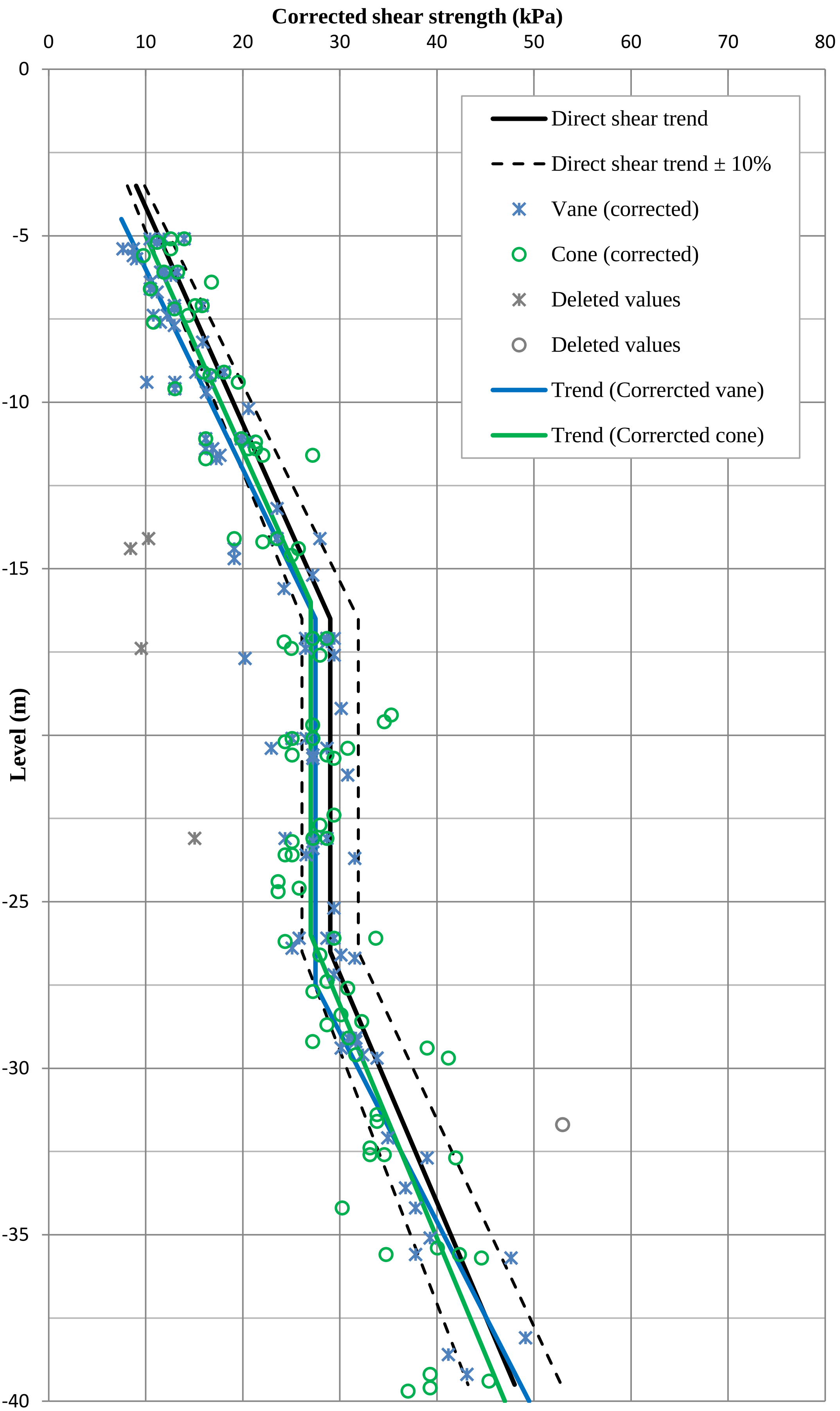


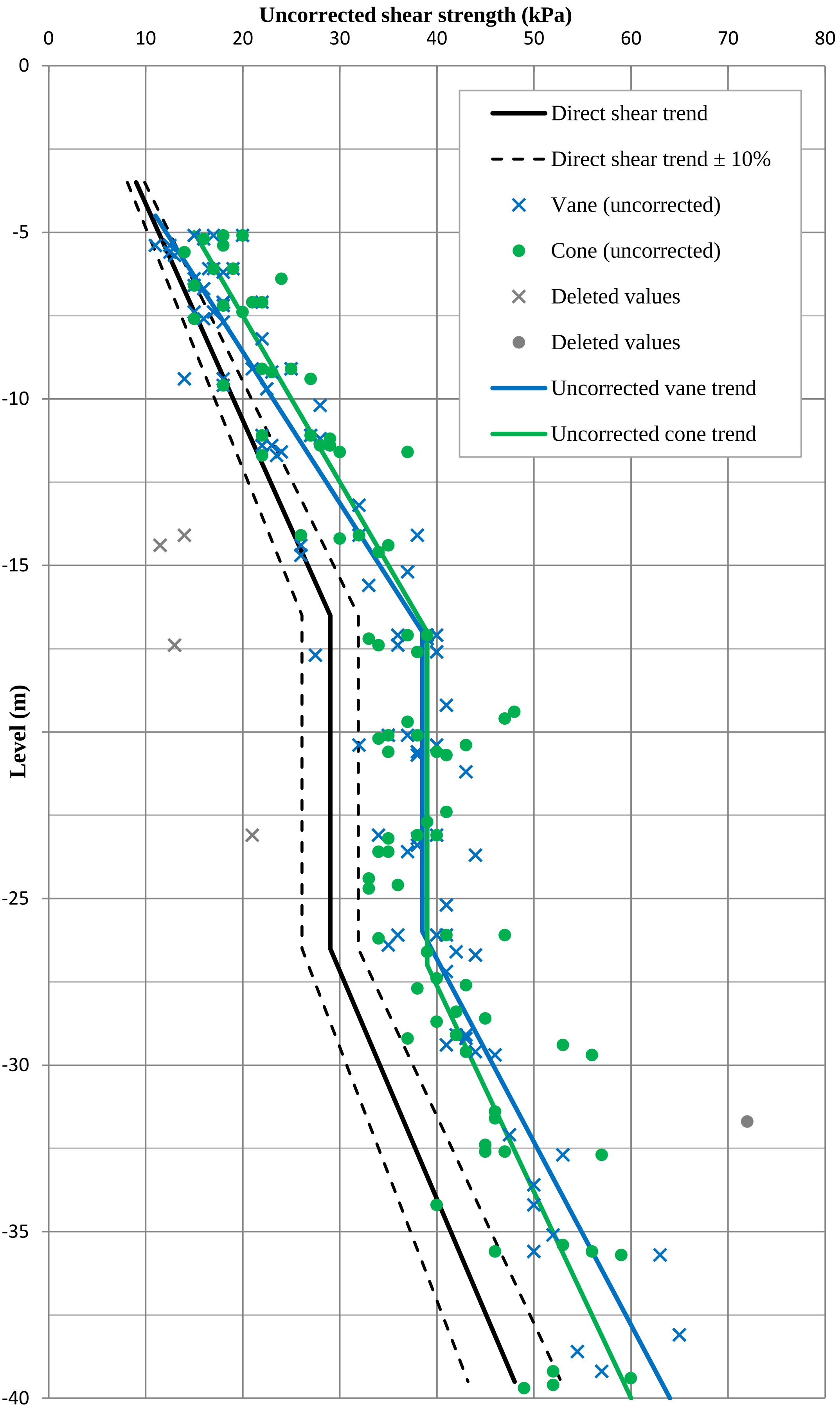

Vertical stress (kPa)
0
50
100
150
200
250
300
0
-5
-10
-15
-20
-25
-30
-35
-40
Level (m)
σ'c - FB1001 (CRS)
σ'c - FB2005 (CRS)
σ'c - FB2536 (CRS)
σ'c - FB2536B (CRS)
σ'c - FB2544B (CRS)
σ'c - FB2552 (CRS)
σ'c - FB2564 (CRS)
σ'c trend (CRS)
σ'0


Shear Strength (kPa)
0
10
20
30
40
50
60
70
80
0
-5
-10
-15
-25
-30
-35
-40
Level (m)
Direct shear trend
Direct shear trend ± 10%
Empirical f(σ'c)
Empirical active f(σ'c)
Empirical f(triax A)
Empirical f(triax P)
FB2004B - (Triax A)
FB2544B - (Triax A)
FB2544B - (Triax P)
FB1001B - (DS)
FB2004B - (DS)
FB2538B - (DS)
FB2544B - (DS)

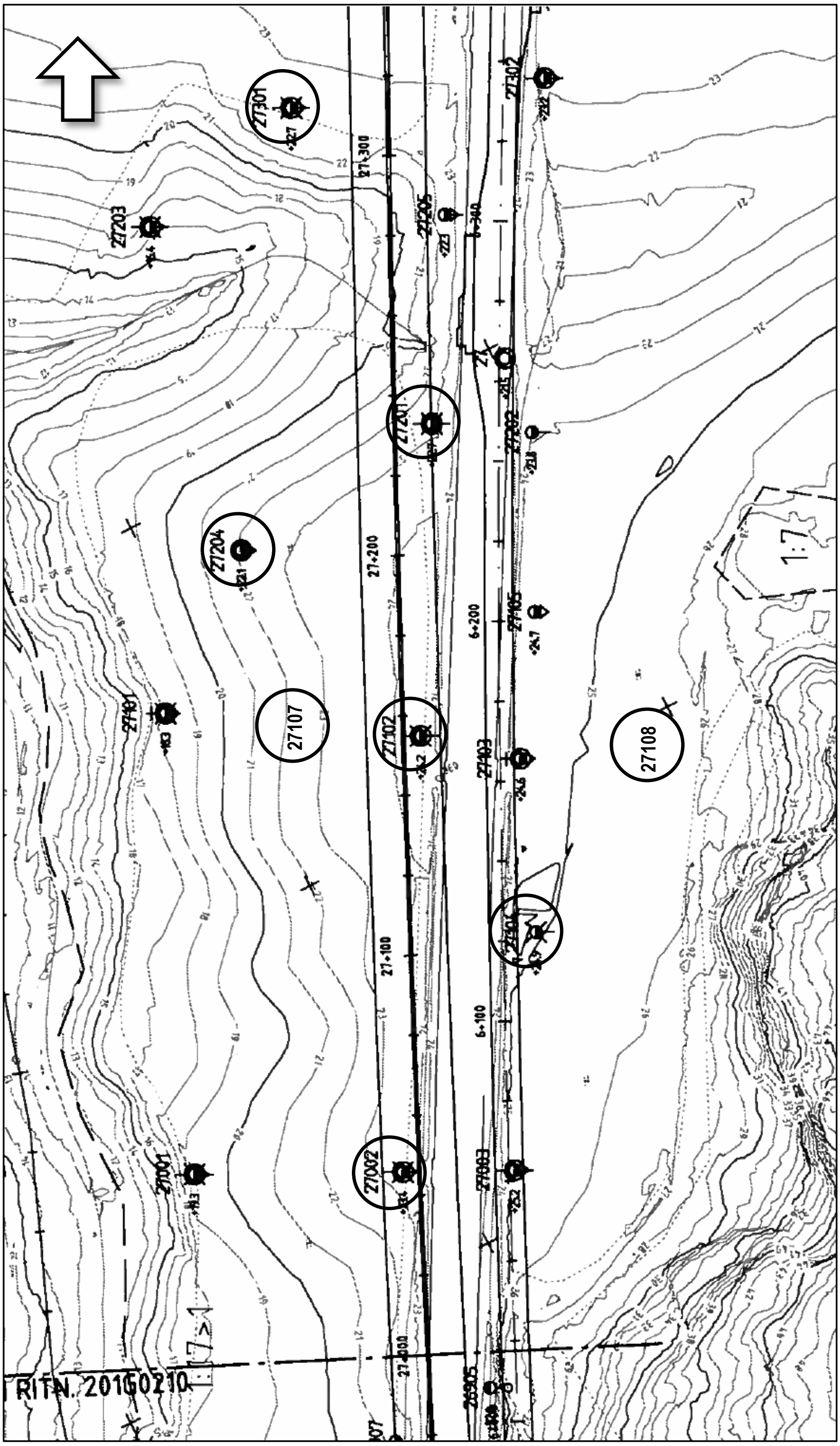


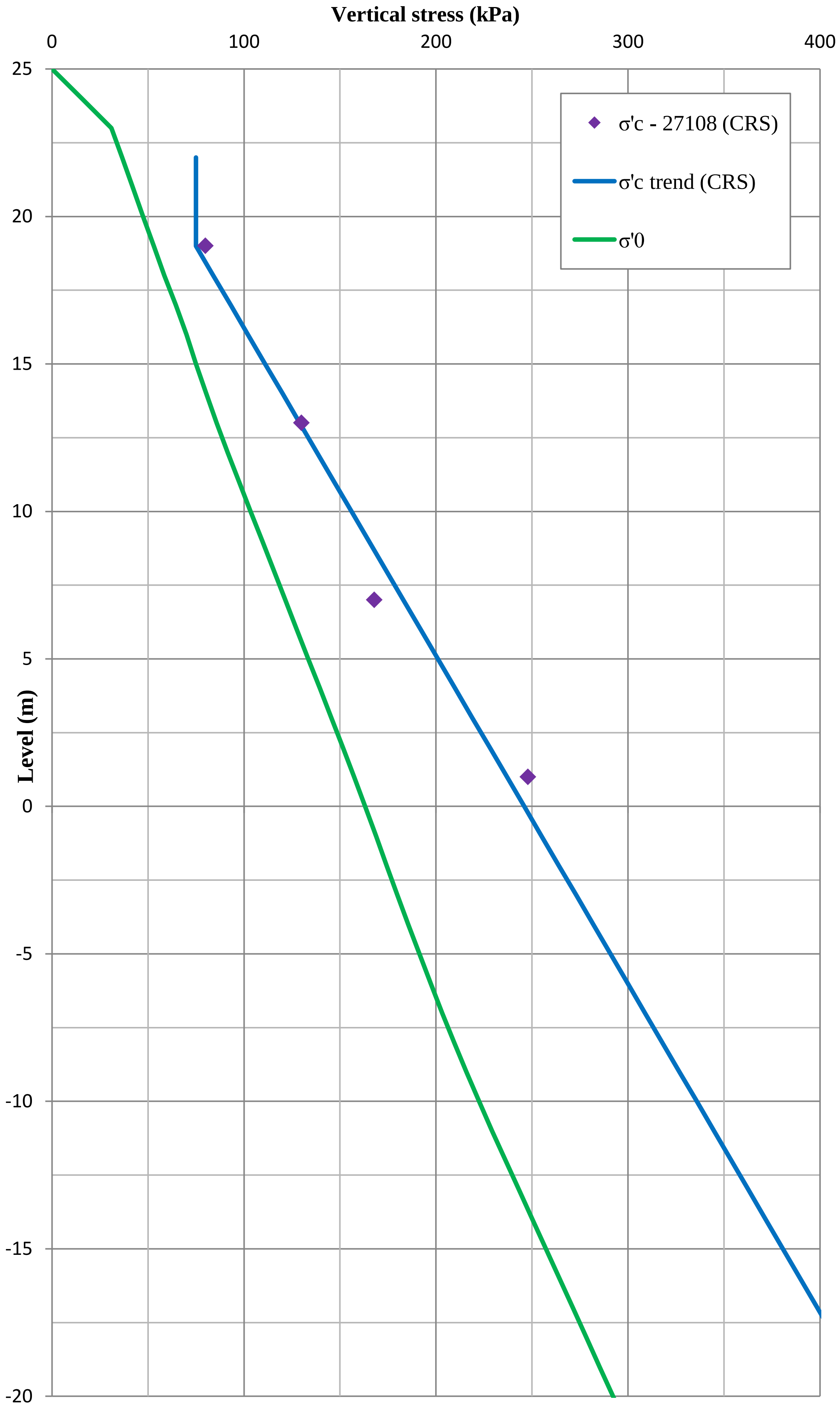
Corrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
Level (m)
0
-5
-10
-15
-20
-25
-30
-35
-40
Direct shear trend
Direct shear trend ± 10%
FB1001B - (vane) (+0,6)
FB1001B - (cone) (+0,6)
FB2004 - (vane) (+0,3)
FB2004 - (cone) (+0,3)
FB2005 - (vane) (+0,4)
FB2005 - (cone) (+0,4)
FB2538 - (cone) (+0,4)
FB2540 - (vane) (+0,9)
FB2544 - (vane) (+0,6)
FB2544 - (cone) (+0,6)
FB2552 - (vane) (+0,9)
FB2552 - (cone) (+0,9)
FB2564 - (vane) (+0,9)
FB2564 - (cone) (+0,9)
1181 - (vane) (+0,8)
1181 - (cone) (+0,8)


Corrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
0
-5
-10
-15
-20
-25
-30
-35
-40
Level (m)
Direct shear trend
Direct shear trend ± 10%
Vane (corrected)
Cone (corrected)
Deleted values
Deleted values
Trend (Corrercted vane)
Trend (Corrercted cone)

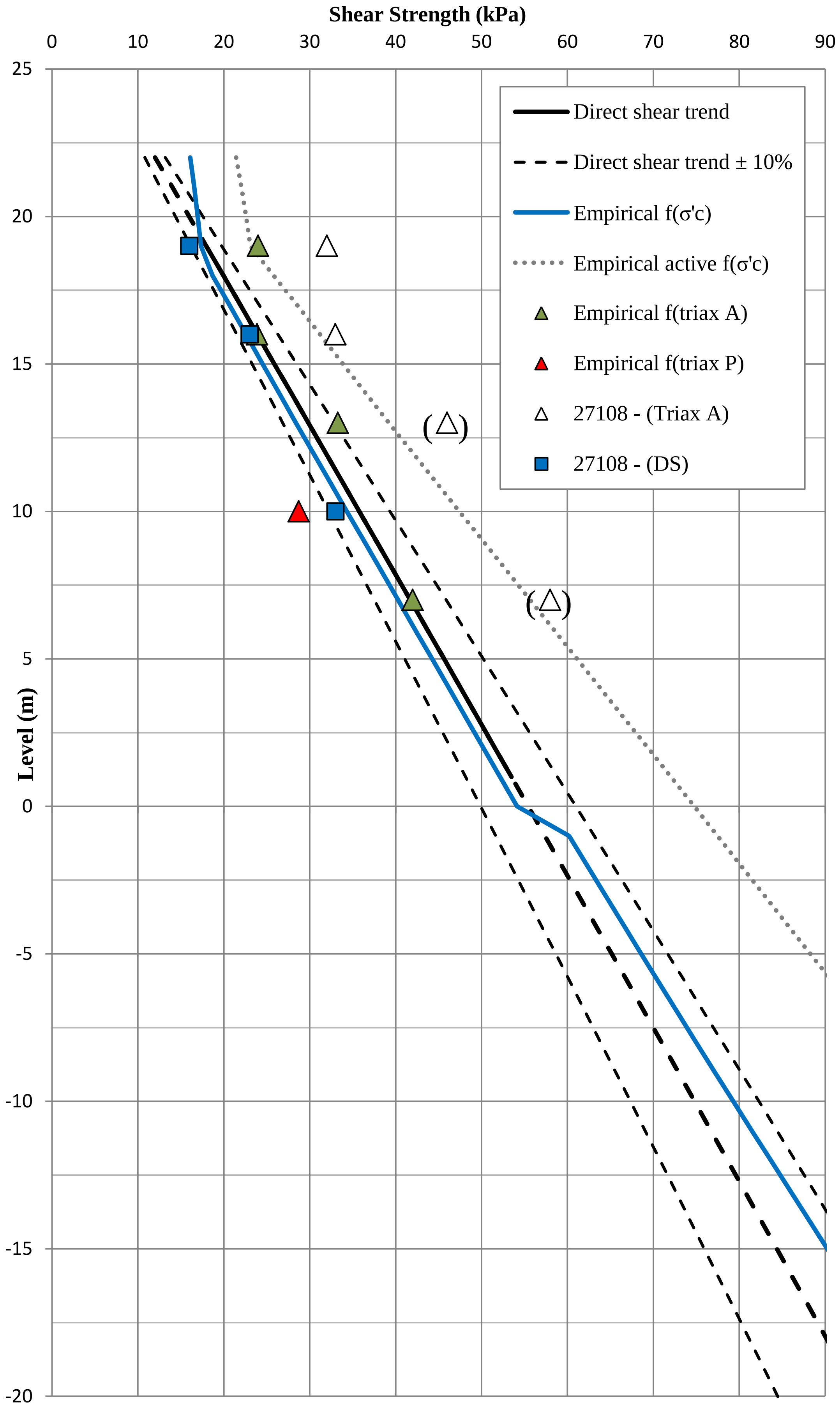

Uncorrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
0
-5
-10
-15
-20
-25
-30
-35
-40
Level (m)
Direct shear trend
Direct shear trend ± 10%
Vane (uncorrected)
Cone (uncorrected)
Deleted values
Deleted values
Uncorrected vane trend
Uncorrected cone trend

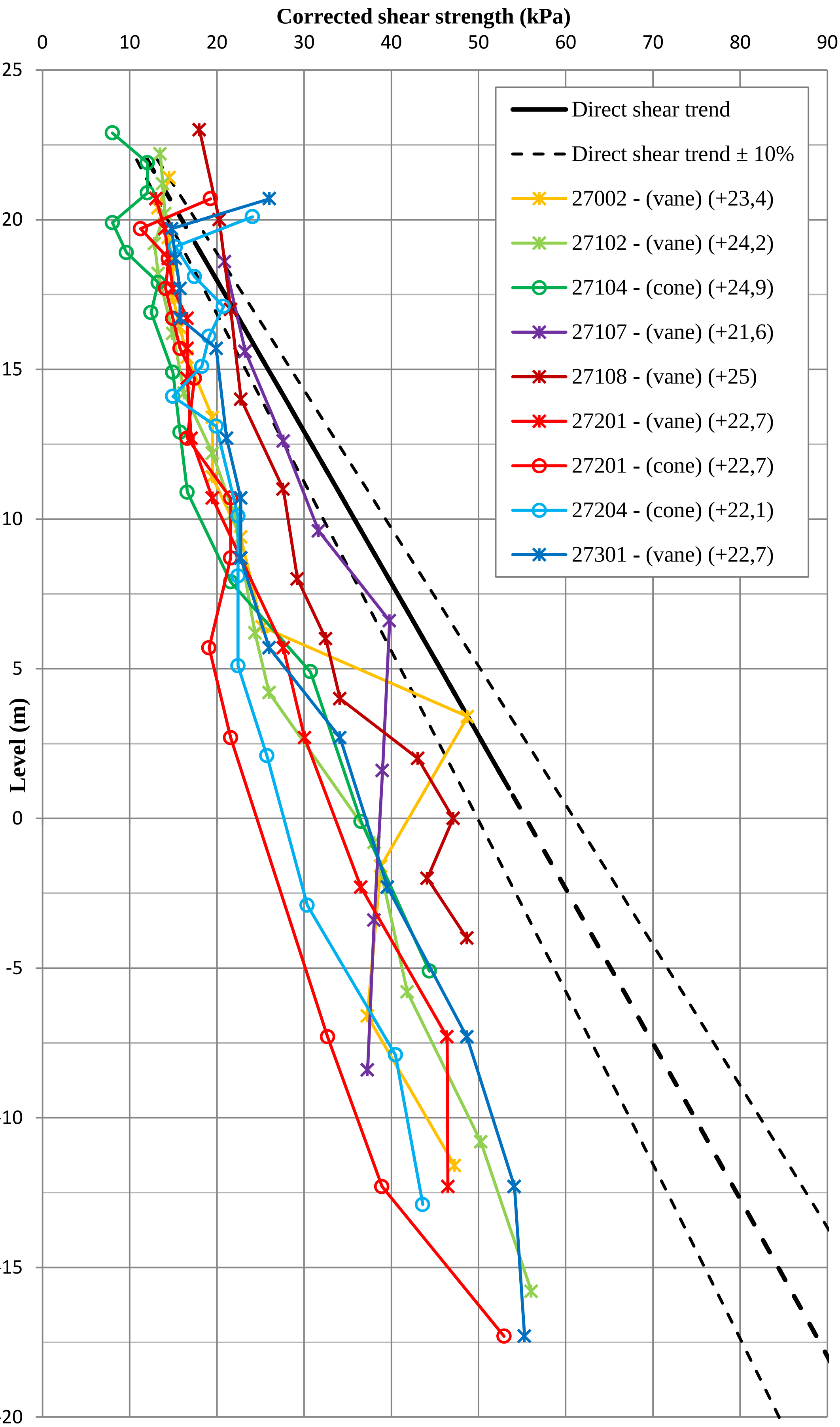
Borehole plan

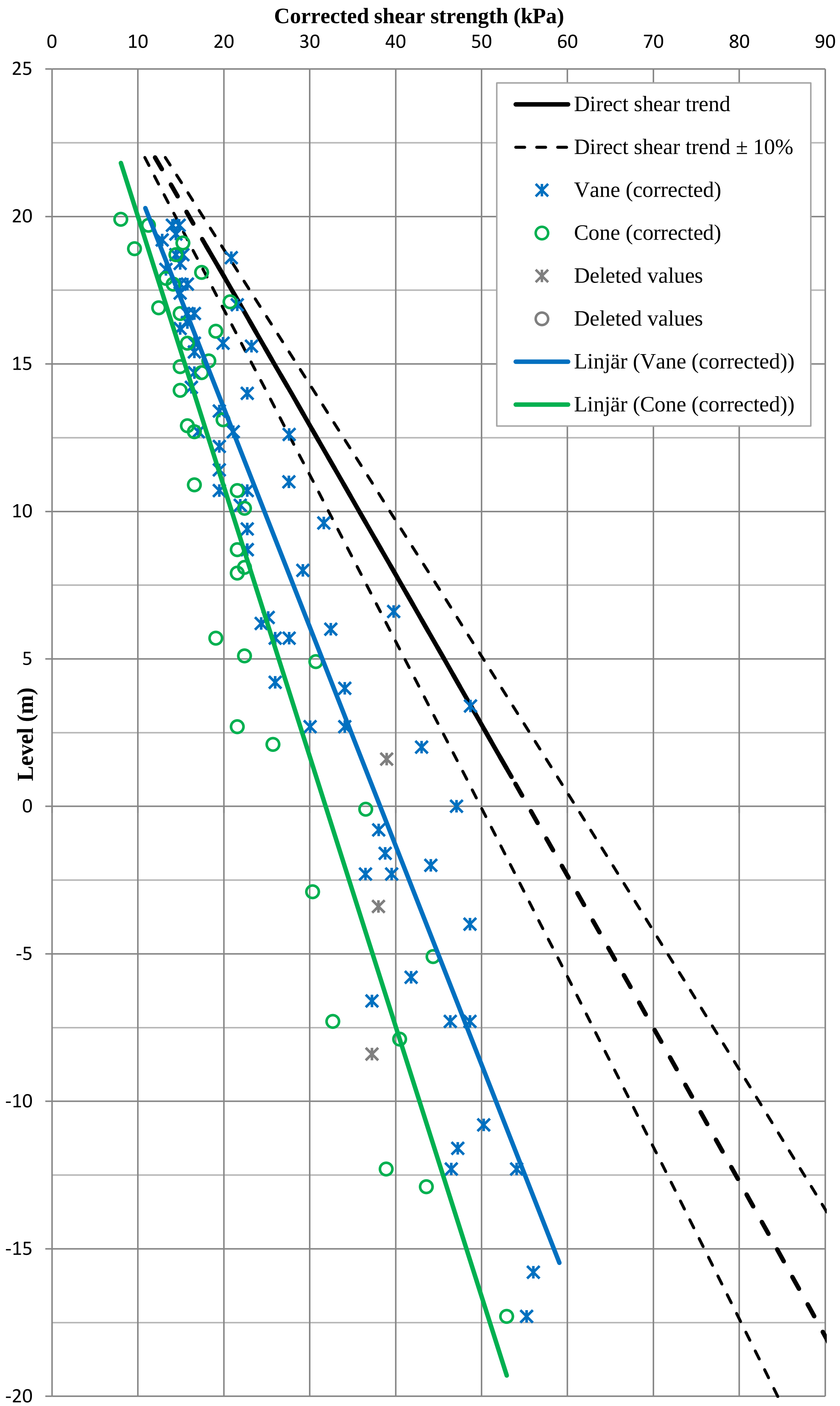




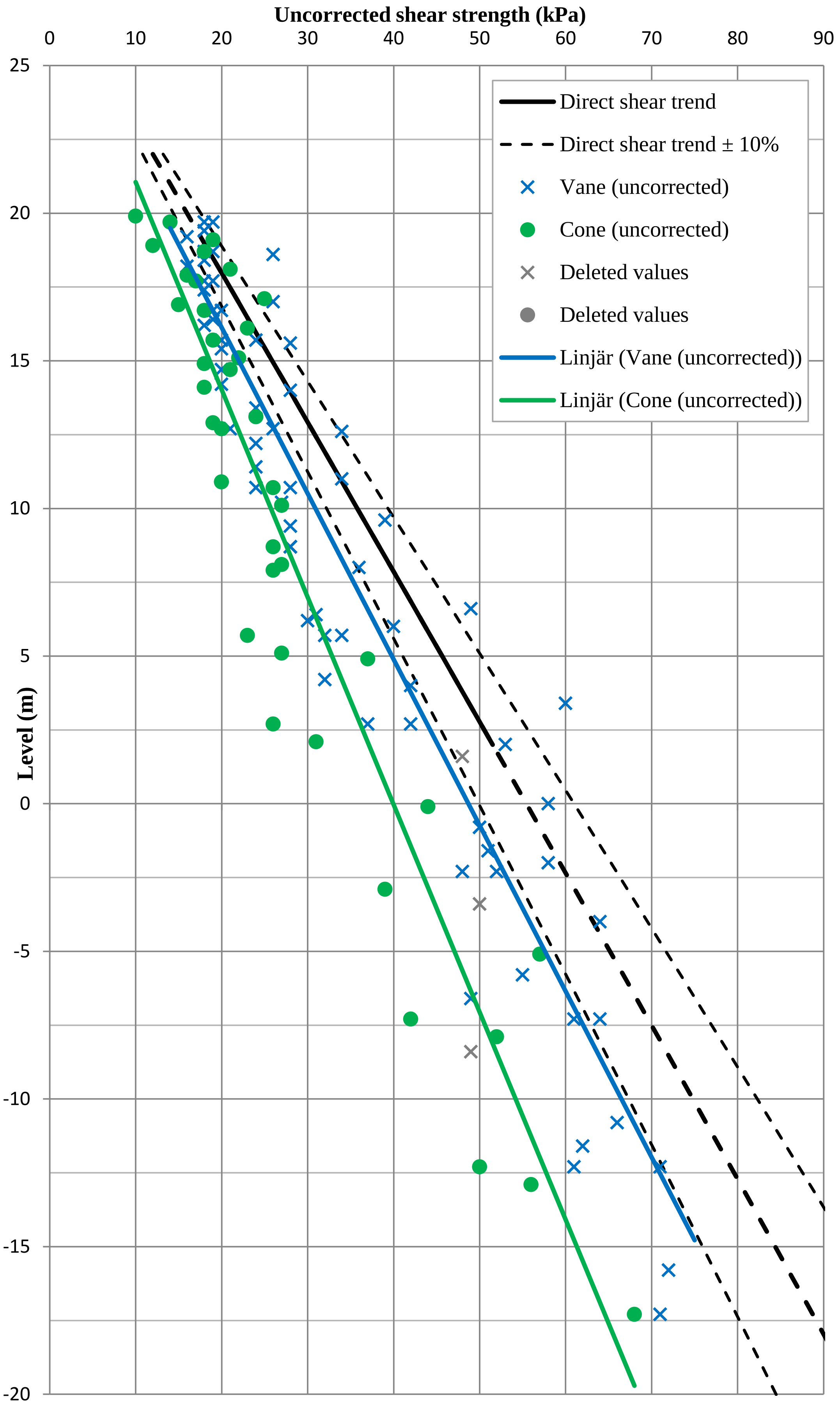
Vertical stress (kPa)
0
100
200
300
400
25
20
15
10
5
0
-5
-10
-15
-20
Level (m)
σ'c - 27108 (CRS)
σ'c trend (CRS)
σ'0


Shear Strength (kPa)
0
10
20
30
40
50
60
70
80
90
25
20
15
10
5
0
-5
-10
-15
-20
Level (m)
Direct shear trend
Direct shear trend ± 10%
Empirical f(σ'c)
Empirical active f(σ'c)
Empirical f(triax A)
Empirical f(triax P)
27108 - (Triax A)
27108 - (DS)
(△)
(△)

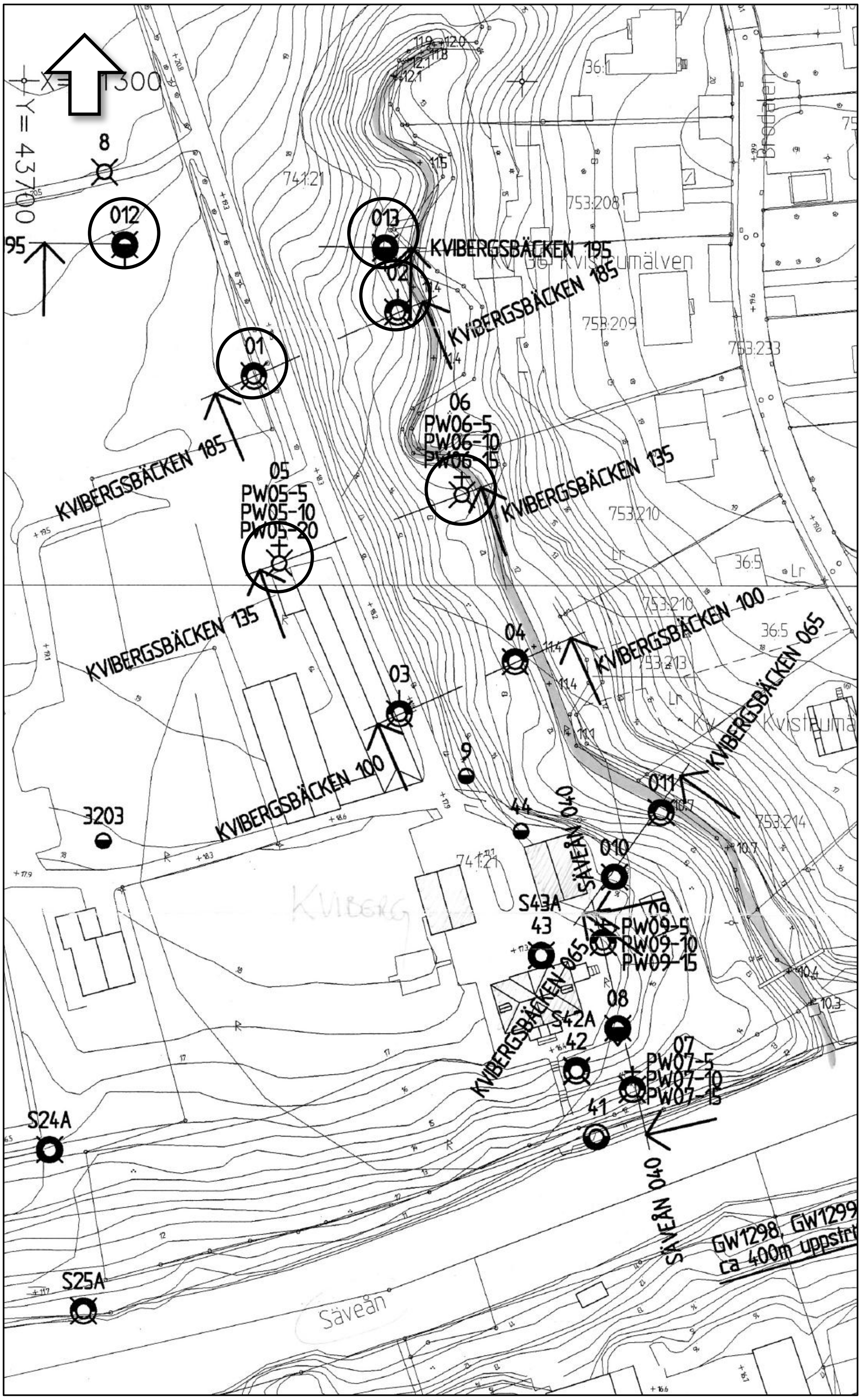

Corrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
90
25
20
15
10
5
0
-5
-10
-15
-20
Level (m)
Direct shear trend
Direct shear trend ± 10%
27002 - (vane) (+23,4)
27102 - (vane) (+24,2)
27104 - (cone) (+24,9)
27107 - (vane) (+21,6)
27108 - (vane) (+25)
27201 - (vane) (+22,7)
27201 - (cone) (+22,7)
27204 - (cone) (+22,1)
27301 - (vane) (+22,7)

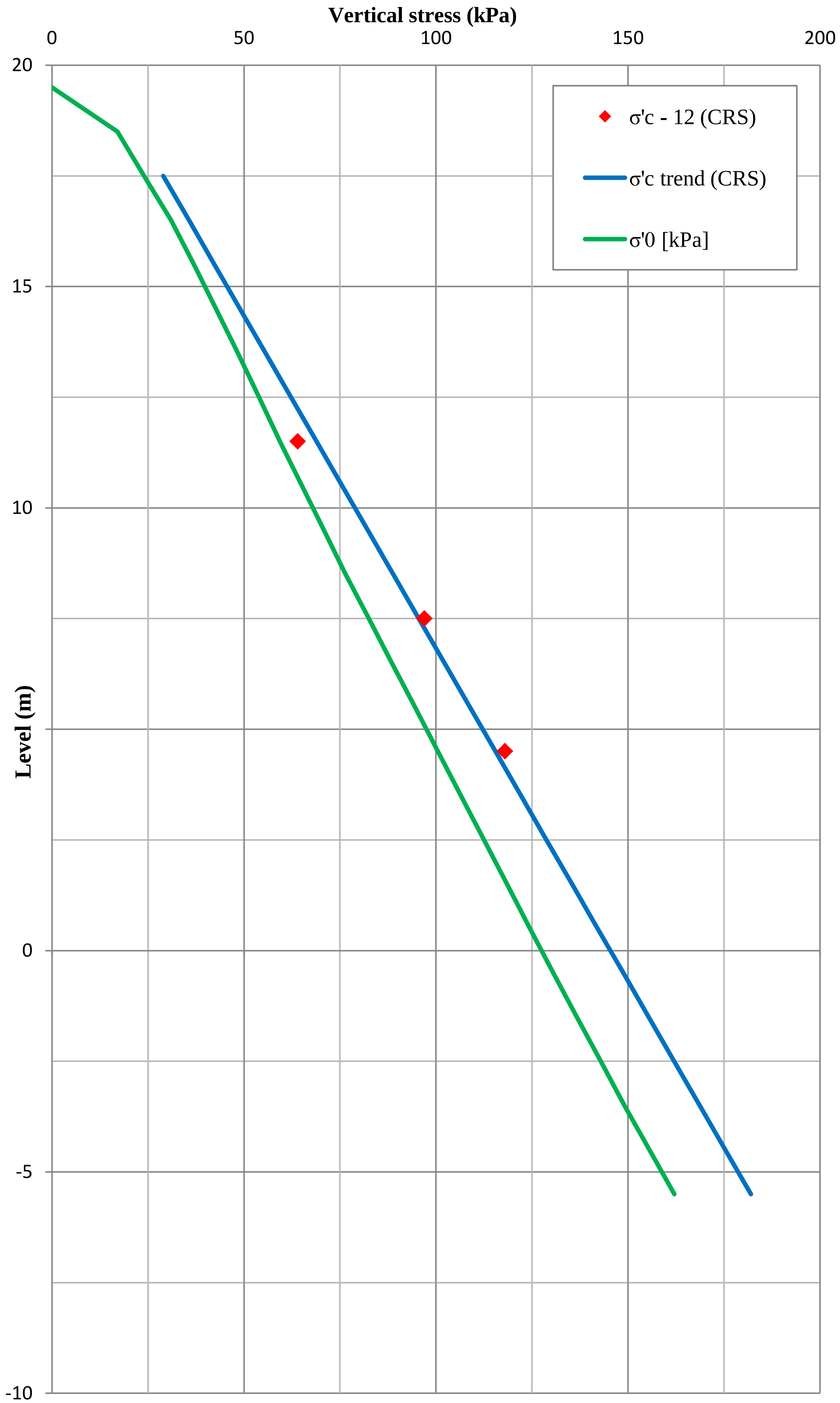


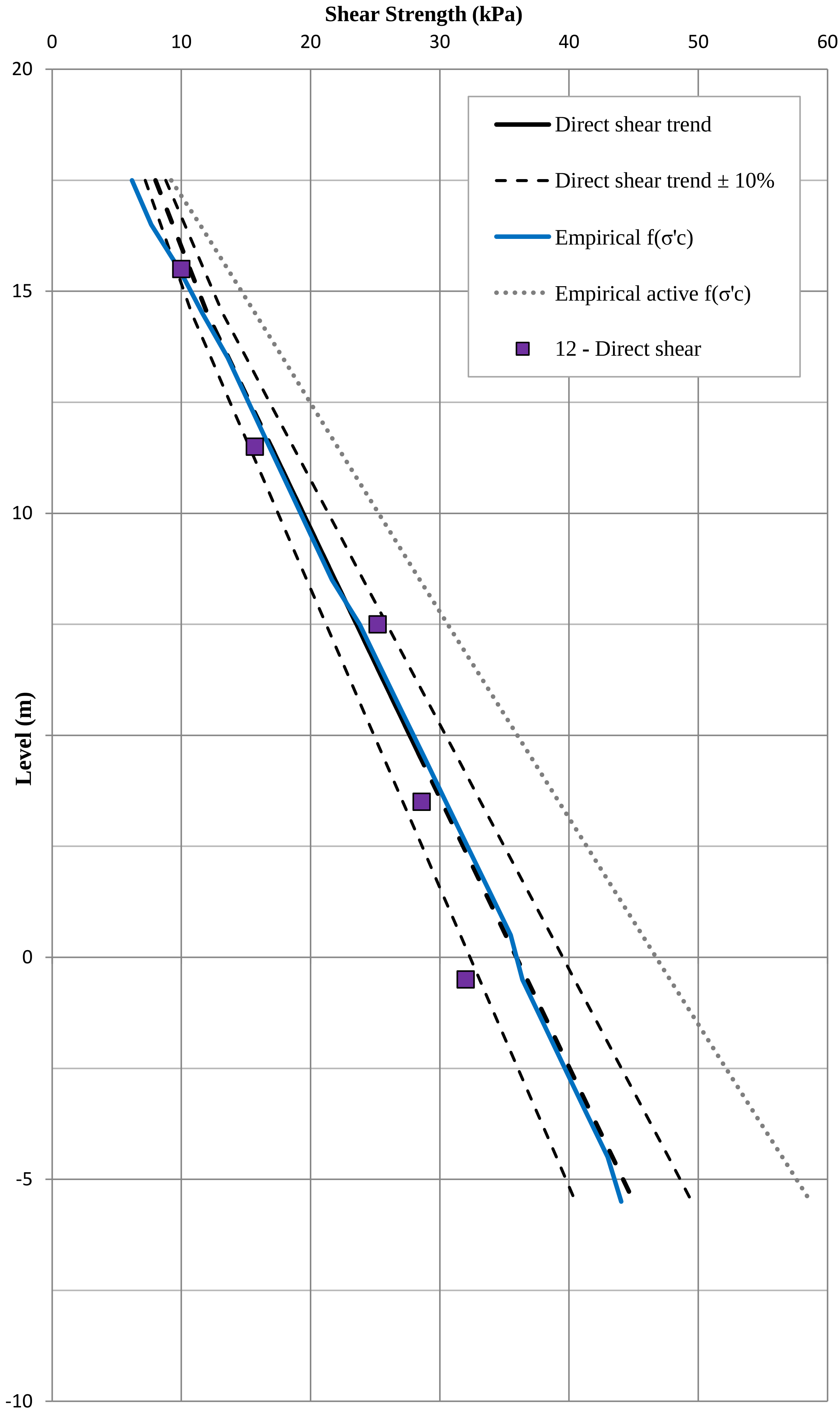
Corrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
90
Level (m)
25
20
15
10
5
0
-5
-10
-15
-20
Direct shear trend
Direct shear trend ± 10%
Vane (corrected)
Cone (corrected)
Deleted values
Deleted values
Linjär (Vane (corrected))
Linjär (Cone (corrected))

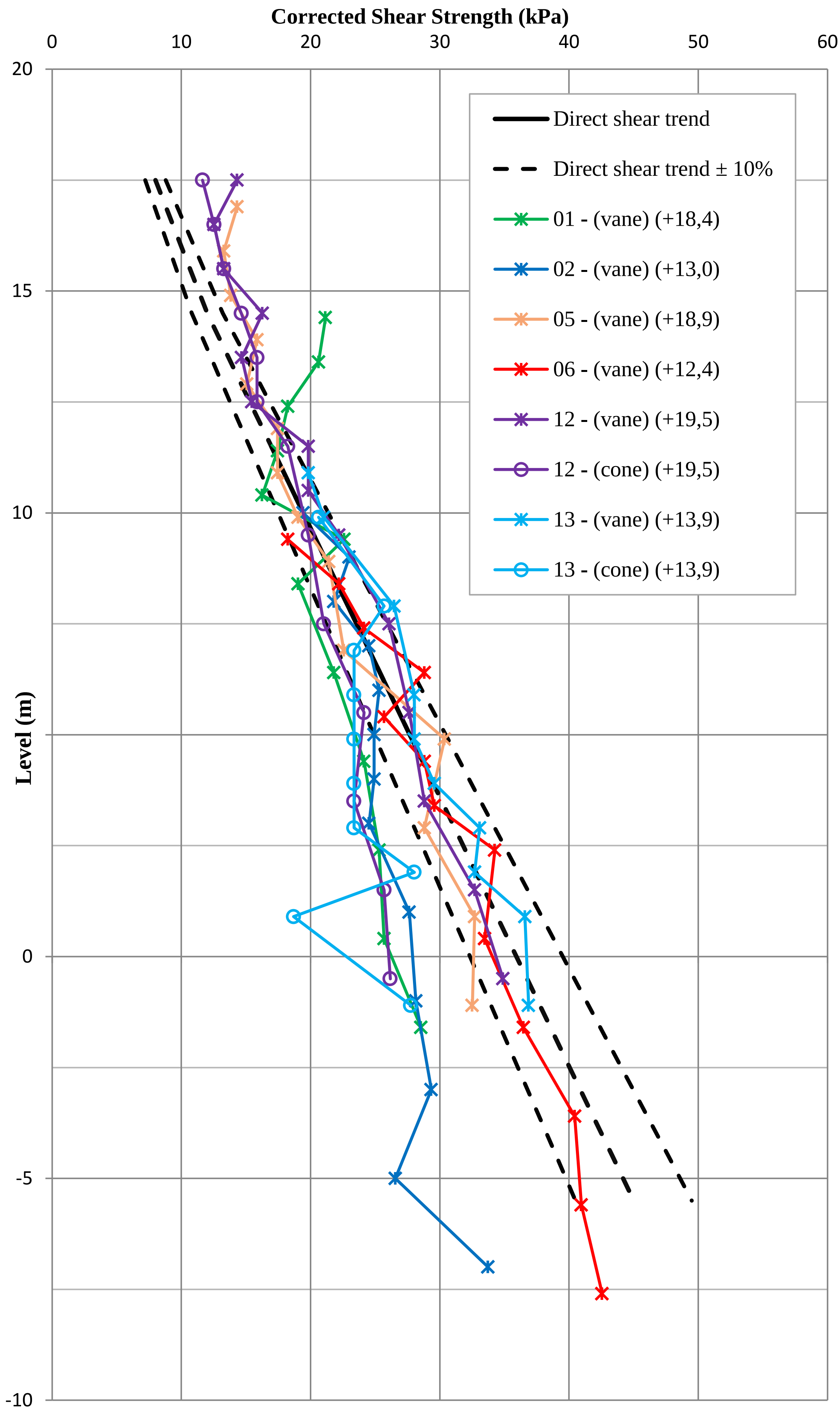

Uncorrected shear strength (kPa)
0
10
20
30
40
50
60
70
80
90
Level (m)
25
20
15
10
5
0
-5
-10
-15
-20
Direct shear trend
Direct shear trend ± 10%
Vane (uncorrected)
Cone (uncorrected)
Deleted values
Deleted values
Linjär (Vane (uncorrected))
Linjär (Cone (uncorrected))

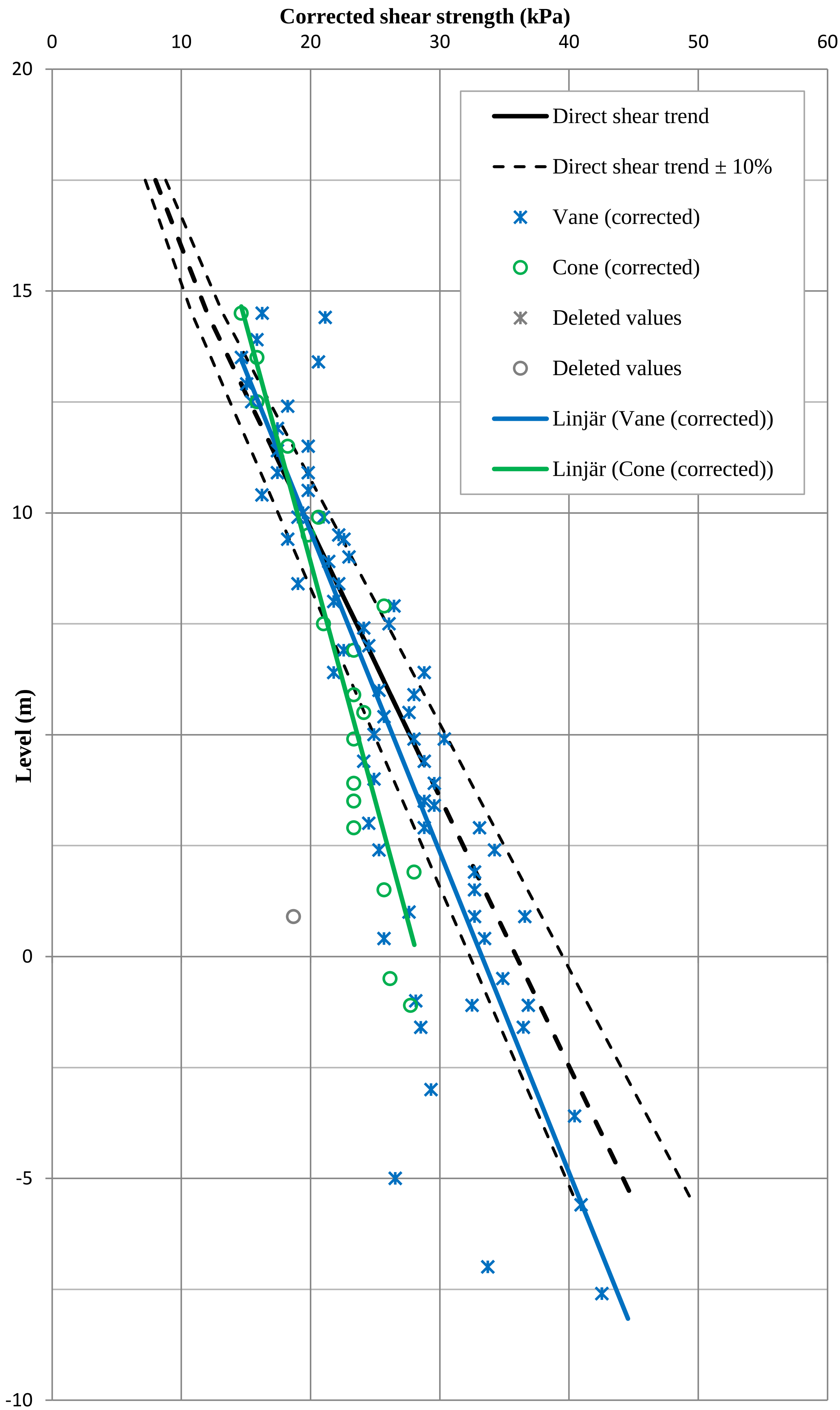
Borehole plan

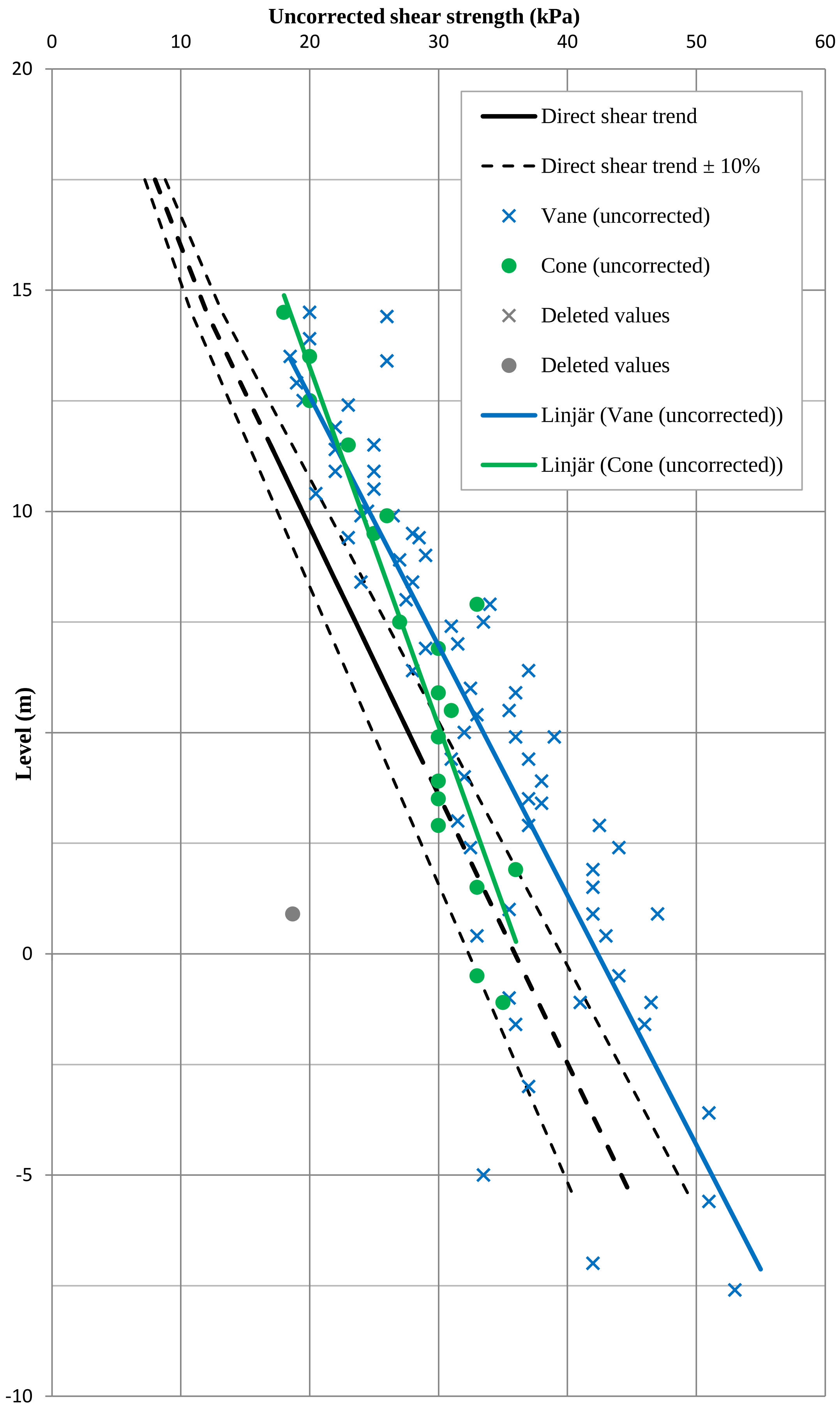



Vertical stress (kPa)
0
50
100
150
200
20
15
10
0
-5
-10
Level (m)
σ'c - 12 (CRS)
σ'c trend (CRS)
σ'0 [kPa]


Shear Strength (kPa)
0
10
20
30
40
50
60
20
15
10
0
-5
-10
Level (m)
Direct shear trend
Direct shear trend ± 10%
Empirical f(σ'c)
Empirical active f(σ'c)
12 - Direct shear


Corrected Shear Strength (kPa)
0
10
20
30
40
50
60
20
15
10
5
0
-5
-10
Level (m)
Direct shear trend
Direct shear trend ± 10%
01 - (vane) (+18,4)
02 - (vane) (+13,0)
05 - (vane) (+18,9)
06 - (vane) (+12,4)
12 - (vane) (+19,5)
12 - (cone) (+19,5)
13 - (vane) (+13,9)
13 - (cone) (+13,9)


Corrected shear strength (kPa)
0
10
20
30
40
50
60
20
15
10
5
0
-5
-10
Level (m)
Direct shear trend
Direct shear trend ± 10%
Vane (corrected)
Cone (corrected)
Deleted values
Deleted values
Linjär (Vane (corrected))
Linjär (Cone (corrected))


Uncorrected shear strength (kPa)
0
10
20
30
40
50
60
20
15
10
5
0
-5
-10
Level (m)
Direct shear trend
Direct shear trend ± 10%
Vane (uncorrected)
Cone (uncorrected)
Deleted values
Deleted values
Linjär (Vane (uncorrected))
Linjär (Cone (uncorrected))