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Evaluation of shear strength in cohesive soils with special reference to Swedish practice and experience

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EVALUATION OF SHEAR STRENGTH IN COHESIVE SOILS WITH SPECIAL REFERENCE TO SWEDISH PRACTICE AND EXPERIENCE

In this publication, information is given on how different test results and empirical experience can be weighted and combined to give the best possible estimation of the shear strength properties in cohesive soils.

The procedure described has evolved during the continuing work at SGI as research results and practical experience from the Institute as well as others have been obtained and taken into consideration.

The publication describes how the Institute currently proceeds in the evaluation of shear strength in cohesive soils. The intention has also been to bring about a more unified and objective procedure than that has been used up to now.

Valuable views on this work have been given by colleagues in and outside the Institute.

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Rolf Larsson Ulf Bergdahl Leif Eriksson

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INTRODUCTION

In Sweden, calculation of stability in cohesive soil is mainly based on shear strength values determined by vane shear tests or fall cone tests. These values of shear strength have to be corrected before they are used in calculations. An engineering judgement must also be made as to the relevancy of these corrected strengths to the particular case.

An empirical knowledge of how the shear strength varies with plasticity, preconsolidation pressure and overconsolidation ratio has evolved from results from other types of tests such as triaxial tests and direct shear tests.

A new sounding method, the combined cone penetration test and pore pressure sounding (CPTV-test), has also come into use. This method gives a good picture of stratigraphy and variation in shear strength, although it cannot yet be considered to give good enough numerical values of the shear strength.

In this publication, a recommendation is given for how the shear strength values obtained by vane shear tests and fall cone tests should be corrected. Guidance is also given for how other test results, empirical knowledge and results from soundings can be incorporated in the engineering evaluation of "real" shear strength.

The recommendation for correction of shear strength values obtained by vane shear tests and fall cone tests given here does not involve any larger differences in the end result compared to the old SGI recommendation from 1969. The new procedure, however, is more precise and objective.

The recommendation and the guidelines are mainly based on experience of Scandinavian soils and cannot be applied directly to soils from other geological regions.

Part of the historical development and research that has led to the present recommendation and the empirical relations that are used is presented in an appendix.

DEFINITIONS AND SYMBOLS

c'	shear strength parameter
CRS-test	oedometer test with constant rate of strain
F_B	calculated factor of safety at failure
k	constant
q_c	point resistance in cone penetration tests
t	time to failure
t_1	reference time
w_L	liquid limit (w_L in equations is given in decimal numbers)
β	coefficient
$\Delta \sigma'_c$	increase in preconsolidation pressure
$\Delta\tau_{fu}$	increase in undrained shear strength
ϕ'	shear strength parameter
μ	correction factor for shear strength values obtained by vane shear tests and fall cone tests
σ_o	existing overburden pressure
σ'	effective normal stress
σ'_c	preconsolidation pressure
σ'_o	existing effective overburden pressure
τ	shear stress
τ_v	uncorrected value of shear strength obtained by vane shear test
τ_k	uncorrected value of shear strength obtained by fall cone test
τ_{fu}	undrained shear strength
τ_{fd}	drained shear strength
τ_{ACTIVE}	undrained shear strength at "active shear"
$\tau_{DIRECT\ SHEAR}$	undrained shear strength at "direct shear"
$\tau_{PASSIVE}$	undrained shear strength at "passive shear"
τ_{CR}	mobilized shear stress in passive triaxial tests at a deformation corresponding to the deformation at failure in active triaxial tests
$\tau_{AVERAGE}$	average undrained shear strength $(\tau_{ACTIVE} + \tau_{DIRECT\ SHEAR} + \tau_{PASSIVE})/3$
τ_{LAB}	$\tau_{AVERAGE}$ from laboratory tests
τ_o	value of shear strength obtained by vane shear test with a waiting time of 5 minutes between installation and start of the test
τ_{1D}	value of shear strength obtained by vane shear test with a waiting time of 24 hours between installation and start of the test
τ_1	value of shear strength obtained by vane shear test with standard rate of rotation
τ_t	value of shear strength obtained by vane shear test with a rate of rotation giving the time to failure

UNDRAINED SHEAR STRENGTH IN NORMALLY CONSOLIDATED AND SLIGHTLY OVERCONSOLIDATED SOILS

Correction of values of undrained shear strength obtained by vane shear tests and fall cone tests

The correction that was previously recommended by SGI dated 1969 and consequently did not include the research results obtained during the last 15 years. It implied a pretreatment of the measured values which was subjective and also differed from what is customary in other countries.

New building codes, new methods for statistical treatment of measured data and also the need for compilation and comparison of experiences demand a uniform treatment of the measured data. This also calls for an according adjustment of SGI's correction factors. It is therefore now recommended that shear strength values obtained by vane shear tests and fall cone tests be corrected with respect to the liquid limit of the soil in accordance with

$$\tau_{fu} = \mu \cdot \tau_v$$
$$\tau_{fu} = \mu \cdot \tau_k$$

where τ_{fu} = undrained shear strength
μ = correction factor
τ_v = shear strength value obtained by vane shear test
τ_k = shear strength value obtained by fall cone test

The correction factor μ is a function of the liquid limit w_L and can be calculated from

$$\mu = \left(\frac{0{,}43}{w_L}\right)^{0{,}45} \geq 0{,}5$$

or taken from Fig. 1.

w_L in the equation is expressed as a decimal number.

Correction factors higher than 1.2 ought not to be used without supporting evidence from complementary investigations.

Each shear strength value ought to be corrected using a correction factor calculated from the appropriate liquid limit. For correction of a shear strength value obtained by vane shear test a determination of the liquid limit at the same point is thus normally required.

The corrected shear strengths are then plotted as a shear strength profile versus depth. In slopes created by erosion, the strengths are normally plotted versus elevation. An engineering evaluation of the corrected strengths considering all other known properties of the soil is then made. After even-

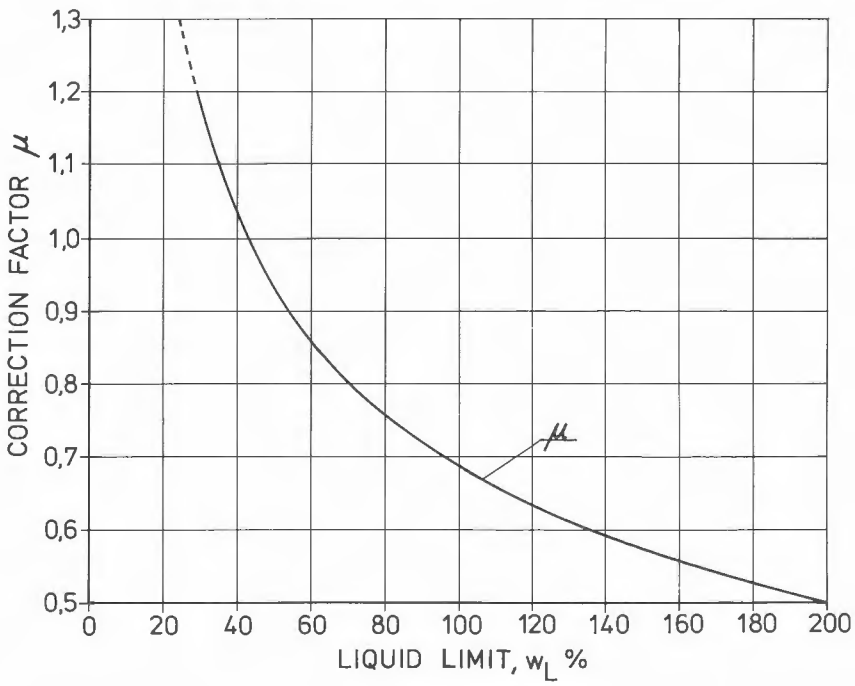


Fig 1. Recommended correction factor for shear strength values obtained by vane shear tests or fall cone test.

tual exclusion, supplementation or modification of values considered erroneous on the basis of the following guidelines, averages of the corrected shear strengths are then used in the calculations.

The correction factors are based on arithmetic averages of measured shear strength values and consequently arithmetic averages of corrected shear strengths must be used when more than one shear strength value from the same level is taken into consideration.

In cases where only a few determinations of shear strength have been made, this deficiency should be accounted for by the required calculated factor of safety.

Estimation of shear strength values on the basis of Hansbo's relation

The correction factors are mainly derived from comparisons between averages of measured shear strength values and mobilized shear stress at failure in full scale failures in the field. Comparisons of averages of shear strength values obtained by vane shear tests and fall cone tests in relation to averages of shear strength values from direct shear tests and active and passive triaxial tests have been used as complements. These comparisons have naturally resulted in a certain spread and the correction factors represent averages.

A condition for these correction factors to give shear strengths of use is that the vane shear test and the fall cone test give results which are normal for the type of soil in question. The typical result in normally consolidated and slightly overconsolidated Scandinavian soils is that the shear strength values obtained by vane shear tests and fall cone tests mainly follow Hansbo's relation.

$$\tau_{v,k} = \sigma'_c \cdot 0.45\, w_L \quad \text{(Hansbo 1957)}$$

where

τ_v = uncorrected shear strength value from vane shear tests

τ_k = uncorrected shear strength value from fall cone test

σ'_c = preconsolidation pressure

w_L = liquid limit

A compilation of a number of shear strength values obtained by vane shear tests in Scandinavian soils is shown in relation to preconsolidation pressure and liquid limit in Fig. 2.

As can be observed in the figure, the spread is considerable. In the figure a number of cases where the shear strength values obtained by the vane shear tests have been unusually high or low have been specially marked. In these cases the recommended correction factors have also proved to be too high or too low respectively.

As a simple estimation in this way can be made of the reasonableness of the shear strength values a comparison with Hansbo's relation ought to be made. If the shear strength value is unusually high there is a **great risk** that it must be reduced more than

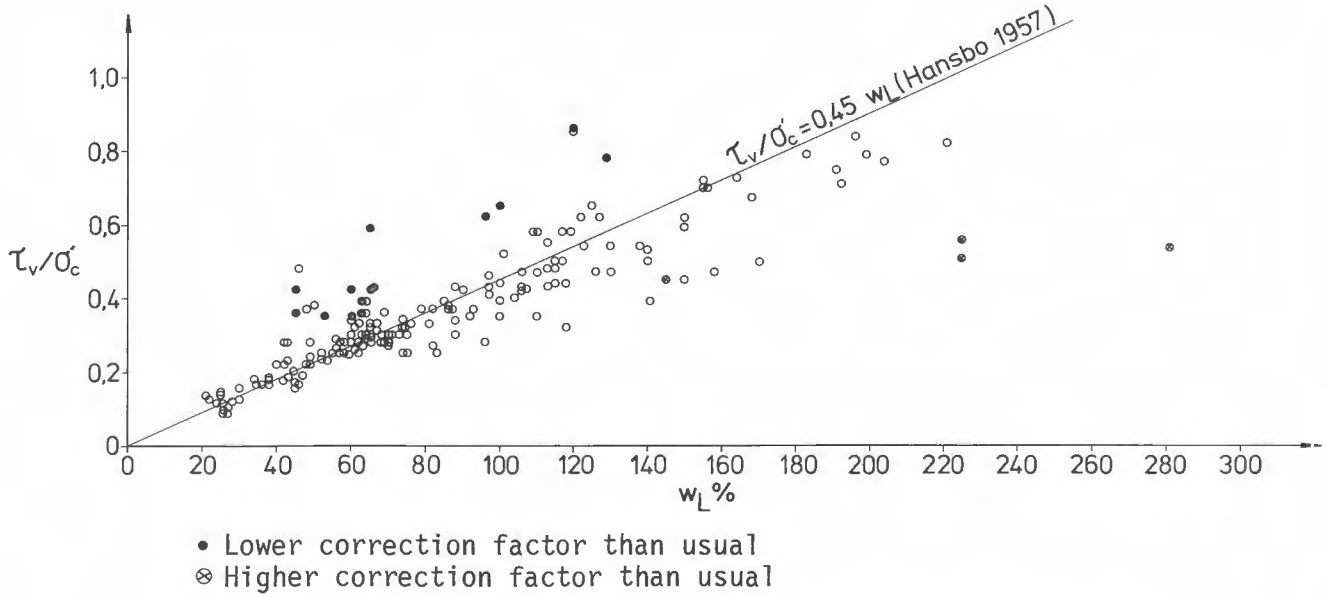


Fig 2. The relation τ_v/σ'_c as a function of liquid limit for a number of Scandinavian soils.

the general correction factor specifies and if it is unusually low supplementary investigations will **probably** give higher values of the shear strength.

No further conclusions can be drawn and the empirical relation can never replace real tests.

Estimation of corrected shear strengths on the basis of results from other soundings

The use of vane shear tests and fall cone tests involves certain problems which may effect the measured values of shear strength. In undisturbed sampling a stratigraphy with a stiffer layer overlying softer soil often entails that the samples in the upper part of the soft soil become disturbed. This problem almost always occurs at the lower boundary of the dry crust but can here be avoided by preboring through the crust. Varying stiffness at greater depths is more difficult to handle, which is often reflected in the results from fall cone tests. Sampling involves an unloading and redistribution of stresses in the samples, which also affects the results from fall cone tests. Based on experience, shear strength values measured in samples from greater depths than 10 to 15 metres are thus often too low.

Disturbance effects at the transition from stiffer to softer soil also occur in vane shear tests. When a vane apparatus type SGI with a casing for the vane is used, this disturbance is confined to the first test level below the stiffer layer. In very layered or varved soil a conutinous disturbance may occur. When a vane apparatus without casing is used the risk of disturbance is greater, as soil from an overlying stiffer layer may stick to the vane and all of the underlying softer clay profile may then be disturbed at the installation of the vane.

Occurrence of dense layers of silt and other strata may in some cases give too high values of shear strength. In other cases with for example layers of loose silt the shear strength values may be too low.

For estimation of the reasonableness of the corrected shear strengths, support may therefore have to be sought from other observations and empirical relations.

An important complement is the combined cone penetration test and pore pressure sounding. The equipments for this type of testing used in Sweden today are too coarse for a direct evaluation of the shear strength. Besides, there is no generally accepted method for evaluation yet. The development is promising, though, and some conclusions can be drawn from the test results. From the curves it can be seen whether the stiffness of the soil increases, decreases or is constant with depth. It can also be seen whether there are any layers. From the point resistance it can be observed whether the layers are stiff or loose and from the pore pressure it can often be concluded what kind of soil the layers contain.

If low shear strength values are thus obtained from vane shear tests and/or fall cone tests at a certain level and a loose layer is detected at the same level by cone penetration testing, the existence of the layer can be considered confirmed. If the cone penetration test does not register any loose layer at the level, the existence of a layer is more doubtful and if the test results show that an overlying stiffer layer has just been passed disturbance is often a probable explanation for the low shear strength values. Low values of shear strength can also be measured in connection with silt layers with high pore water pressures, especially in vane shear tests.

Estimation of corrected shear strengths on the basis of laboratory tests and empirical relations for undrained shear strength

In order to obtain a broader basis for the estimation of the reasonableness of corrected values of undrained shear strength from vane shear tests and fall cone tests, correlations can be made with empirical relations between undrained shear strength, preconsolidation pressure and liquid limit, if necessary after supplementary, more qualified laboratory tests. Effects of anisotropy cause these relations to vary with loading case. The loading cases are normally divided into active shear, direct shear and passive shear. These cases can be simulated in the laboratory in active triaxial tests, direct shear tests and passive triaxial tests, Fig. 3.

Compilations of test results from Scandinavian inorganic clays have given empirical relations for the way in which the undrained shear strength in the different cases varies with preconsolidation pressure and liquid limit, Fig. 4. A corresponding empirical relation for organic clays is not yet at hand.

The undrained shear strength that should be compared to the corrected shear strength values from vane shear tests and fall cone tests is the shear strength at direct shear in a horizontal slip surface.* This relations

* see Fig. 19

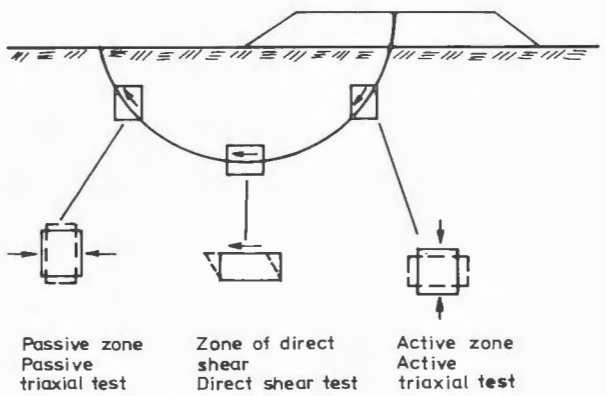


Fig 3. Division of loading cases and relevance of laboratory tests to different loading cases.

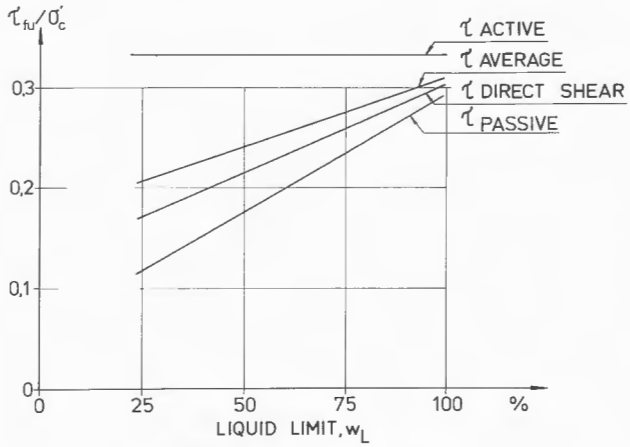


Fig 4. Empirical relations between preconsolidation pressure and undrained shear strength for Scandinavian inorganic clays.

gives shear strength values which are diretly applicable to calculations of slope stability where the inclination of the slip surface is small and which gives results that are somewhat on the safe side when applied to steeper slip surfaces. In calculations of stability of embankments on horizontal ground the average shear strength ($\tau_{AVERAGE}$) should preferably be used but compared with the difference the shear strength at direct shear is small.

Active triaxial tests generally give the highest value of undrained shear strength but these values are applicable only to zones with active shear. Passive triaxial tests give the lowest values and these may be relevant in cases with passive shear.

The effects of anisotropy are largest in low plastic clays and in these clays large differences in evaluated undrained shear strength may occur depending on whether anisotropy is considered or not. (Fig. 4).

For estimation of the shear strength the preconsolidation pressure σ'_c must be known. This value is normally determined by oedometer tests.

In, for example, investigations of the stability conditions in larger areas, a general picture of the preconsolidation in the whole area can often be obtained by performing a number of oedometer tests and by linking the evaluated preconsolidation pressures to the geological history of the area. This is especially valuable in areas with such stratigraphies that vane shear testing as well as fall cone testing involves problems.

If no oedometer tests have been performed the empirical relations in Fig. 4 can be used together with the existing effective overburden pressure. A lower boundary value of the undrained shear strength is hereby obtained. In this procedure, however, the risk of nonhydrostatic pore water pressures must be considered when the existing effective overburden pressure is estimated.

There is, of course, a certain spread in the empirical relations for undrained shear strength and the relations can never substitute for other tests but only be complementary to them. A special case where the empirical relations may overestimate the shear strength is in low plastic and highly sensitive clays. In these soils, however, vane shear tests and fall cone tests give low values of shear strength and low values of point resistance are also registered in cone penetration tests. In such cases the low values ought to be used and the empirical values should be disregarded. A lower boundary value of $\tau_{fu} \geqslant 0.12\sigma'_c$ may be considered, though. Undrained shear strengths lower than this value, have not, as far as known, been found for Scandinavian soils in either direct shear tests in laboratory or by back calculation of full scale failures in the field. (Liquefaction as well as thawing of frozen masses is disregarded.)

Selection of undrained shear strength

When the undrained shear strength to be used in calculation is selected an estimate has, as described above, to be made as to the reasonableness and the relevancy of the measured shear strengths and the applicability of the empirical relations, whereupon obviously erroneous or irrelevant values can be eliminated. Thereafter, the remaining values can be weighted and combined into an undrained shear strength that can be used in the calculations.

In this evaluation the corrected values from vane shear tests are very weighty unless they significantly deviate from Hansbo's relation. Corrected values from fall cone tests are also often considered relevant with the same limitation. In samples taken at greater depths than 10 to 15 metres the shear strength values from fall cone tests often are too low. Results from qualified laboratory tests are of very great weight provided that they are relevant to the particular loading case. Empirical relations between shear strength and preconsolidation pressure can be used to support and supplement determinations of the shear strength. Results from cone penetration tests can so far only be used to study the trend for shear strength evolution with depth and to evaluate the occurrence of layers.

Estimation of increase in shear strength at consolidation

When normally consolidated clay is loaded and consolidates the preconsolidation pressure and thereby the undrained shear strength increase. The increase in shear strength can be assumed to follow the increase in preconsolidation pressure in accordance with relations determined by tests or by empirical relations: $\Delta\tau_{fu} = k \cdot \Delta\sigma'_c$ where k depends on the loading case. (See Fig. 4).

This increase in shear strength can be utilized in stage construction where the load is applied in steps with intervals for consolidation as well as in old constructions, where the increase in strength beneath the construction can be utilized when these are to be added or enlarged.

Direct measurement of this increase of shear strength often involves difficulties as well as expense. Holes have to be made through the construction and the shear strengths vary laterally if the construction has a limited width. A simplified way to estimate the increase of shear strength below, for example, an old road embankment is to make the assumption that the shear strength increase is confined to the extent of the embankment and that the increase follows the empirical relations. Fig. 5. In this context, it should be considered that the change in preconsolidation pressure is different in different parts below the embankment. The increase in vertical stress $\Delta\sigma'_{c_{xz}}$ should be calculated according to theory of elasticity. Furthermore it must be confirmed that the loading really has brought a change in preconsolidation pressure, that is, that the soil has consolidated for the load increase.

As criterion of consolidation of the soil, observations showing that the settlements have stopped can be used. Alternatively, pore pressure measurements can be made. In this case the pore pressures should be measured in the middle of the layers where an increase of the preconsolidation pressure is expected.

The empirical relations are valid only for inorganic soils and if the same procedure is to be used in organic soils, the relations between undrained shear strength and preconsolidation pressure must be determined by laboratory tests for the particular case.

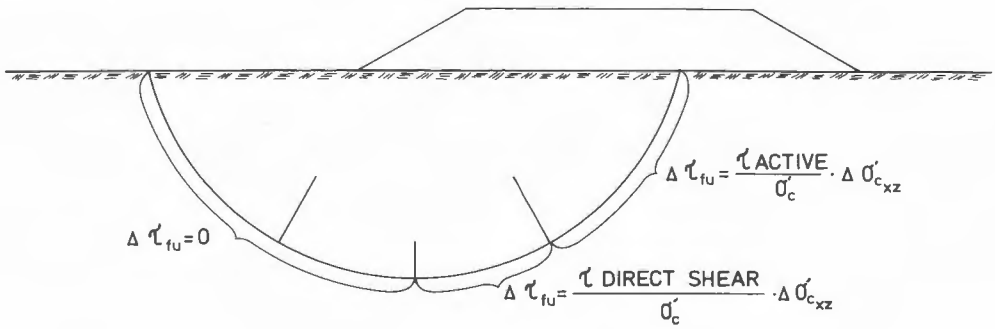


Fig 5. Simplified estimation of increase in shear strength due to consolidation below an embankment.

SHEAR STRENGTH IN DRY CRUST, OVERCONSOLIDATED CLAY AND IN CONNECTION WITH LAYERS WITH HIGH PORE WATER PRESSURES

The undrained shear strength is often regarded as a constant although tests have shown that it decreases when the effective pressure decreases and the overconsolidation ratio increases (e.g. Ladd et al, 1977).

However, when the shear strength is selected, the drained shear strength also has to be considered. When the latter becomes lower than the undrained shear strength, it also becomes the strength that should be used in calculations, except in a very short time aspect. Drained shear strength becomes lower than the undrained shear strength at an overconsolidation ratio of about 2. At this overconsolidation ratio the decrease in undrained shear strength in relation to the normally consolidated state is small. The decrease in shear strength with increasing overconsolidation ratio thus normally becomes of limited interest, Fig. 6.

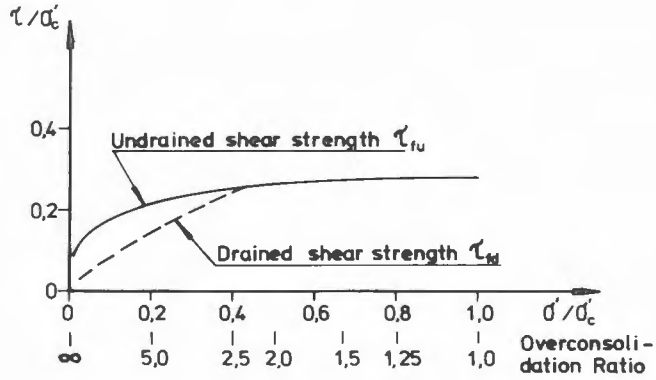


Fig 6. Variation in shear strength with overconsolidation ratio at direct shear.

DRAINED SHEAR STRENGTH

The drained shear strength $_{fd}$ is expressed by the shear strength parameters c' and ϕ' in such a way that $\tau_{fd} = c' + \sigma' \tan \phi'$ where σ' is the effective stress perpendicular to the shear surface. As a lower boundary value for drained shear strength in overconsolidated soils, the shear strength parameters c' = 0 and ϕ' = 30° can be used, provided that the undrained shear strength is not hereby exceeded. These values can be used for dry crust and overconsolidated clays, whereby the influence of cracks and fissures is taken into account. They can also be used for silt at stress changes which cause a decrease in effective stresses in the silt, e.g. at increasing pore pressures in silt layers. In calculations of stability of natural slopes, the shear strength in the cracked dry crust in the active zone is assumed to be zero. In unfissured, overconsolidated clay the shear strength parameters c' = 0.03 σ'_c and ϕ' = 30° may be used.

The values given above have been selected with caution. If comprehensive testing has indicated other values, the test values may naturally be used.

The angle of friction in coarser soils is selected from test results or on the basis of particle size distribution and relative density from Table 1.

TABLE 1. Approximate values for angle of friction in cohesionless soil.

Relative density	Type of soil						
	Silt	Sand	Gravel	Sandy till	Gravelly till	Macadam	Rock fill
Loose	26°	28°	30°	35°	38°	30°	40°
Dense	33°	35°	37°	42°	45°	38°	45°

Relative density for natural soils is normally estimated from results from sounding (e.g. SGI Information 2).

In loose silt the undrained shear strength must also be considered. This shear strength is normally determined by vane shear tests and the relevancy of the corrected must be estimated as described above.

CHOICE OF DRAINED OR UNDRAINED SHEAR STRENGTH

Consideration to the overconsolidation ratio is automatically taken at the choice of shear strength if a combined analysis is made (see SGI Report No. 19). In other cases the "undrained shear strength" in the undrained analysis must be modified by a manual comparison between drained and undrained shear strength in the different parts along the shear surface and selection of the lowest strength for each part. In this comparison the effective stress state existing after a change in the loading situation should be considered. This "undrained" analysis should also be controlled by a completely drained analysis.

EXAMPLE

As an example, a section in a larger investigation of slope stability along the river Norsälven has been chosen, Section IX drawing Nos. 8 and 9. The soil consists of a 3 to 5 m thick layer of silt and sand which covers a varved clay with frequent silt layers. The layer of varved clay extends to great depths.

In the section the following have been performed: undisturbed sampling with standard piston sampler, sampling with auger vane shear tests, weight sounding static penetration testing, cone penetration testing, pore pressure sounding and pore pressure measurements. The investigations are thus fairly comprehensive. As it was known already that the stability of the area was poor and that the investigations due to the stratigraphy of the soil would involve some difficulties, the field investigations have been performed with special care and most of the different ways of estimating the shear strength have been used. In addition to the routine tests a large number of oedometer tests have been performed in order to establish the preconsolidation in various parts of the section.

Geologically, the ground surface in the section was probably originally mainly horizontal. The river has then eroded downwards so that the bottom of the river is now 13 metres below the surrounding ground surface.

In connection with the erosion the ground water level which was originally close to the ground surface has been lowered so that at the time of the measurements it was situated just above the lower boundary of the silt and sand layer. The lowest seasonal ground water level will probably be at this boundary. This general picture is supported by the results from the oedometer tests. Figs. 7 and 8. The general picture is further supported by the results from the other sections in the investigation.

Fig. 7 shows the test results from point 0/061.5 in Section IX. The preconsolidation pressures from the oedometer tests show a certain spread which can be expected considering the difficulties in taking undisturbed samples in such a stratigraphy. The general trend supports the assumption of a soil normally consolidated for a ground water level at the lower boundary of the silt and sand layer. In the figure the corrected values of undrained shear strength obtained by vane shear tests and fall cone tests are shown together with empirical shear strengths. The latter are based on the curve for the variation of preconsolidation pressure with depth and measured liquid limits. The relation for the case of direct shear has been used. The trend for shear strength variation obtained by cone penetration testing is also shown. The shear strengths from vane shear tests and fall cone tests are equal down to a depth of 13 metres from where the fall cone values consistently become lower than the values from vane shear tests. The shear strength values indicate a weaker layer at the +34 m to +35 m level but a comparison with the cone penetration test, the static penetration test and the pore pressure sounding shows that this is probably an effect of passing a stiffer silt layer at the +35 m level so that the +35 m to +34 m layer has become disturbed. There is nothing in the other soil parameters, such as sensitivity or liquidity index, that indicates anything special at this level. Like the trend obtained from cone penetration testing the shear strengths obtained by vane shear tests increase from the +38 m level and downwards. The results show a certain spread at greater depths but when the empirical shear strength values are added a considerably broader basis is obtained for the evaluation of the undrained shear strength.

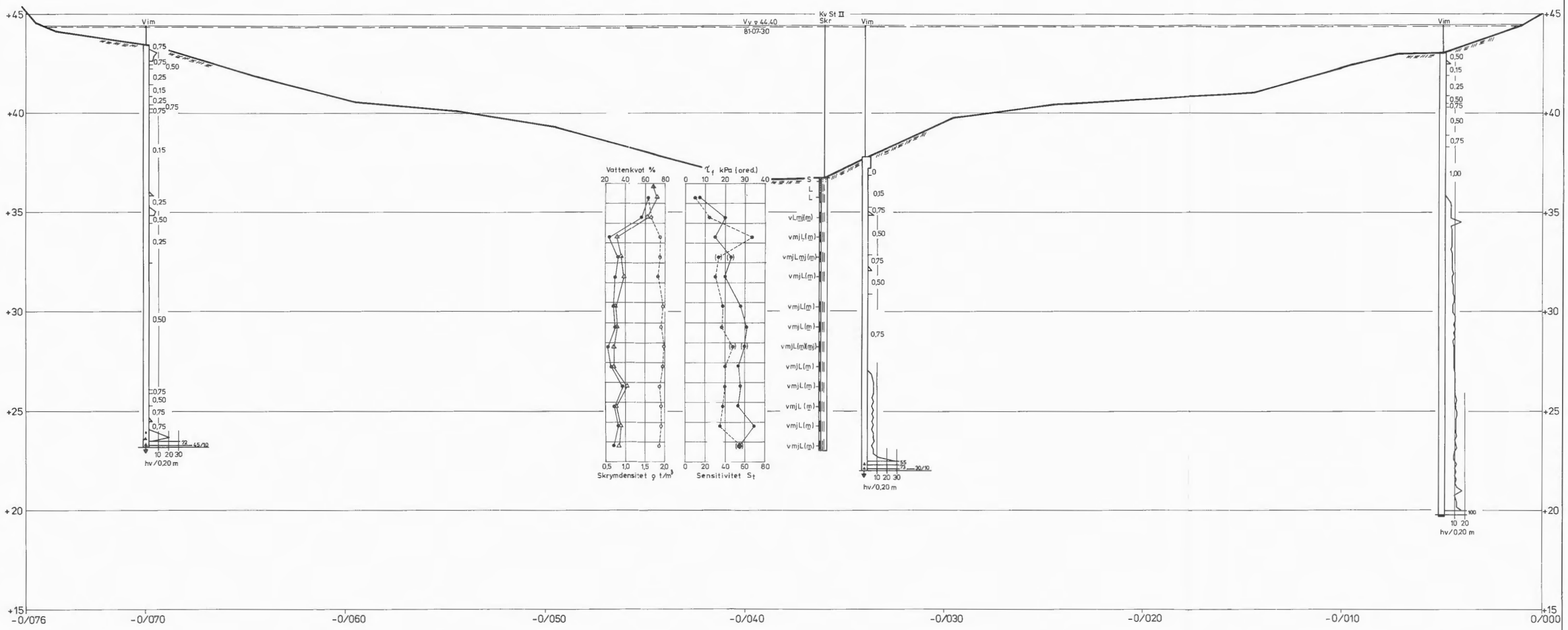
The evaluated undrained shear strength is thus mainly based on the corrected shear strength values from vane shear tests. The empirical shear strengths and the results from the cone penetration test are used to support the trend in shear strength development versus depth and to even out the spread in the corrected shear strength values from the vane shear tests.

In Fig. 8 the corresponding results from point -0/036 are shown. This point is situated close to the middle of the river. At this point only weight sounding and standard piston sampling have been performed. The results from the oedometer tests have been compared to maximum previous overburden pressure with the assumption that the soilprofile originally was identical to that of the surrounding ground. The comparison shows that the soil is normally consolidated for a ground surface level with the surrounding ground surface and a ground water level about 2 m below this ground surface. This is in good agreement with the results from the rest of the area.

The figure shows the corrected shear strength values from fall cone tests and the empirical shear strength for direct shear from preconsolidation pressures and liquid limits. The two sets of shear strength values are close down to a depth of 10 m below the river bottom. Below this level the values from fall cone tests consistently become lower. As there is nothig special in terms of sensitivity or other properties or comparisons with the results from vane shear tests at point 0/009 indicates that any weaker layers the empirical shear strengths are considered as most relevant.

Due to the erosion the upper layers of the clay are heavily overconsolidated. A comparison between the undrained shear strength and the empirically estimated drained shear strength shows that the drained shear strength is lower than the undrained down to a depth of about 6 metres below the river bottom.

The evaluated "real" shear strength to be used in the stability calculations for this point thus differs somewhat from the results of the only direct determinations of shear strength that have been made here. In the upper layers the shear strength is estimated to be lower than the undrained shear strength determined by fall cone tests due to the overconsolidation ratio and at great depths it is estimated to be higher than the corrected values from fall cone tests, due to the effect of stress relief and redistribution at sampling at greater depths. That an estimated shear strength higher than the measured values from fall cone tests may be used at this point without supporting investigations is due to the comprehensive investigations in the surrounding areas. Without this support the presented evaluation could not have been made.



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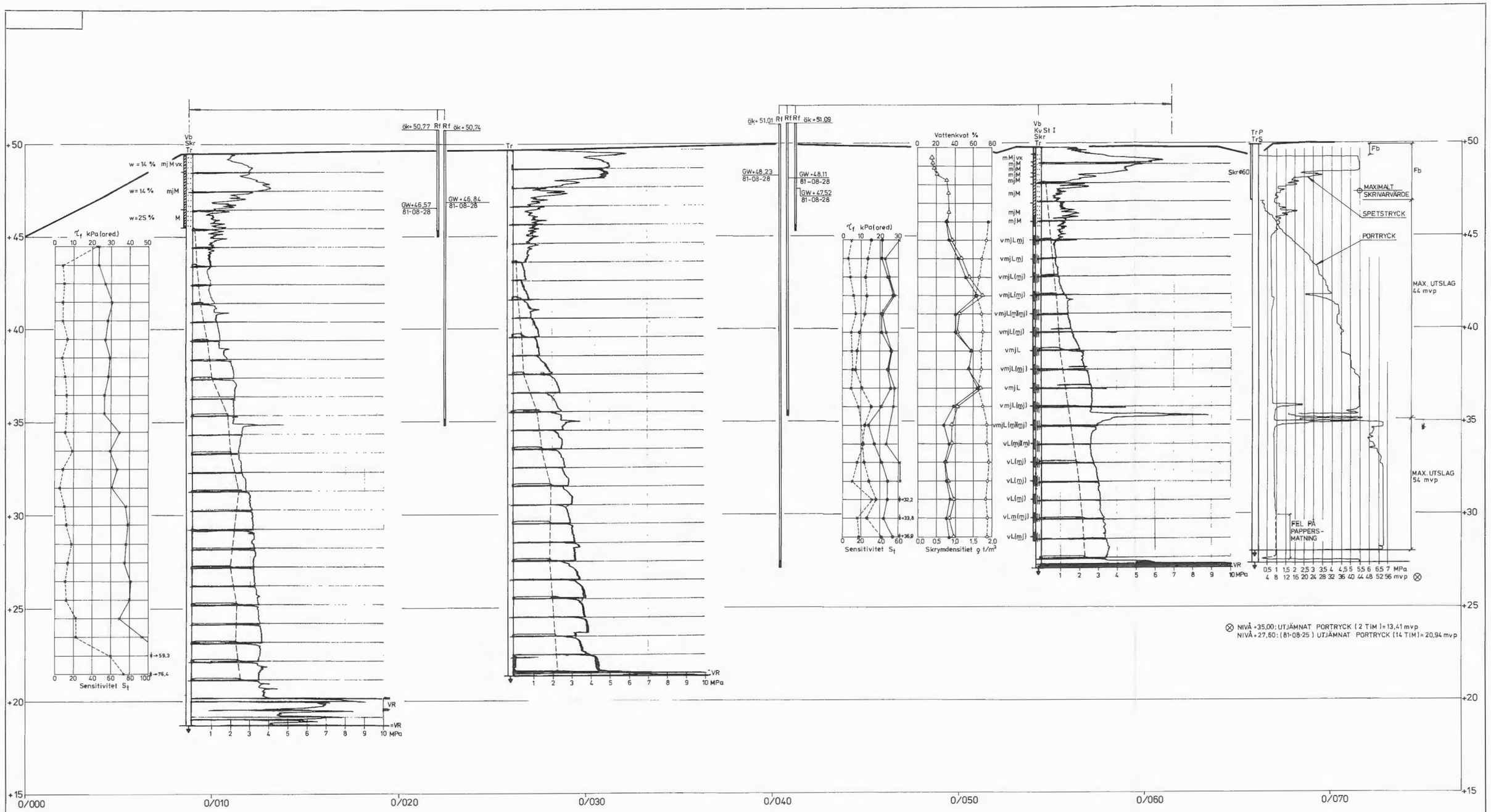

REV	REVIDERINGEN AVSER	DAT	SIGN

STATENS GEOTEKNISKA INSTITUT

FACK, 581 01 LINKÖPING 1 TEL. 013-11 51 00

KARLSTAD KOMMUN
NORSÄLVEN - STABILITETSUTREDNING

VÅLBERG - SEKTION IX

GEOTEKNISK UNDERSÖKNING

FÄLTARB	RITARB	HANDLÄGGN	SKALA	UPPDRAG	RITN NR	REV
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LINKÖPING

Section IX

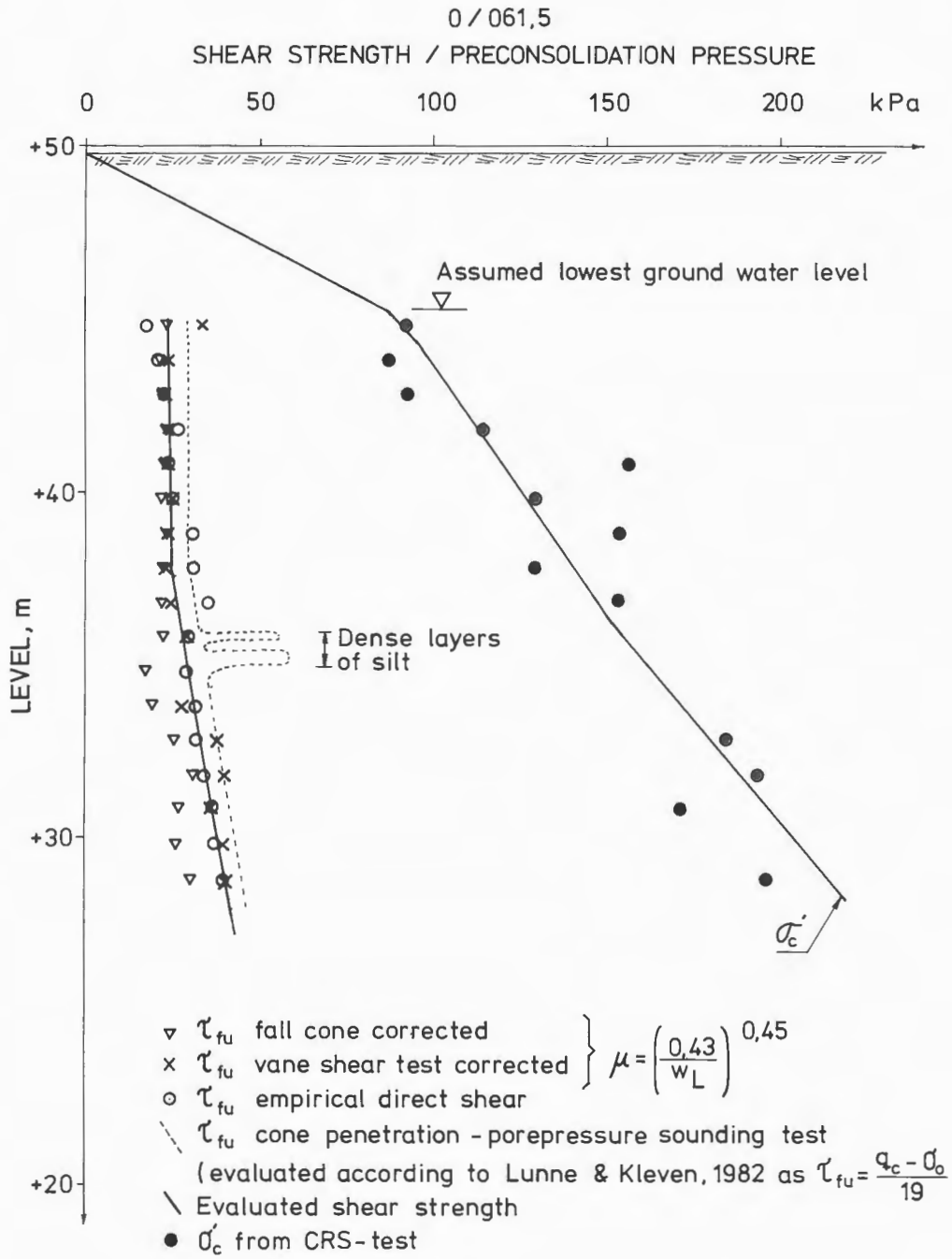

Fig 7. Test results and evaluated shear strength at point 0/061.5.

Section IX

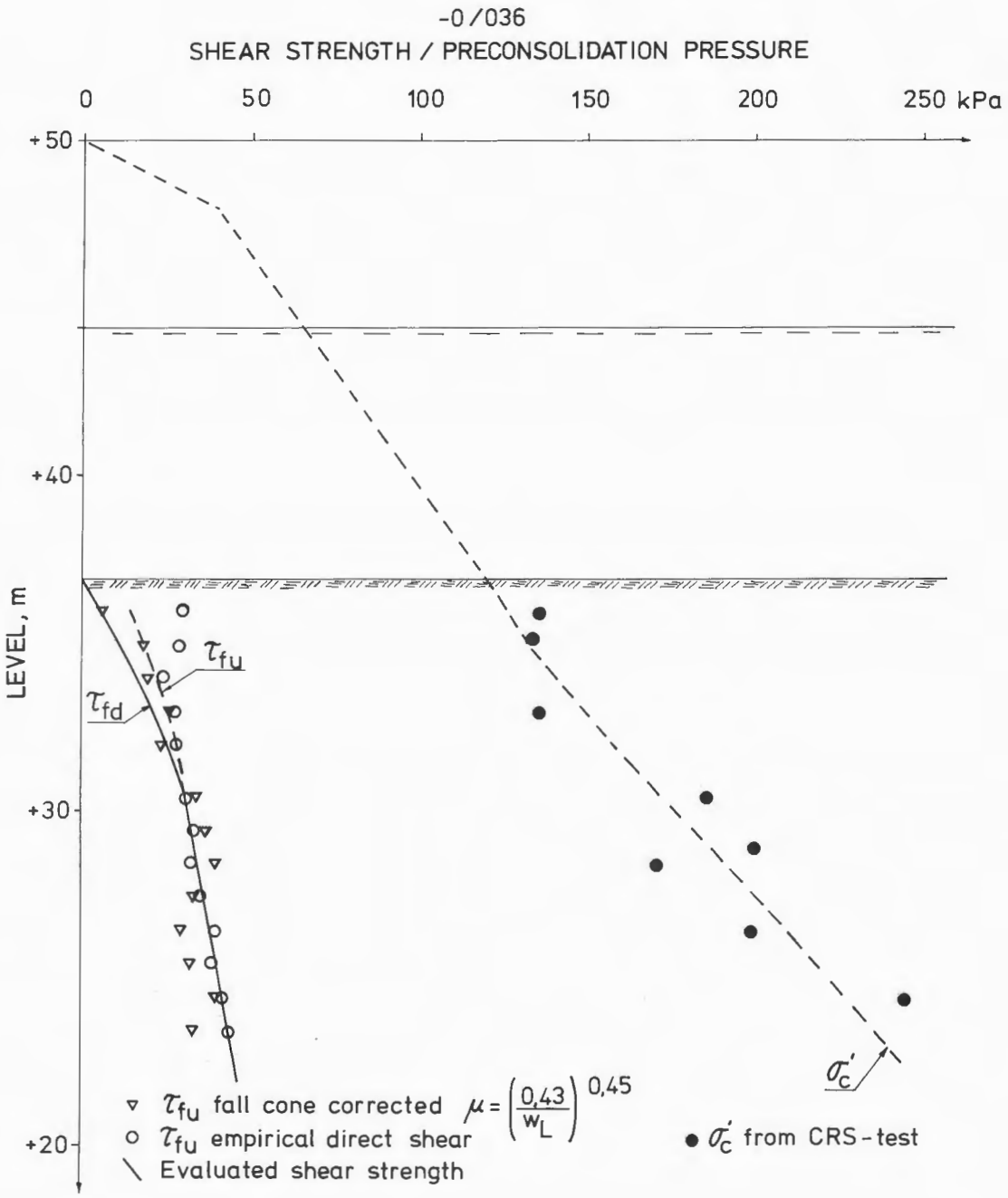

Fig 8. Test results and evaluated shear strength at point 0/036.

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APPENDIX

BACKGROUND TO THE NEW RECOMMENDATION FOR CORRECTION OF VALUES OF SHEAR STRENGTH OBTAINED BY VANE SHEAR TESTS AND FALL CONE TESTS TOGETHER AND TO EMPIRICAL RELATIONS FOR UNDRAINED SHEAR STRENGTH

Introduction

In Sweden, undrained shear strength is mainly estimated by fall cone tests and vane shear tests. From the values obtained the geotechnician makes a judgement of what can be regarded as "safe" measured values and these values are then corrected with the SGI correction factors from 1969. These correction factors have now been used for 15 years.

Provided that the corrected shear strength is used in the normal way, together the appropriate safety factors, the procedure works fairly well. However, the user must be aware of the development that has led to the SGI correction factors and how these are intended to be used.

The use of primarily the vane shear test has spread all over the world. New research results have been obtained and a number of different correction factors have been suggested. To enable a separation of real differences in results and soil properties from differences in treatment of the test results, the background and prerequisites for correction factors must be taken into account. In recent years, other types of estimations of the undrained shear strength than fall cone tests and vane shear tests have come into use, for example cone penetration, pore pressure sounding and pressometer tests in the field and triaxial tests and direct shear tests in the laboratory. This is primarily the case abroad, but is valid also in Sweden.

Statistical methods of estimating the averages of results are used more and more and partial safety factors are now planned to be introduced in geotechnics. If statistical averages and partial safety factors are to be used, correction factors which themselves imply another treatment of the test results cannot be used. In these cases, new correction factors based on averages of measured strength values must be used.

In view of this, there is reason for noting the historical development which has led to the SGI correction factors from 1969 and the way in which they were supposed to be used as well as for providing recommendations on the correction factors to be used befor cal-

Historical development up to 1969

Estimation of shear strength and stability started in Sweden in connection with the work of the Geotechnical Commission of the Swedish State Railways and the Committee on Stability Problems in Gothenburg Harbour. These led to the method of calculating stability with the Swedish Slip Circle Method (Hultin 1916, Pettersson 1916 and Fellenius 1918, 1919, 1926, 1927, 1929) and to the fall cone test which was developed by John Olsson (The Commission of the Swedish State Railways. Final Report 1922).

The fall cone test was calibrated by tests on samples taken with a newly constructed sampler for "undisturbed" samples, which were compared to results from a series of load tests on piles made by Wendel in 1900. The strength values from fall cone tests were also compared to landslides which were back-calculated with the use of undrained shear strength and slip circles and this testing procedure was recommended in the final report from the commission.

The sampling technique was improved and piston samplers came into use (Olsson 1925, Pettersson 1933, Caldenius 1938). The fall cone test was recalibrated for these improved samples, versus various tests for determination of strength in the field, load tests in full scale, full scale failures and landslides and later on also direct shear tests in the laboratory. These calibrations also led to the first correction factors for undrained shear strength measured by fall cone tests in organic and high-plastic clays (Skaven-Haug 1931, Olsson 1936-37, Fellenius 1938, Caldenius 1938). In the calibration of the fall cone test versus landslides also other types of failure surfaces than slip circles were used. It was also made clear that the strength was dependent on the preconsolidation pressure (Fellenius 1936).

The corrected undrained shear strength from fall cone tests was used in all types of stability problems such as excavations, earth pressures and stability of road berms with and without loading berms (Jakobson 1946).

A new type of shear apparatus was constructed in 1936 by Kjellman (1942, 1950) for undrained as well as drained tests. Triaxial tests have also been used in Sweden since 1940.

The vane apparatus for determination of undrained shear strength in the field was introduced in its present shape by Cadling and Odenstad in 1950. This apparatus was calibrated versus a number of slides that had occurred and versus one load test. At first, the values from the vane shear tests were used without any correction. At the end of the 1950s new and improved samplers were developed which led to the new standard piston sampler (Kallstenius 1963). Hansbo (1957) recalibrated the fall cone test on the

new samples versus the vane shear test. This new evaluation differed somewhat from the previous method but not very significantly. Earlier experience had shown that strength values measured by the fall cone test must be corrected with regard to liquid limit. The realization that undrained shear strength from vane tests had to be corrected in the same way as the fall cone values also spread with time.

The correction factors have, like the evaluation of the undrained shear strength, varied with the times according to the kind of sampling equipment used. To sum up experience after the introduction of the standard piston sampler in 1963 and to obtain uniform correction factors for modern sampling technique a technical meeting was held at the Swedish Geotechnical Institute in Stockholm in 1969 (SGI 1970).

SGI correction factors of 1969

At the meeting at SGI a recommendation was made as to the correction factors that should be used for undrained shear strength values estimated by fall cone tests and vane shear tests. The correction factors were based on the liquid limit of the soil. Fig. 9.

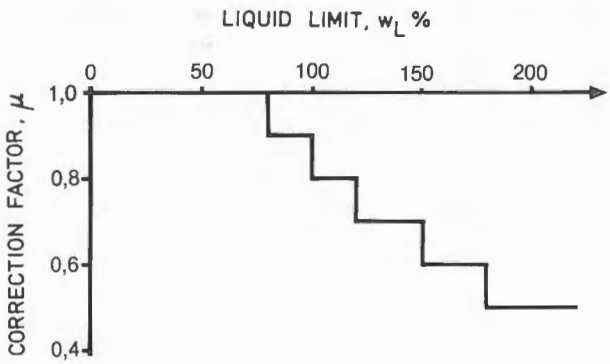


Fig. 9. Correction factors recommended by SGI 1969.

Many participants at the meeting wanted to recommend lower correction factors (greater reduction). The question was resolved by applying the recommended correction factors at cautiously selected values or "low estimates of averages" of the measured strength values.

SGI correction factors of 1969 thus imply that a further reduction is made of the real averages of corrected strength values.

This means that the correction factors must be lower than those given in Fig. 9 when the strength is based on statistical averages of evaluated shear strength.

The corrected strength was intended to be used in all types of calculations and strength anisotropy was not considered. If strength anisotropy has been considered at all in Sweden, the required safety factor has been modified for different loading cases.

Bjerrum's correction factors

Bjerrum (1972 and 1973) made a comprehensive compilation of failures and landslides based on what had been reported in the literature and on the experience of the Norwegian Geotechnical Institute from Norway and Bangkok. After analysis of this material Bjerrum suggested new correction factors. These were based on averages of the shear strength values measured by the vane shear tests and related to the plasticity index I_p.

In Bjerrum's studies a great number of low plastic clays from Norway and clays with varying plasticity from mainly Europe and North America were included. No high plastic or organic soil from Scandinavia was included in the study. The correction factors for this type of soil were mainly based on investigations of organic Bangkok clay. However, comparisons with Swedish soils with corresponding plasticity show that the properties of the Bangkok clay do not correspond to those normal for Swedish conditions.

Andréasson (1974) recalculated Bjerrum's values to obtain corresponding correction factors based on the liquid limit instead of the plastic index. A comparison with the SGI recommendation from 1969 showed that with the Bjerrum correction factors the strength was reduced more for low plastic clays but less for high plastic organic clays. Taking into consideration the proposed use of the SGI correction factors together with cautiously selected shear strength values, the final result was that the evaluated shear strength was almost the same,except for organic clays. The Bjerrum and Andréasson studies did not therefore lead to any change in the SGI recommendations.

Later studies of correction factors

The use of the vane apparatus spread and new suggestions for correction factors were made, for example Pilot 1972, Dascal and Tournier

1975. In 1976 Aas made a suggestion for how the vane shear values could be corrected in Norwegian clays with respect to the relation between the measured strength value and the existing overburden pressure.

An extensive review of the different correction factors, which also included some cases from Sweden not earlier taken into account, was made in 1977 by Helenelund, who at that time was visiting research worker at SGI. Based on this material a correction factor in the order of

$$\mu \simeq \frac{1.45}{1+w_L}$$

was suggested.

This correction essentially corresponds to the Bjerrum correction but the correction of the measured strength becomes greater for organic high plastic soils.

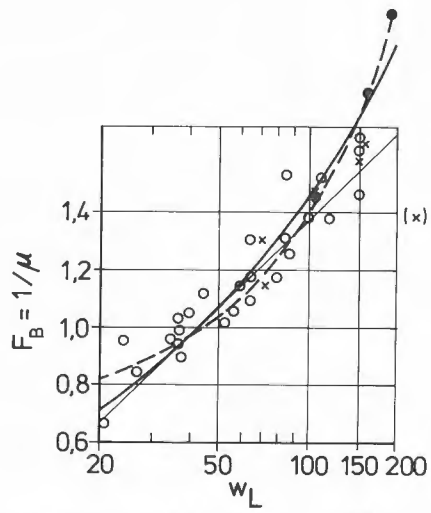
By inserting the Hansbo relation $\tau_v = \sigma'_c \cdot 0.45\, w_L$ Helenelund's suggested correction can also be written

$$\mu \simeq \frac{1.45}{1+2.22\, \tau_v/\sigma'_c}$$

Recently an extensive investigation of the properties of organic soil has been completed in Finland (Slunga 1983) and a similar investigation is in progress at SGI.

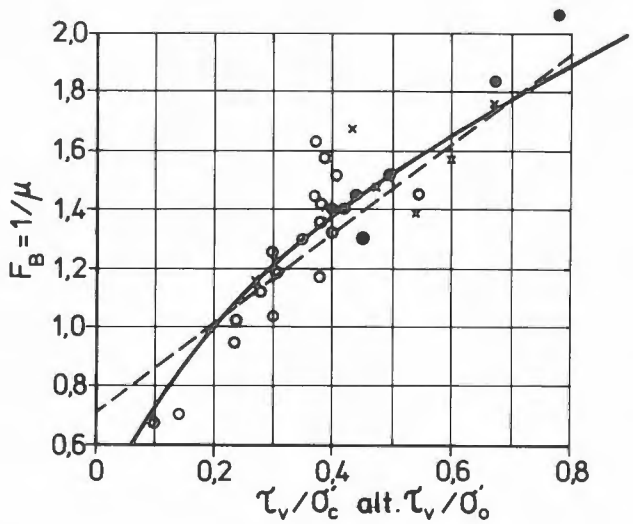
Further results from Norway have also been presented by Aas et al 1984.

All the compilations of test results involve a spread and suggested correction factors become an adaption to the curves for averages.



○ Data from Andréasson 1974
× τ_v / τ_{LAB} Slunga 1983
● τ_v / τ_{LAB} organic soil (SGI)
—— Andréasson's suggestion for correction 1974
--- $\mu = \frac{1,45}{1+w_L}$ (Helenelund 1977)
—— $\mu = \left(\frac{0,43}{w_L}\right) 0,45$

Fig. 10. Evaluated safety factors at failure $F_B = 1/\mu$ as a function of liquid limit compared to suggested correction factors.



○ Data from Helenelund 1977
× τ_v / τ_{LAB} Slunga 1983
● τ_v / τ_{LAB} organic soil (SGI)
--- $\mu = \frac{1,45}{1+2,22\, \tau_v/\sigma'_c}$ (Helenelund 1977)
—— $\mu = \left(\frac{0,193}{\tau_v/\sigma'_c}\right) 0,45$

Fig. 11. Evaluated safety factors at failure $F_B = 1/\mu$ as a function of the relation between the strength value measured by vane shear tests and the preconsolidation pressure (alternatively effective overburden pressure) compared to suggested correction factors.

Fig. 10 shows the values evaluated by Andréasson 1974 supplemented with relations between results from vane shear tests and qualified laboratory tests (averages from direct shear tests and active and passive triaxial tests) from Slunga 1983 and from research at SGI. In the figure the curves corresponding to the corrections suggested by Andréasson 1974 and Helenelund 1977 are inserted together with the correction now recommended by SGI.

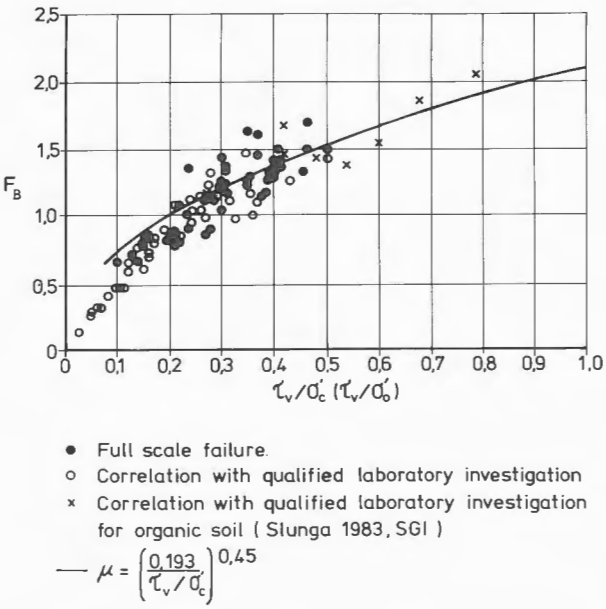


Fig. 12. Evaluated safety factors at failure (Aas et al values) completed with values from organic soils compared to correction factors recommended by SGI.

Fig. 11 shows the values evaluated by Helenelund 1977 supplemented in the same way as the values evaluated by Andréasson in Fig. 10 with values from organic soils. Helenelund presented his values as calculated safety factors at failure $F_B = 1/\mu$ versus τ_V/σ'_O. As τ_V is dependent on σ'_C this presentation gives an unnecessarily large spread since the values from normally consolidated and overconsolidated clays are mixed. To the extent that information has been available the values have therefore been converted and existing effective overburden pressure σ'_O has been replaced by the preconsolidation pressure σ'_C. Some cases, where investigation has shown that the drained strength has been lower than the undrained strength or complementary information is missing, have been excluded. Curves corresponding to the correction suggested by Helenelund 1977 and the correction now recommended by SGI (both rewritten with the Hansbo relation $\tau_V = \sigma'_C \cdot 0.45\ w_L$) are inserted in Fig. 11.

At the Nordic Geotechnical Meeting (NGM) in Linköping 1984 Aas et al presented a compilation of comparisons for the relation between F_B and τ_V/σ'_O, Fig. 12. The compilation included comparisons with full scale failures in the field as well as comparisons with qualified laboratory investigations. The points in the figure originate from normally consolidated and slightly overconsolidated clays and a certain displacement of the points to the left in the figure would occur if all points could be calculated versus σ'_C. To complete the picture, the points for organic soils from Slunga 1983 and SGI have been inserted.

Results from research on vane shear tests

During the 1960s and the beginning of the 1970s extensive research was carried out to determine which factors influence the results from vane shear tests. The factors investigated were shape and construction of the vane, waiting time after installation before strength testing starts and together speed of rotation during the test.

The shape and construction of the vane proved to have great importance for the results. In Sweden, however, vanes with the relationship 2:1 between height and width of the blades are always used, and the empirical experience is based on this type of vane. In Sweden two types of vane shear equipment are used today: vane shear equipment of the SGI type where the vane is withdrawn in a protective casing during most of the driving and vane shear equipment of the Nilcon/Geotech type,where the vane is driven directly into the soil. As the latter vane is unprotected during driving the design must be made somewhat more robust than for the SGI equipment. In the SGI vane shear equipment the vane rod is also encased. Torstensson (1977) has found that at rotation speeds recommended by the manufacturer failure occurs approximately three times faster with the Nilcon/Geotech apparatus than with the SGI apparatus. The results often seem to be compatible.

The SGI apparatus is a relatively heavy piece of equipment while the Nilcon/Geotech apparatus is very easy to handle. On the other hand, the SGI apparatus would normally disturb the soil less and in addition the vane is scraped clean after each test. With the Nilcon/Geotech apparatus there is a risk that clay from a firmer layer can stick to the vane and accompany the vane into looser layers with disturbance of the soil as a result. Preboring through the dry crust is therefore al-

ways recommended when using the Nilcon/ Geotech apparatus. Even then, problems can occur in layered or varved soil sensitive to disturbance.

Through studies of the influence on the waiting time between installation of the vane and the shear tests, it has appeared that the longer the waiting time the higher the measured shear strength is up to a certain waiting time (< 1 day), after which the value becomes constant. The increase in strength with waiting time could be due to reconsolidation after the disturbance occurring at installation of the vane. Recommended procedure is that the strength test be started within 5 minutes after installation. Within this period no significant increase of the strength value occurs.

Compilation of the relations between measured strength after one day of waiting, τ_{1D}, and measured strength according to the standard procedure τ_0, shows that the relation τ_{1D}/τ_0 increases considerably at decreasing liquid limit, which means that the disturbance at installation of the vane is normally greatest in low plastic clays. The spread also increases at decreasing liquid limit, Fig. 13.

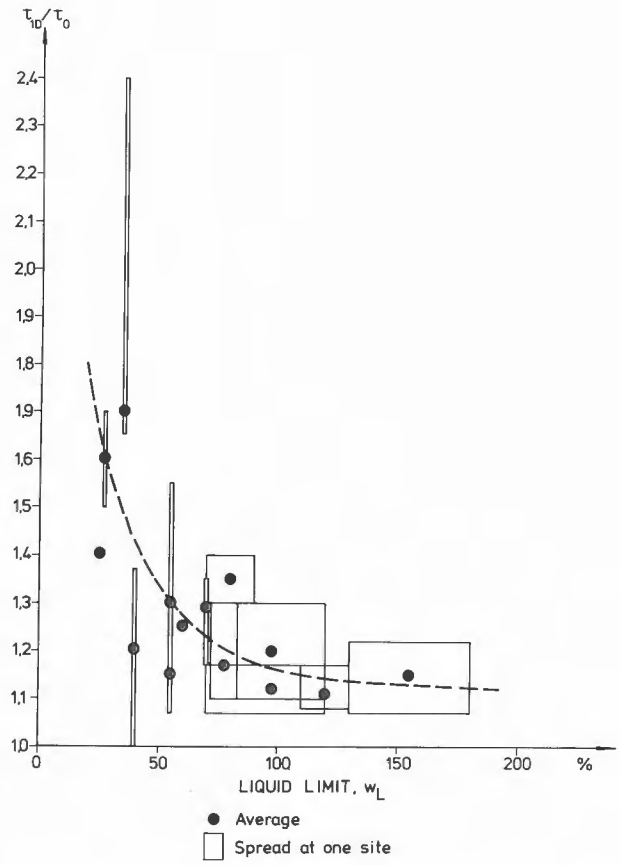


Fig. 13. Comparison between strength values measured by vane shear tests after one day of waiting, τ_{1D}, and values meured according to the standard procedure, τ_0.

In studies of the influence of speed of rotation on measured strength values, a connection between measured strength values and speed of rotation has in most cases been found. This can be written

$$\tau_t/\tau_1 = \left(\frac{t}{t_1}\right)^{-\beta}$$

where

τ_t = measured strength value at a rotation speed which gives the time t to failure

τ_1 = reference strength value measured at a rotation speed which gives time t_1 to failure

t_1 = reference time. Usually 1-3 minutes corresponding to recommended procedure

β = coefficient

Fig. 14 shows measured values of the coefficient from vane shear tests supplemented by β-values from other types of tests on organic soil, collected from Slunga 1983.

As is shown in Fig. 14 the rate effect decreases somewhat at decreasing plasticity.

In a summary of the results from research on vane shear tests, Torstensson (1977) showed that if both disturbance and rate effects are taken into account for time to failure of about one week, strength values are obtained that are about the same as the strength values measured by the standard procedure and corrected according to Bjerrum's recommendation of 1973.

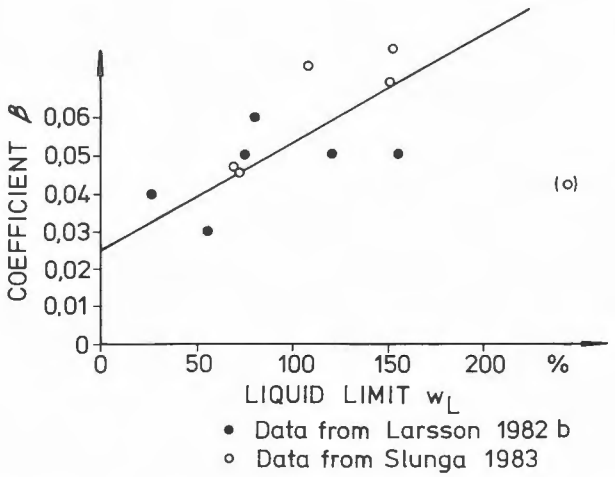


Fig. 14. The coefficient β as a function of liquid limit.

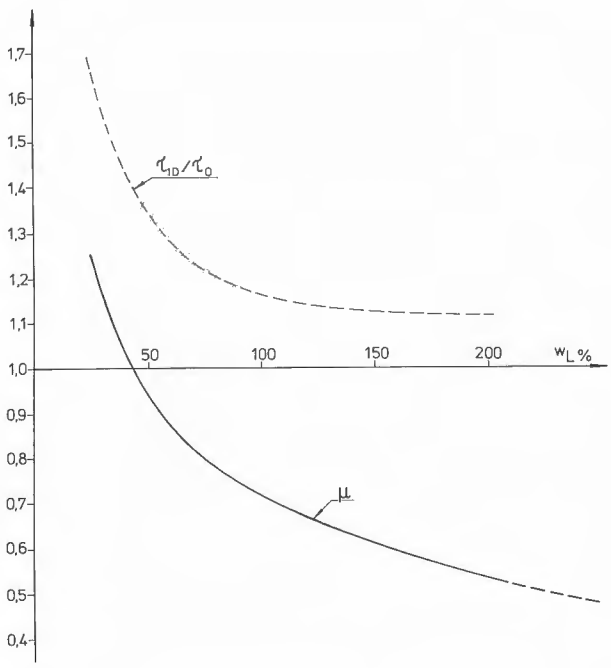


Fig. 15. Correction factor μ for strength values measured according to standard procedure by vane shear tests including effects of disturbance at installation of the vane and rate effects for time to failure of 1 week to 1 month.

If the correction for disturbance at installation obtained in Fig. 13 is corrected for rate effects so that, instead of applying to a time to failure of 1-3 minutes, it corresponds to more normal times to failure of 1 week to 1 month, a total correction for the measured strength according to the standard procedure is obtained, Fig. 15. The correction factor μ has been calculated as $\mu = 10000^{-\beta} \cdot \tau_{1D}/\tau_0$ and has been assumed to follow the relation in Fig. 14.

As the disturbance has been found to be greatest in low plastic clays where the rate effects are smallest, the final result is that the strength values measured by the normal procedure in low plastic clays are not be reduced and that the correction factor may even be greater than 1. The higher the plasticity is, the smaller is the disturbance and the greater the rate effects become. Consequently, the correction factor decreases with increasing plasticity.

Empirical relations for undrained shear strength

Different values of the undrained shear strength are obtained in different tests depending on the loading case. The shear strength is thus anisotropic. A hypothesis for the variation of the shear strength with the loading case in inorganic clays was presented by Larsson in 1977.

The undrained shear strength is usually divided into shear strengths in three main types of loading; active shear when the vertical stress is the largest principal stress; passive shear when the horizontal stress is the largest principal stress and direct shear where the normal stress towards the horizontal slip surface is constant, but the shear stresses increase. In the laboratory the three loading cases can be simulated by active and passive triaxial tests and direct shear tests. Fig. 16.

Results from active and passive triaxial tests and direct shear tests on Scandinavian clays reported in the literature together with consistency limits and preconsolidation pressures were compiled together with unpublished data from investigations at SGI in 1980. (Larsson, 1980). Since then additional data have beem obtained and is included in Fig. 17.

There are no corresponding empirical relations for organic clays as yet, but research on the properties of these soils is going on. The results from direct shear tests are close to the average of active and passive triaxial tests and direct shear tests.

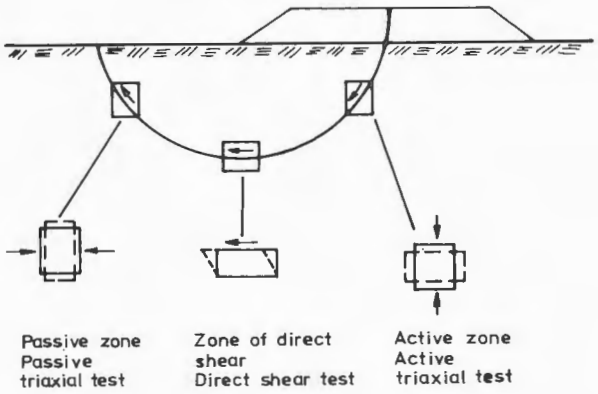


Fig. 16. Main types of loading cases with respect to soil anisotropy and corresponding laboratory tests.

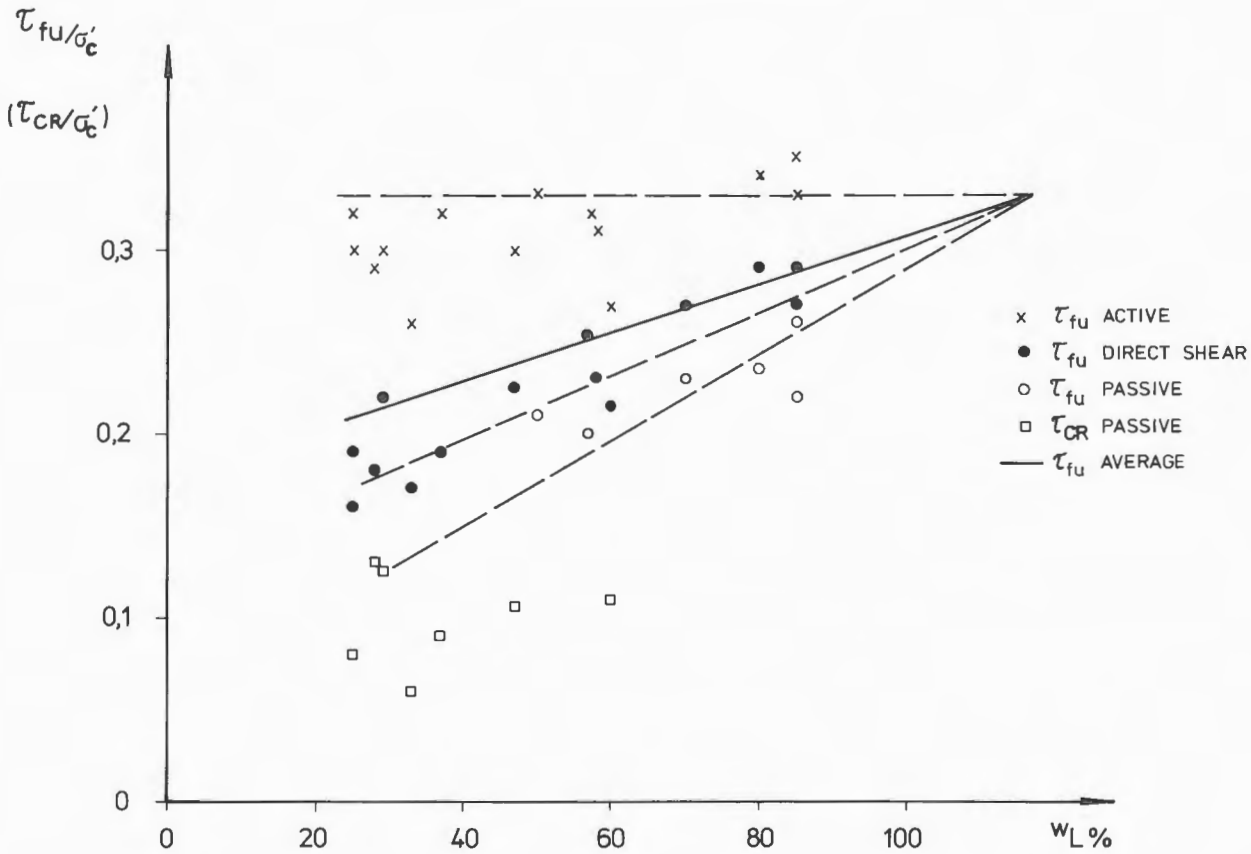


Fig. 17. Undrained shear strength in Scandinavian inorganic clays as a function of the preconsolidation pressure and the liquid limit, τ_{CR}, corresponds to the shear strength mobilized at a deformation corresponding to deformation at failure in active tests. Therefore the strength of failure $\tau_{PASSIVE}$ is somewhat higher than τ_{CR}. Data mainly from Larsson 1980.

Correlations between empirical relations and full scale failures in the field

The relevance for undrained shear strength determined by qualified laboratory investigation has been continuously shown in the investigations primarily made by the Norwegian Geotechnical Institute (NGI). NGI has developed the ADP-analysis (active-direct shear-passive) and this analysis is used in all qualified investigations, Aas 1976. Normally, no correlation with the preconsolidation pressure is made.

Corresponding analysis has also been developed at the Massachusetts Institute of Technology (MIT) but here all shear strengths are normalized according to the SHANSEP procedure (Ladd & Foott, 1974).

A compilation of full scale failure reported in literature together with information on plasticity, mobilized shear strength and preconsolidation pressure was made in 1980 (Larsson, 1980). Fig. 18.

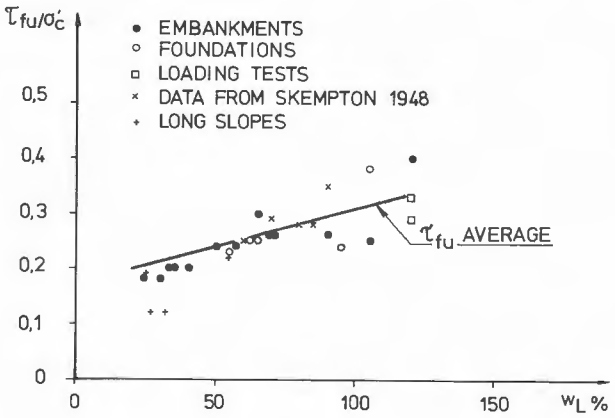


Fig. 18. Mobilized undrained shear strength at failure as a function of the liquid limit (Larsson, 1980).

The compilation includes failures of road embankments, failures under silos and oil storage tanks, results from test embankments and other load tests and some slope failures. Fig. 18 shows that the results spread somewhat but apart from some landslides in low plastic sensitive clay the empirical value for the average shear strength from labratory tests correlated well to the strength that could be mobilized in the field.

Correlations between empirical relations and shear strength values obtained by vane shear tests and fall cone tests

Direct shear tests usually give shear strengths directly compatible to strength values from vane shear tests and fall cone tests corrected according to this recommendation. This is as expected if the empirical relation for shear strength and Hanbo's relation $\tau_V = \sigma'_c \cdot 0.45\ w_L$ is valid. Fig. 19.

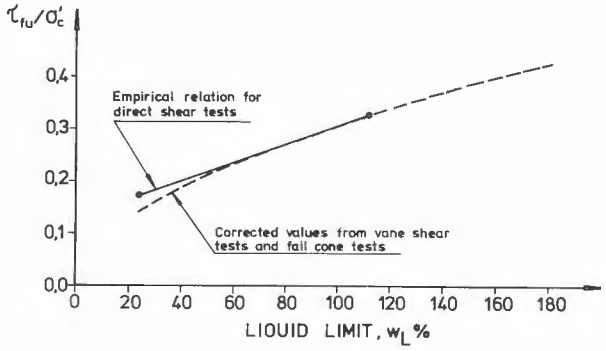


Fig. 19. Correlations between empirical shear strength at direct shear and corrected vane shear and fall cone values calculated by Hansbo's relation $\tau_V = \sigma'_c \cdot 0.45\ w_L$ and new recommended correction.

This means that compared to the corrected strength determined by vane shear tests and fall cone tests, active triaxial tests normally give higher shear strengths and passive triaxial tests normally give lower undrained shear strengths. Averages of the undrained shear strength from active and passive triaxial tests and direct shear tests are somewhat higher than the corrected strength from vane shear tests and fall cone tests. The difference increases with decreasing plasticity.

Fig. 20 shows correction factors for strength values measured by vane shear tests and fall cone tests, which would normally have to be used to obtain a complete correspondence both to results from direct shear tests and also to averages of active and passive triaxial tests and direct shear tests.

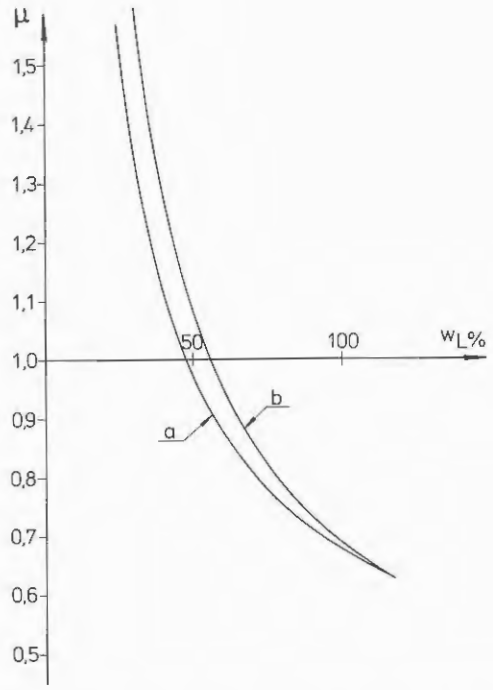


Fig. 20. Correction factors for strength values determined by vane shear tests and fall cone tests required to correspond to
a. direct shear tests
b. averages from active and passive triaxial tests and direct shear tests.

Compilation of results and comparison with the new recommended correction

The results from all the above mentioned investigations since 1969 are fairly unanimous. Different correction factors suggested or indirectly derived are compiled in Fig. 21.

The correction factor $\mu = 1.45/(1+w_L)$ suggested by Helenelund in 1977, gives a relatively good correspondence to all other results. However, it gives a flatter curve than the other relations in Fig. 21 and therefore a somewhat low correction factor for very low plastic clays, a somewhat high correction factor for medium plastic clays and a somewhat low correction for very high plastic organic soils. Andréasson's suggestion from 1974 agrees well with other results for low and medium plastic soils but gives relatively high correction factors for high plastic and organic soils. As mentioned earlier, no Scandinavian very high plastic clays or organic soils were included in the basic data for Andréasson's suggestion.

Swedish experience is that in soils with high liquid limit considerably lower correction factors must be used than those in Andréasson's suggestion. According to the SGI recommendations from 1969 a correction factor of 0.5 should be used for all soils with a liquid limit over 180% and according to recommendations from the Swedish Road Administation correction should be made for organic content according to $\mu = 0.8/w_L$. The Swedish National Road Administration also uses 0.5 as the lowest correction factor but on the other hand it considers that a higher calculated factor of safety against failure is required in very high plastic soils.

The numerical base for correction factors for Scandinavian organic soils with very high liquid limits is, however, very deficient and is mainly limited to the correlations with qualified laboratory investigations reported by Slunga 1983 and findings from current research at SGI.

The mathematical expression for the correction factor best corresponding to the correction factors which have emerged from different investigations is shown in Fig. 22.

$$\mu = \left(\frac{0{,}43}{w_L}\right)^{0{,}45}$$

This relation gives an upper limit of $\mu_{max} \approx 1.3$ when applied to Scandinavian clays and organic soils. Clays with a lower liquid limit than about 24% hardly occur in this region. Owing to the great sensitivity of the correction factor for small changes in the liquid limit and the great spread in shear strength values often found in low plastic soils, no correction factors higher than 1.2 should be used without support from complementary investigations.

Norwegian experience (Aas et al, 1984) and the curve for average shear strength from Larsson (1977, 1980) indicate that for stability calculations of embankments on horizontal ground the strength could be increased further for very low plastic clays. On the other hand, anisotropy is neither in Sweden nor abroad determined from this type of strength test but a value intended to be used generally is produced. It can hardly be regarded as acceptable to use a test procedure where the obtained strength value should be increased by more than 20% before it is used. In these extreme cases it can be observed that neither vane shear tests nor cone tests work sufficiently well and that if higher strengths are to be utilized other test procedures such as direct shear tests and triaxial tests should be used.

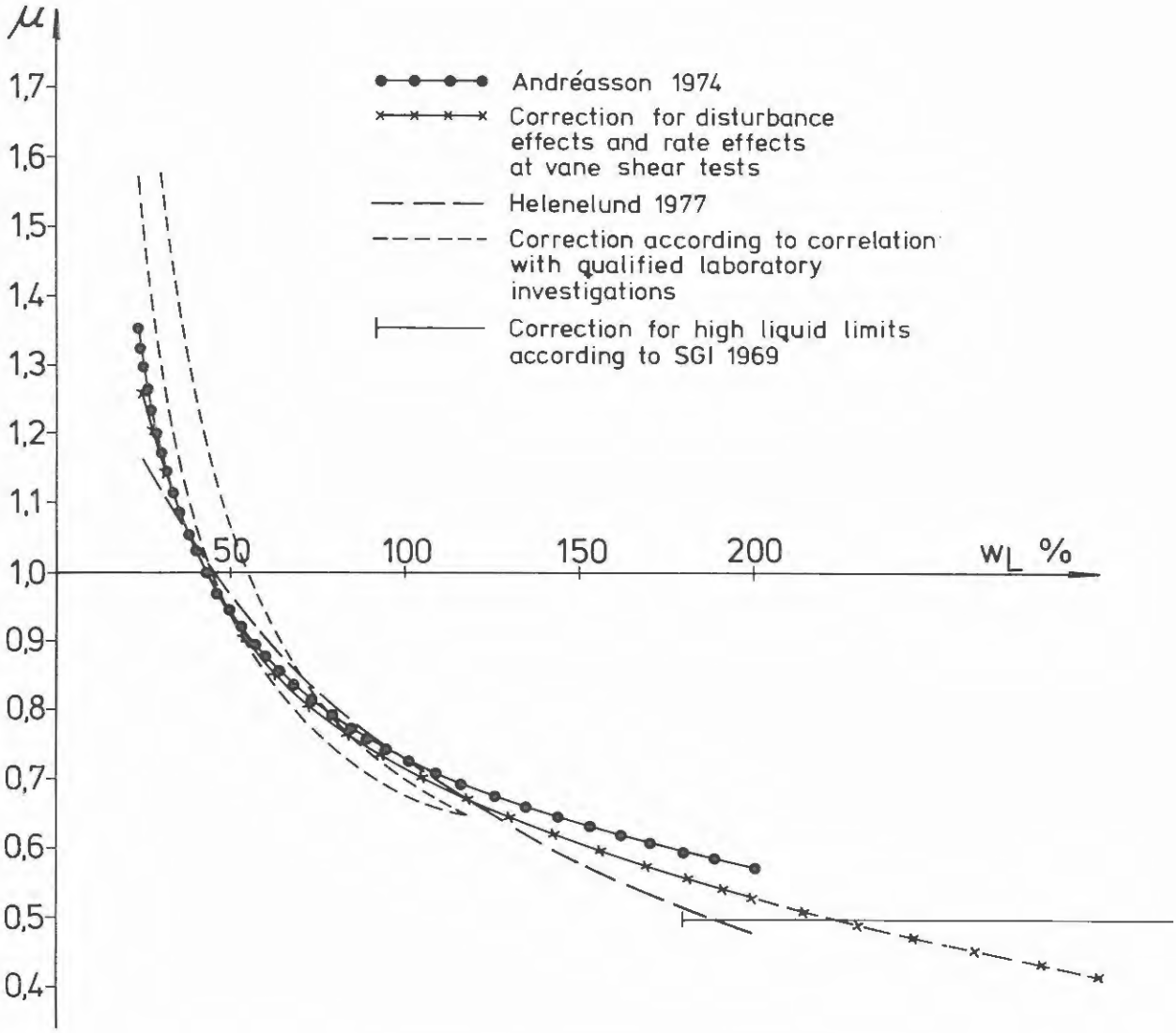


Fig. 21. Correlations between different correction factors for strength values determined by vane shear tests and fall cone tests.

The following is therefore recommended as a general correction factor for vane shear test values and cone test values **applied to averages** of the strength values obtained:

$$\mu = \left(\frac{0{,}43}{w_L}\right)^{0{,}45} \geq 0{,}5$$

with the reservation that correction factors higher than 1.2 should not be used without complementary investigations.

As was concluded from correlations between Bjerrum's correction factors and SGI's recommendation from 1969 (Andréasson 1974) this new procedure does not imply any greater difference in evaluated undrained shear strength compared with the earlier procedure with cautiously selected strength values and SGI's correction from 1969. The corrected strength for very low plastic clays becomes, however, somewhat higher according to the new suggestion.

The intention with the new procedure is primarily an adaption to a different and more uniform evaluation of the measured data.

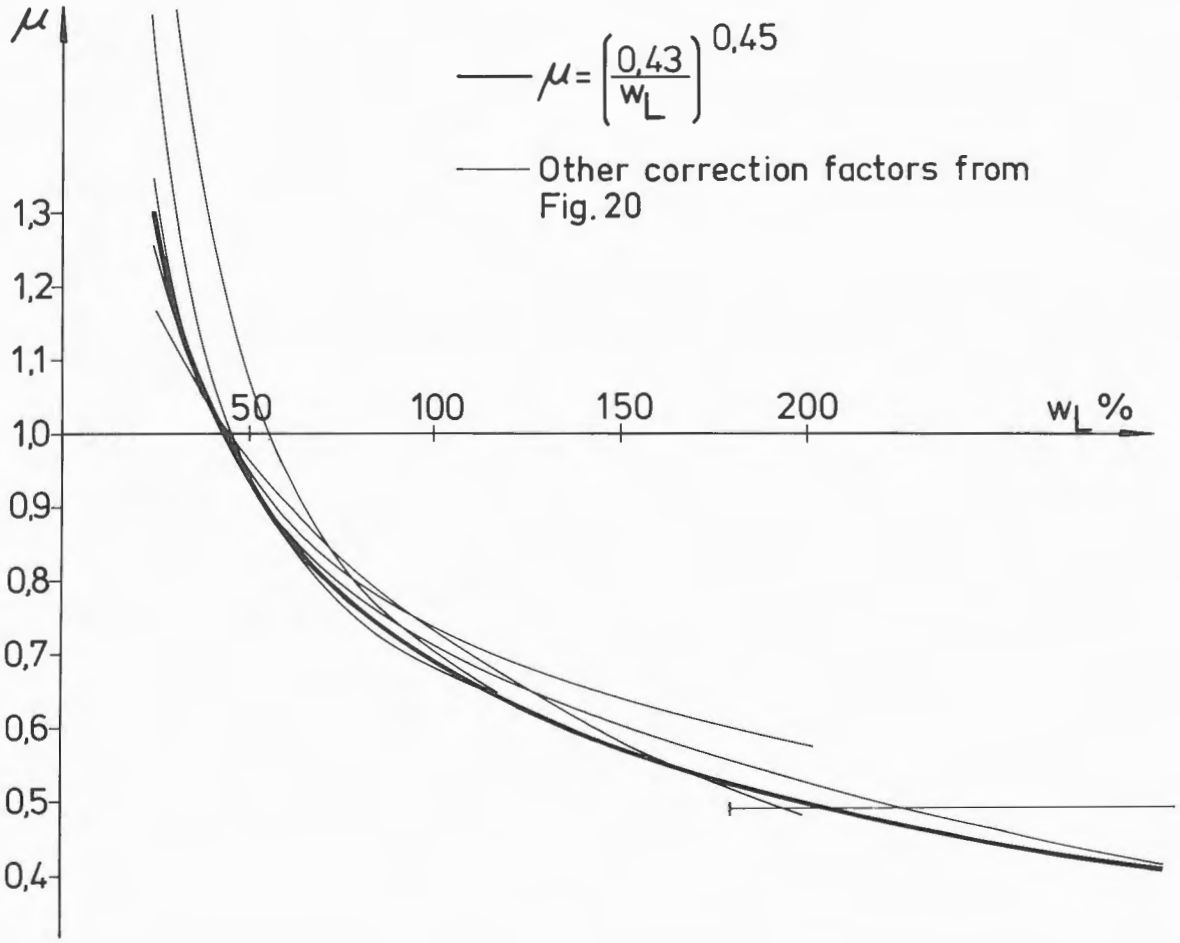


Fig. 22. Correlations between the relation $\mu = \left(\frac{0.43}{w_L}\right)^{0.45}$ *and other suggested correction factors.*

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