ROYAL SWEDISH
GEOTECHNICAL INSTITUTE
PROCEEDINGS
No. 14

# A NEW APPROACH TO THE DETERMINATION OF THE SHEAR STRENGTH OF CLAY BY THE FALL-CONE TEST

*By*

SVEN HANSBO

STOCKHOLM 1957

ROYAL SWEDISH
GEOTECHNICAL INSTITUTE
PROCEEDINGS
No. 14

# A NEW APPROACH TO THE DETERMINATION OF THE SHEAR STRENGTH OF CLAY BY THE FALL-CONE TEST

*By*

SVEN HANSBO

STOCKHOLM 1957

Ivar Hæggströms Boktryckeri AB
Stockholm 1957

# Contents

# Preface

This investigation was carried out by Mr Sven Hansbo at the Research Department of the Royal Swedish Geotechnical Institute at the suggestion of Mr Justus Osterman, Director of the Institute.

Mr Osterman has given the author several ideas in connection with the experimental investigation and the theoretical treatment in this report. The investigation of the cone motion was also suggested by Mr Osterman and was carried out experimentally at the Research Institute of National Defence (FOA) by Mr Jörgen Lexander. The laboratory investigations on the clay were carried out at the Consulting Department of the Geotechnical Institute under the supervision of Mr Rudolf Karlsson.

Of the two samplers mentioned in the report, Sampler SGI IV was designed by the Geotechnical Section of the Swedish Board of Roads and Waterways and Sampler SGI VI by the Mechanical Department of the Royal Swedish Geotechnical Institute.

The report was prepared by Mr Hansbo and the text was reviewed by Mr H. P. Vaswani and Mr J. N. Hutchinson.

Stockholm, March, 1957

ROYAL SWEDISH GEOTECHNICAL INSTITUTE

# 1. Synopsis

This paper presents a new approach to the interpretation of the fall-cone test.

The region of failure created around the cone when dropped into the clay is studied both theoretically and experimentally. A theoretical and experimental investigation of the cone motion has also been carried out.

A relation is established between the depth of cone penetration $h$ and the undrained shear strength $\tau_f$ of the clay. Thus, with sufficient accuracy we may write

$$\tau_f = KQ/h^2$$

where $Q$ is the weight of the cone and $K$ is a constant whose magnitude depends upon the cone angle $\beta$. For "undisturbed" clay, $K$ depends also upon the type of sampler used.

The values of $\tau_f$ obtained from this formula are compared with the undrained shear strength values obtained from other laboratory tests and from the field vane test.

Tables are included giving the corresponding values of $\tau_f$ and $h$ for undisturbed clay taken with the ordinary piston sampler (Sampler SGI IV) in Table I and for remoulded clay in Table II.

# 2. Introduction

In Sweden the undrained shear strength of clay is usually investigated by means of the fall-cone test, often in combination with the unconfined compression test or the vane test. The fall-cone test was developed by the Geotechnical Commission of the Swedish State Railways between 1914 and 1922, and was conceived by John Olsson, Secretary of the Commission. Compared to other methods of investigation the fall-cone test is extremely simple and has therefore gained a wide use in Scandinavia.

The test is carried out as follows. A metal cone is placed vertically with its apex just in contact with the top surface of the clay sample, Fig. 1. The cone is then dropped freely into the clay and the depth of penetration measured.

To interpret the fall-cone test, the Geotechnical Commission made a thorough investigation of the effect of different cone weights and apex angles on the depth of cone penetration. The results of this investigation were published by the Commission in its final report (Stat. Järnv. Geot. Komm., 1922, p. 46). These have also been summarized and published in English by Lundström (Caldenius and Lundström, 1956, p. 30).

The Commission found it convenient to reduce the number of different cones to a minimum. Three different standard cones were chosen, *viz.* the 100 gm.—30° cone (weight = 100 grammes; apex angle = 30°), the 60 gm.—60° cone, and the 10 gm.—60° cone.

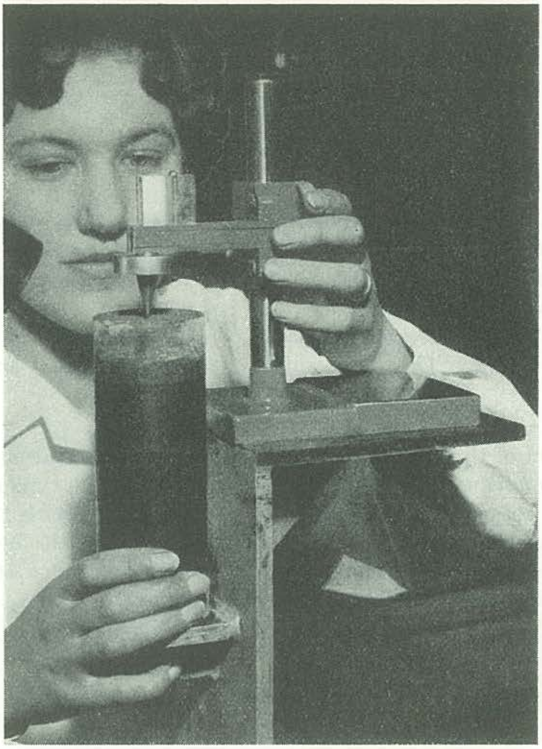


*Fig. 1. The fall-cone test apparatus in use.*

The strength of clay was defined by "the relative strength number" $H$. Clay for which the depth of penetration by the use of the 60 gm.—60° cone is 10 mm. was given an $H$-number equal to 10. The Commission assumed that proportionality exists between the resistance offered by different clays and the amount of external work done by the cone weights in causing an equal depth of penetration.[1]

According to this assumption and with the given definition, the $H$-number of any clay could be obtained in the following manner. The clay is tested with a 60° cone. The weight of the cone is varied until a depth of penetration of 10 mm. is obtained. Then this weight in grammes divided by 60 gm. is equal to one tenth of the $H$-number of the clay.

The relative strength number of remoulded clay is represented by $H_1$ and that of undisturbed clay by $H_3$. Originally, the symbol $H_2$ was used when dealing with partly disturbed clay, but this symbol has gone out of use. The $H$-

[1] It can only be said that the resistance seems to be proportional to the weight of the cone necessary to cause an equal depth of penetration.

quotient $H_3/H_1$ is evidently a measure of the sensitivity of the clay, though not in full accordance with the current definition of sensitivity.

Empirical formulas have been derived which make it possible to calculate the undrained shear strength $\tau_f$ of the clay on the basis of the strength number $H$. Thus for coarse-grained Norwegian clays, SKAVEN HAUG (1931), by comparisons with shear box tests[1], found

$$\tau_f = \frac{H_3}{32 + 0.073\, H_3} \text{ t/m}^2 \qquad (1)^2$$

For fat clay of the Gothenburg type, comparisons with punch tests and with unconfined compression tests (3 cm. cubes) (*cf.* for instance HULTIN, 1937, p. 87, and CALDENIUS, 1938, p. 141) showed that

$$\tau_f = \frac{H_3}{40 + 0.055\, H_3} \text{ t/m}^2 \qquad (2)$$

Usually, the mean value between these two formulas is used for normal Swedish clay, *i.e.*

$$\tau_f = \frac{H_3}{36 + 0.064\, H_3} \text{ t/m}^2 \qquad (3)$$

The values of $\tau_f$ obtained from these formulas are often different from the values obtained from other laboratory tests or from vane tests carried out *in situ*. This may be due to the fact that the punch and the shear box tests on which the formulas are based are just as hard to interpret as the cone test because of complex stress distributions.

Attempts have also been made to calculate the shear strength of the clay directly from the cone penetration. For the "push-cone" test, SKEMPTON and BISHOP (1950, p. 90) give the formula

$$\tau_f = \frac{P}{K\pi\left(h \cdot \tan\frac{\beta}{2}\right)^2} \qquad (4)$$

where $P$ is the force required to cause penetration,
$\beta$ is the cone angle,
$h$ is the depth of penetration,
$K$ is an empirical coefficient which, according to Skempton and Bishop, varies with water content for any given clay but also widely from one clay to another ($K = 3$ to $7$).

[1] The shear box used by Skaven Haug has very small length, and results in a stress-distribution that may be compared to that in the punch test.

[2] "t" is used to represent metric tonnes throughout this paper.

The formula gives the impression that $K$ is independent of the cone angle and may therefore be misleading.

The best way to find in the fall-cone test a theoretical relation between shear strength and cone penetration appears to be to study theoretically the motion of the cone when dropped into the clay and to verify it by experiment.

To estimate the resistance to the penetration of the cone it is necessary to have a knowledge of the factors influencing the deformation of the clay. The resistance to penetration of the cone depends not only on the modulus of shear but also on the viscosity of the clay. The influence of the viscosity depends upon several factors as for example the water content, the microscopic structure, and the rate of deformation of the clay. In the cone test the rate of deformation is different for different penetrations $h$ but is always very high, the fall time of the cone being only some hundredths of a second. The shear strength obtained will therefore be higher than in a slow shear test (*cf.* CASAGRANDE and SHANNON, 1948, p. 29 to 34). No doubt the penetration will also be affected by the sensitivity of the clay. The problem is a complex one and for the time being it seems impossible to find a strictly theoretical solution. The main object of this investigation has thus been to find an approximate solution suitable for engineering purposes.

## 3. Region Disturbed by the Cone

For the solution of the cone problem it is useful to study the effect on the clay of the cone penetration. The only visible effect is a heave in the immediate vicinity of the cone, which however is far less in volume than the hole in the clay made by the cone. The total disturbance of the clay must therefore be much more extensive than that observed at the clay surface. Any strictly theoretical attempt to determine the extent of this disturbance leads to intricate and difficult equations and therefore a more approximate treatment is employed, the conclusions being checked by experiments.

Let us consider the forces acting upon the surface of a cone element. These are shear stresses $\tau$ and normal stresses $\sigma$, Fig. 2. The stresses have different values at different depths $\zeta$ underneath the surface, their magnitude depending on the degree of deformation of the surrounding clay. Assuming as a first approximation that the stress variation along the cone surface corresponds to the stress-deformation curve, Fig. 3 (*cf.* HVORSLEV, 1937, p. 110), $\tau$ and $\sigma$ would vary as shown in Fig. 4. Now the extent of the region of failure depends upon $\tau$ and $\sigma$, and some conclusions about its shape may be drawn from the mathematical treatment of kindred problems as, for example, partial yielding in a thick-walled tube subjected to internal pressure, wedge indentation, *etc.* (*cf.* for instance NÁDAI, 1927; NÁDAI, 1931, p. 186 and p. 253; HILL, 1950, p. 106 and p. 215; HOFFMAN and SACHS, 1953, p. 80). For instance, in the case of a cylindrical thick-walled tube subjected to internal pressure $p$, the radius $r$ of the plastic region is

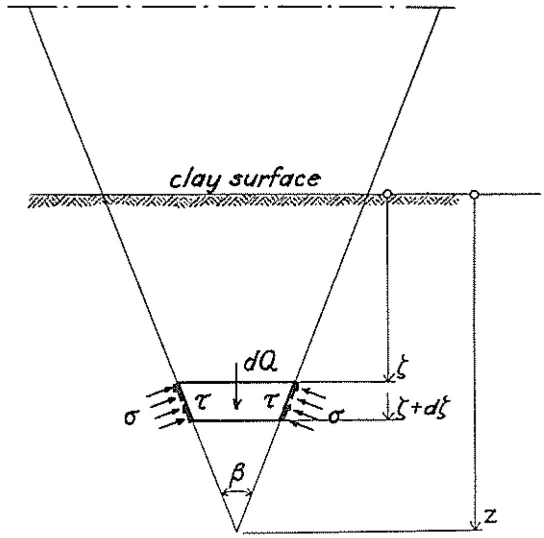



*Fig. 2. Forces acting upon a cone element during penetration.*

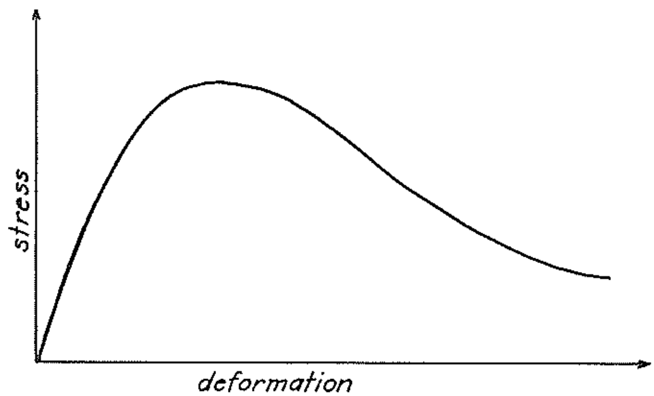



*Fig. 3. Stress-deformation curve for clay.*

$$r \approx a \cdot e^{\frac{1}{2}\left(\frac{p\sqrt{3}}{\sigma_0} - 1\right)} \qquad (5)$$

where $a$ is the inner radius of the tube, and
$\sigma_0$ is the yield stress according to von Mises yield criterion.

This formula gives an idea of the configuration of the plastic zone. Its width measured from the cone surface will reach a maximum (minimum) value at the level of maximum (minimum) stresses. With the depths of penetration common in practice ($z_2$ in Fig. 4) the stresses near the surface reach only a fraction of the failure stresses for undisturbed clay, $\tau_f$ and $\sigma_f$, so that the plastic state is

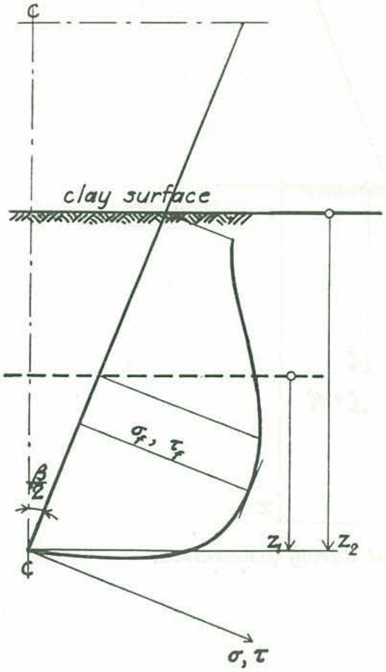



*Fig. 4. Assumed stress variation along the cone surface.*

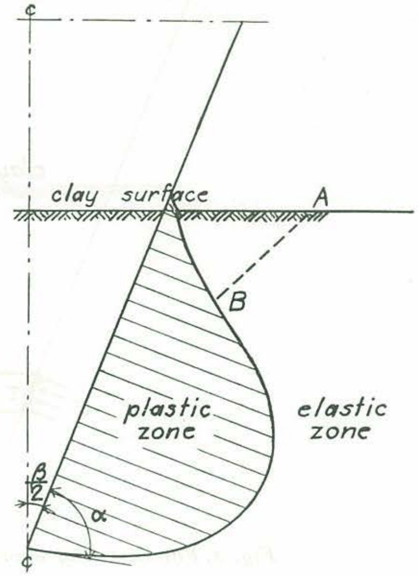



*Fig. 5. Assumed configuration of the plastic zone created around the cone during penetration.*

unlikely to occur at the clay surface. The deformation of the clay around the cone increases with increasing cone angle and consequently the tangent angle $\alpha$, shown in Fig. 5, will also increase with increasing cone angle.

There is, however, another effect with perhaps greater influence on the shape of the disturbed region. The clay volume forced aside by the cone although somewhat influenced by compression of contained gas bubbles and by dilatancy must be accomodated by displacement of the surrounding media. This happens in three ways. Firstly, an upward plastic flow of clay takes place along the cone surface and produces the heave previously remarked upon. Secondly, the clay surrounding the plastic region, being elastic, will be strained horizontally,[1] and, finally, the pressure increase might produce a slip on a surface such as AB, Fig. 5 (*cf.* Nádai, 1931, Figs. 322 to 324). The effect of the confinement of the plastic clay between the cone surface and the surrounding elastic region will be to change the stresses shown in Fig. 4 and thus alter the width of the plastic region. Due to boundary effects no increase of the plastic region will occur at the clay surface.

[1] Suitable precautions should be taken to prevent horizontal deformation of the vertical boundary of the sample. At the Institute this is done by retaining the sample in the brass cylinder in which it is taken.

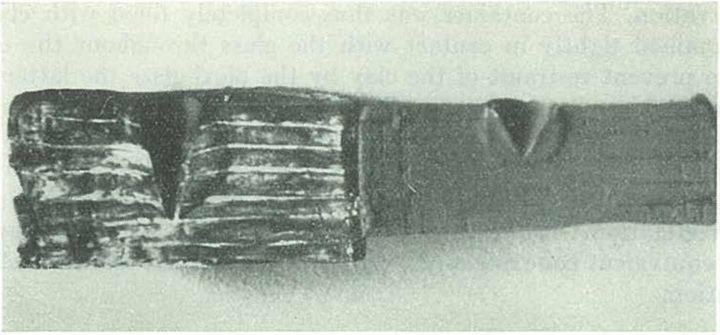


*Fig. 6. Configuration of the plastic zone created around the cone during penetration in experiment I.*

To investigate the reliability of this conception some experiments were carried out. An unconfined cylindrical clay sample was split diametrically into two equal parts. One half of the sample was then placed with the plane surface in contact with a plexi-glass sheet, and a half-cone was pressed into the clay against the glass. The failure pattern caused by the cone penetration was thus seen. These experiments differ from the fall-cone test but are considered to produce a plastic region of comparable shape. The results of the experiments are shown in Fig. 6.

It proved difficult to hold the clay sample tightly in contact with the glass during the run of the experiment, clay within the plastic region being squeezed out of the sample towards the glass. The observed shape of the plastic region was consequently considered doubtful.

New experiments were therefore made, in which a half-cylindrical container with the plane wall made of plexi-glass was pressed into the clay in the bottom

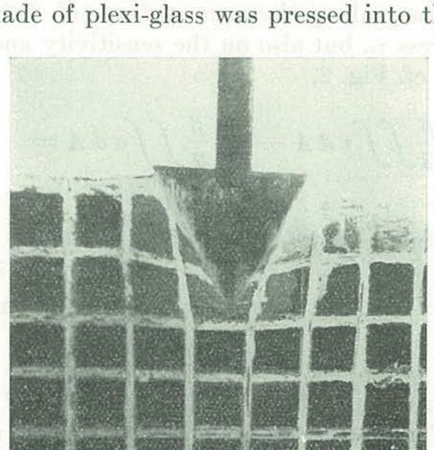


*Fig. 7. Configuration of the plastic zone created around the cone during penetration in experiment II.*

of an excavation. The container was thus completely filled with clay and the sample remained tightly in contact with the glass throughout the experiment. In order to prevent restraint of the clay by the plexi-glass the latter was lubricated. The experiment was then carried out in the same way as described above. A typical result of the experiment is shown in Fig. 7.

A comparison of Fig. 5 with Figs. 6 and 7 shows that there is sufficient agreement with experiment for the present theory to be used as a working basis for further investigation. From Figs. 6 and 7 it is also realized that the shapes of this region for equivalent cone angles are very nearly similar, regardless of the depth of penetration.

## 4. Study of the Cone Motion

The motion of a body of mass $m$, subjected to a force vector $\boldsymbol{P}$, is defined by

$$\boldsymbol{P} = m\boldsymbol{a} \tag{6}$$

where $\boldsymbol{a}$ is the acceleration vector of the body.

Using the engineering measurement system, the vertical motion of the cone may be written

$$P = \frac{Q}{g}\ddot{z} \tag{7}$$

where $P$ is the vertical resultant of the forces acting upon the cone,
$Q$ is the weight of the cone,
$g$ is the acceleration due to gravity, and
$z$ is the depth of penetration at a certain time $t$ $\left(\ddot{z} = \dfrac{d^2 z}{dt^2}\right)$.

Consider again the forces acting upon a cone element, Fig. 2. Evidently, the stresses $\tau$ and $\sigma$ will vary along the cone surface (*cf.* § 3) and will depend not only on the failure stress $\tau_f$, but also on the sensitivity and on the rate of shear. The exact expression, *cf.* Fig. 2,

$$P = Q - \cos\frac{\beta}{2}\iint \tau\, d\,A - \sin\frac{\beta}{2}\iint \sigma\, d\,A =$$

$$= Q - 2\,\pi\tan\frac{\beta}{2}\int_0^z \tau\,(z-\zeta)\,d\,\zeta - 2\,\pi\tan^2\frac{\beta}{2}\int_0^z \sigma\,(z-\zeta)\,d\,\zeta \tag{8}$$

is therefore difficult of solution and is replaced by the approximate expression

$$P = Q - Tz^2 \qquad (9)^{1}$$

where $T$ is a function mainly of the shear strength $\tau_f$ of the clay and the cone angle $\beta$ but is also influenced by the rate of deformation and by the sensitivity.

---

[1] For remoulded clay this expression for $P$ seems to be a better approximation than for undisturbed clay. The approximation is justified owing to the fact that the shape of the disturbed region is similar regardless of the depth of penetration.

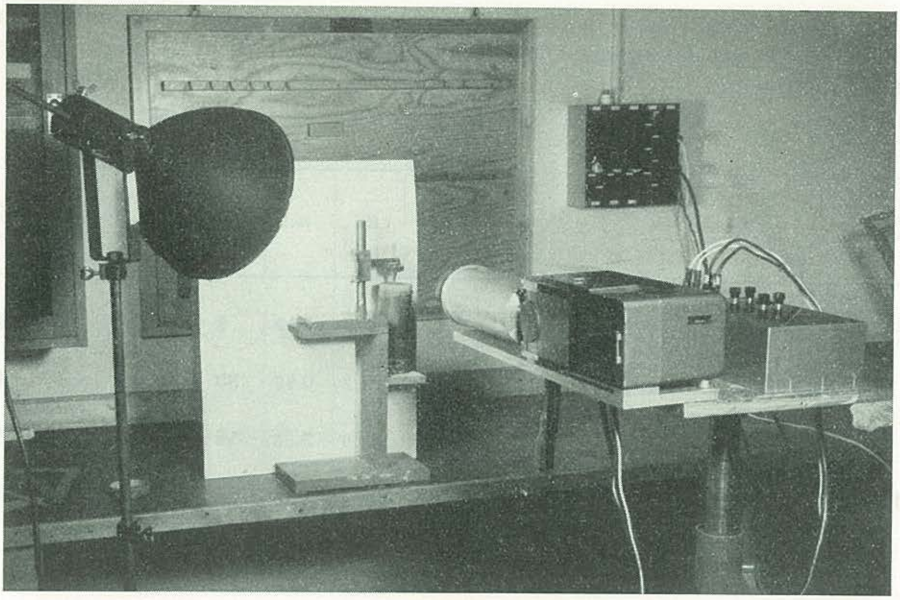


*Fig. 8. Arrangement for the experimental investigation of the cone motion during penetration.*

Eq. (7) can thus be rewritten

$$\ddot{z} + gTz^2/Q = g \tag{10}$$

whence

$$\dot{z} = \sqrt{C + 2\,gz - 2\,gTz^3/3\,Q} \tag{11}$$

The value of the constant of integration $C$ is obtained from the boundary condition $\dot{z} = 0$ at $z = 0$ and is found to be zero.

If the final value of the depth of penetration is $h$, we have, since $\dot{z} = 0$ at $z = h$,

$$T = 3\,Q/h^2 \tag{12}$$

Introducing Eq. (12) in Eqs. (11) and (10), we find

$$\dot{z} = \sqrt{2\,g\,z\left[1 - \left(\frac{z}{h}\right)^2\right]} \tag{13}$$

and

$$\ddot{z} = g\left[1 - 3\left(\frac{z}{h}\right)^2\right] \tag{14}$$

According to Eqs. (13) and (14) the maximum velocity will be reached at $z = h/\sqrt{3}$ and is $0.868\,\sqrt{gh}$.

To investigate the reliability of these equations, experiments were made in which the motion of the cone was photographed with a high-speed camera, Fig. 8. Ten different cone tests were photographed. Five different types of clay,

Table 1, were tested, first in "undisturbed" state and then in remoulded state. Sample V was too heterogeneous to give reliable values of strength number, water content, *etc.*

*Table 1*

| Sample No | Type of soil | Cone test[1] $h$ mm/$Q$ gm Undist. sample | Re-moulded sample | $H_3$ | $H_1$ | $H_3/H_1$ | Unit weight t/m³ | Water content $w$ % | Relative fineness[2] $F$ |
|---|---|---|---|---|---|---|---|---|---|
| I | Grey clay ................ | 8.0/100 | 10.0/60 | 79.0 | 10.00 | 8 | 1.59 | 65 | 65 |
| II | Grey clay ................ | 5.9/60 | 13.3/10 | 27.7 | 0.93 | 30 | 1.52 | 79 | 48 |
| III | Varved clay with seams of fine sand .............. | 11.0/100 | 16.5/60 | 40.0 | 3.35 | 12 | 1.66 | 63 | 51 |
| IV | Nekron mud *(gyttja)* ..... | 9.6/100 | 10.9/60 | 53.4 | 8.40 | 6 | 1.18 | 220 | 204 |
| V | Dark grey peaty fine sand and silt ................ | 5.4/100 | 19.0/100 | 168.0 | — | — | 1.64 | 51 | — |

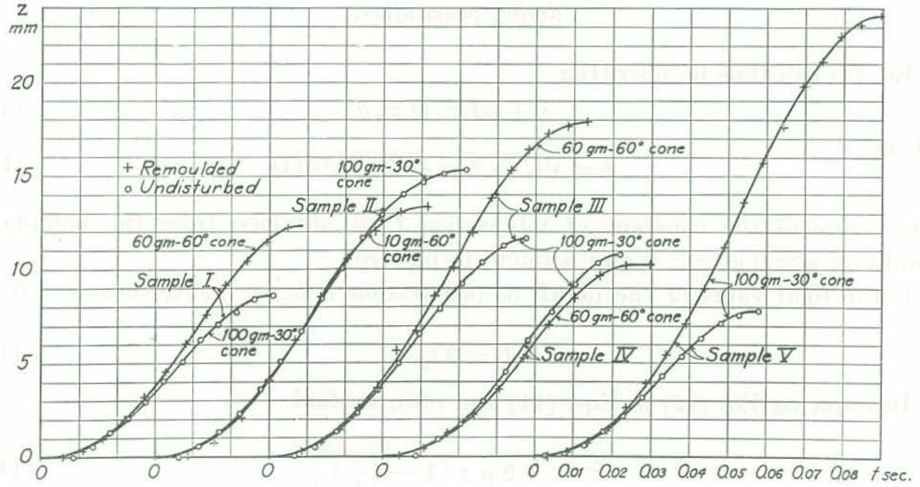



*Fig. 9. Motion of the cone in the different clay samples according to Table 1.*

[1] The values given in Table 1 were obtained in the course of routine testing at the Institute. The photographic investigation however, was made at the Research Institute of National Defence where, due to disturbance in transport and to heterogeneity of the clay samples, the cone penetrations were different from the values given in Table 1. Thus, for Sample I, $h/Q$ was 8.6/100 and 12.4/60, for Sample II, 15.3/100 and 13.3/10, for Sample III, 11.7/100 and 17.8/60, for Sample IV, 10.8/100 and 10.3/60, and for Sample V, 7.8/100 and 23.6/100, respectively.

[2] The relative fineness ("finlekstal") $F$ is approximately equal to the liquid limit $w_L$ except for quick clays where $F$ lies in between the liquid limit $w_L$ and the plastic limit $w_P$ (*cf.* for instance HVORSLEV, 1937. p. 46).

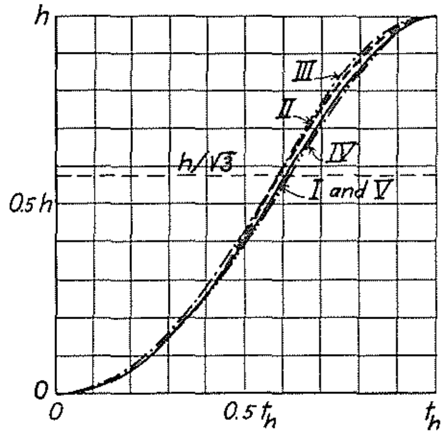



*Fig. 10. Motion of the cone in remoulded clay (Samples I to V).*

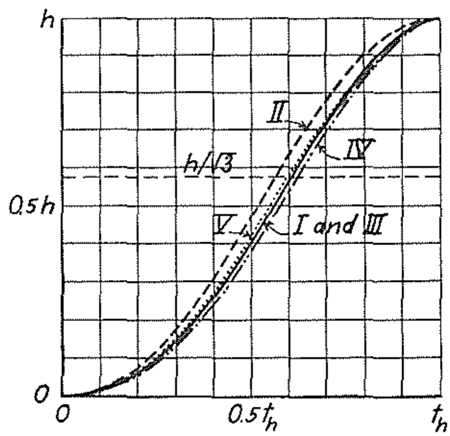



*Fig. 11. Motion of the cone in "undisturbed" clay (Samples I to V).*

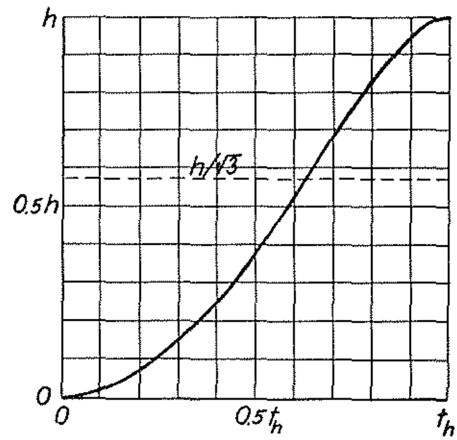



*Fig. 12. Theoretical motion of the cone during penetration.*

Fig. 9 shows the motion of the cone in the different clay samples. These curves are redrawn in Figs. 10 and 11. Here the total depth of penetration $h$ and the total fall-time $t_h$ have been scaled down to constant values equal for all the different clays. The theoretical curve, obtained by graphical integration of Eq. (13), *viz.*

$$t = \int_0^z \frac{dz}{\sqrt{2gz[1-(z/h)^2]}} \tag{15}$$

is plotted in Fig. 12.

The agreement between the theoretical and experimental curves is good, except for the Sample II in the "undisturbed" state, and the approximations made appear thus to be reasonable.

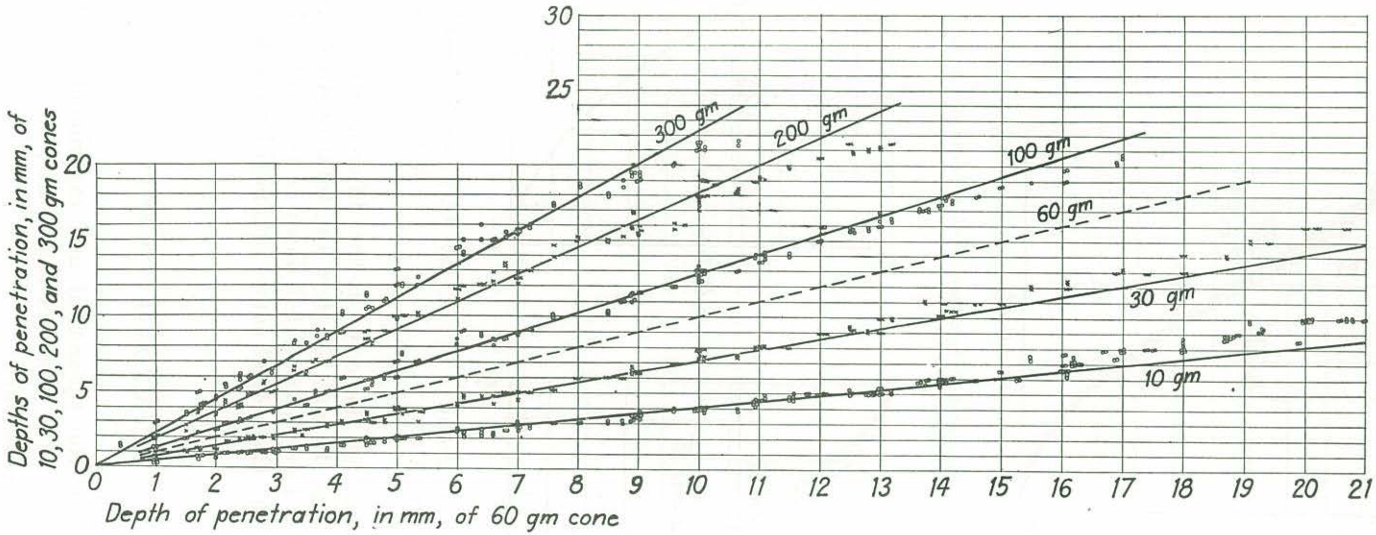



*Fig. 13. Results of cone tests carried out by the Geotechnical Commission of the Swedish State Railways. The depths of penetration of 10 gm, 30 gm, 100 gm, 200 gm, and 300 gm cones represented as functions of the depth of penetration of the 60 gm cone according to Eq. (16). Cone angle = 60°.*

## 5. Relation between Shear Strength and Cone Penetration

The investigation in § 4 shows that it is possible to find an approximate relation, which is satisfactory for engineering purposes, between the undrained shear strength $\tau_f$ and the depth of penetration $h$. Thus, assuming $T = 3\tau_f/K$, Eq. (12) becomes

$$\tau_f = KQ/h^2 \tag{16}$$

where $K$ depends mainly on the cone angle $\beta$ but is also influenced by the rate of shear and by the sensitivity.

Obviously, Eq. (16) would be of little interest if $K$ varied widely for one and the same cone angle. In such a case no practical advantage would be gained over the previous interpretation of the fall-cone test. A comparison between Eq. (16) and the results of the fall-cone tests given in the final report of the Geotechnical Commission shows, however, that $K$ is practically constant for each particular value of $\beta$, *cf.* Fig. 13, which is also confirmed by the following investigation. Thus, the influence of normal variations in the rate of shear and the sensitivity appears to be small. Further, it should be borne in mind that other factors, such as for example in the case of "undisturbed" clay, disturbances caused during sampling and transport, may have a much greater influence on $K$ than those due to varying sensitivity and rate of shear in the actual test.

The clays investigated here have been selected to represent most of the different types of Swedish clays, which are normally marine clays of the illite group with sensitivity of about 10. The shear strength $\tau_f$ of most of these clays,

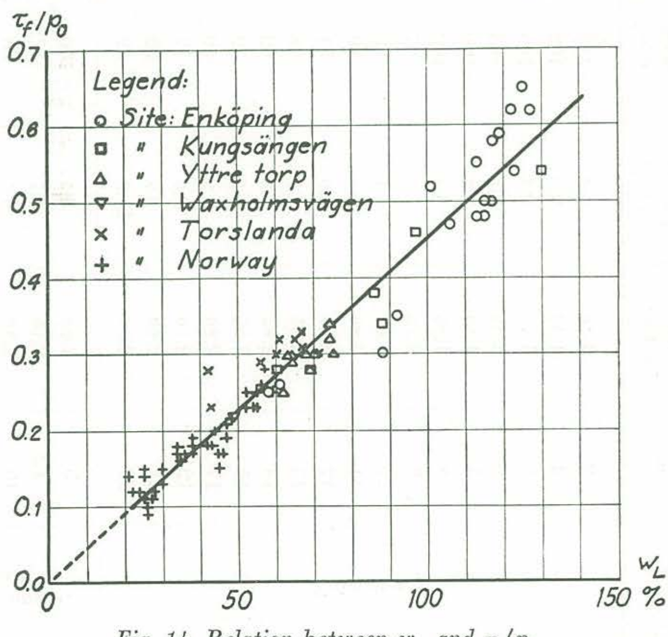



*Fig. 14. Relation between $w_L$ and $\tau_f/p_0$*

*Table 2.*

| Site | Depth m | Field vane test $\tau_f$ t/m² | Overburden pressure $p_0$ t/m² | Natural water content $w$ % | Liquid limit $w_L$ % | Plastic limit $w_p$ % | Unit weight $\gamma$ t/m³ | Sensitivity (vane test) | Organic matter content % | $\tau_f/p_0$ |
|---|---|---|---|---|---|---|---|---|---|---|
| Enköping | 3 | 1.10 | 1.7 | 113 | 125 | 32 | 1.37 | 11 | 1.5 | 0.65 |
| | 4 | 1.10 | 2.0 | 102 | 113 | 31 | 1.41 | 10 | 1.2 | 0.55 |
| | 5 | 1.15 | 2.3 | 105 | 115 | 31 | 1.40 | 11 | 1.2 | 0.50 |
| | 6 | 1.30 | 2.6 | 103 | 117 | 31 | 1.42 | 11 | 1.3 | 0.50 |
| | 7 | 1.50 | 3.1 | 100 | 113 | 31 | 1.43 | 12 | 1.3 | 0.48 |
| | 8 | 1.85 | 3.4 | 100 | 123 | 32 | 1.44 | 8 | 1.4 | 0.54 |
| | 9 | 2.15 | 3.7 | 97 | 119 | 33 | 1.47 | 11 | 1.4 | 0.58 |
| | 10 | 2.55 | 4.1 | 98 | 127 | 39 | 1.46 | 9 | 2.2 | 0.62 |
| | 11 | 2.80 | 4.5 | 90 | 122 | 38 | 1.48 | 9 | 1.9 | 0.62 |
| | 12 | 2.85 | 4.9 | 81 | 117 | 34 | 1.51 | 9 | 1.5 | 0.58 |
| | 13 | 2.85 | 5.5 | 76 | 101 | 32 | 1.57 | 11 | 1.3 | 0.52 |
| | 14 | 2.85 | 6.0 | 80 | 115 | 33 | 1.52 | 12 | 1.0 | 0.48 |
| | 15 | 3.05 | 6.5 | 72 | 106 | 29 | 1.58 | 10 | 0.9 | 0.47 |
| | 17 | 2.80 | 7.9 | 69 | 92 | 27 | 1.60 | 8 | 0.6 | 0.35 |
| | 19 | 2.70 | 9.0 | 77 | 88 | 27 | 1.58 | 11 | 0.4 | 0.30 |
| | 21 | 2.60 | 9.9 | 53 | 61 | 22 | 1.70 | 9 | 0.4 | 0.26 |
| | 23 | 2.80 | 11.0 | 49 | 58 | 22 | 1.78 | 9 | 0.3 | 0.25 |
| Kungsängen, Uppsala | 5 | 1.90 | 3.5 | 100 | 130 | 48 | 1.46 | 9 | 2.6 | 0.54 |
| | 7.5 | 2.15 | 4.7 | 77 | 97 | 37 | 1.51 | 10 | 1.5 | 0.46 |
| | 10 | 2.30 | 6.1 | 68 | 86 | 31 | 1.58 | 12 | 1.1 | 0.38 |
| | 12.5 | 2.60 | 7.6 | 68 | 88 | 29 | 1.59 | 10 | 0.9 | 0.34 |
| | 15 | 2.55 | 9.1 | 54 | 60 | 23 | 1.64 | 11 | 0.6 | 0.28 |
| | 17.5 | 3.00 | 10.7 | 65 | 69 | 26 | 1.62 | 13 | 0.5 | 0.28 |

*Table 2 (Continued)*

| Site | Depth m | Field vane test $\tau_f$ t/m² | Overburden pressure $p_0$ t/m² | Natural water content $w$ % | Liquid limit $w_L$ % | Plastic limit $w_P$ % | Unit weight $\gamma$ t/m³ | Sensitivity (vane test) | Organic matter content % | $\tau_f/p_0$ |
|---|---|---|---|---|---|---|---|---|---|---|
| Yttre torp, Göta älv | 3.5 | 2.10 | 8.3 | 69 | 62 | 27 | 1.59 | — | — | 0.25 |
| | 4.5 | 2.55 | 8.6 | 68 | 64 | 28 | 1.61 | — | — | 0.30 |
| | 5.5 | 2.70 | 9.4 | 72 | 64 | 27 | 1.59 | — | — | 0.29 |
| | 6.5 | 2.95 | 9.8 | 66 | 63 | 27 | 1.59 | — | — | 0.30 |
| | 7.5 | 3.10 | 10.4 | 73 | 68 | 28 | 1.56 | — | — | 0.30 |
| | 8.5 | 3.15 | 11.1 | 72 | 69 | 29 | 1.57 | — | — | 0.28 |
| | 9.5 | 3.90 | 11.6 | 78 | 74 | 30 | 1.53 | — | — | 0.34 |
| | 11.5 | 4.05 | 12.7 | 78 | 74 | 31 | 1.54 | — | — | 0.32 |
| | 12.5 | 4.00 | 13.3 | 75 | 70 | 30 | 1.55 | — | — | 0.30 |
| | 13.5 | 4.05 | 13.8 | 79 | 75 | 30 | 1.54 | — | — | 0.30 |
| Waxholms-vägen | 3.1 | 0.65 | 3.0 | 77 | 49 | 22 | — | — | — | 0.22 |
| Torslanda, Göteborg | 5 | 2.35 | 7.8 | 73 | 71 | 26 | 1.57 | — | — | 0.30 |
| | 6 | 2.50 | 8.1 | 67 | 67 | 24 | 1.60 | — | — | 0.31 |
| | 7 | 2.80 | 8.4 | 65 | 67 | 25 | 1.62 | — | — | 0.33 |
| | 8 | 2.80 | 8.8 | 62 | 65 | 25 | 1.64 | — | — | 0.32 |
| | 9 | 2.95 | 9.2 | 60 | 61 | 25 | 1.66 | — | — | 0.32 |
| | 10 | 2.90 | 9.6 | 59 | 60 | 24 | 1.66 | — | — | 0.30 |
| | 11 | 2.85 | 10.0 | 54 | 56 | 23 | 1.67 | — | — | 0.29 |
| | 12 | 2.90 | 10.4 | 52 | 42 | 21 | 1.68 | — | — | 0.28 |
| | 13 | 2.50 | 10.9 | 48 | 43 | 22 | 1.75 | — | — | 0.23 |

where not influenced by desiccation or preconsolidation, may be determined with reasonable accuracy from the relation $\tau_f = 0.45\, w_L p_0$ where $p_0$ is the effective vertical overburden pressure and $w_L$ is the liquid limit, see Table 2 and Fig. 14.[1]

Unfortunately it has not been possible to give such a representative chart of the Swedish clays in the diagram, Fig. 14, as could be desired, as pore water measurements and oedometer tests have seldom been carried out here in connection with field vane tests. Moreover, oedometer tests are difficult to interpret (*cf.* CASAGRANDE, 1936, p. 60), and values of $p_0$ obtained from them are consequently uncertain. Pore water measurements may also be misleading. Thus in Fig. 14 are shown only four of the clays which have been utilized for the determination of $K$ in Eq. (16). Some Norwegian clays with sensitivities from 3 to 500 are also shown (BJERRUM, 1954, Table 4).

The shear strength of normal Swedish clays also seems to vary linearly with $I_P p_0$ ($I_P$ is the plasticity index), but this relation has proved less accurate than that stated above.

*Determination of K for "Undisturbed" Clay.*

The most reliable method in use for investigating the undrained shear strength of "undisturbed" clay is the field vane test (CADLING and ODENSTAD, 1950). As is well known, in this method the soil is investigated *in situ* and the clay is only slightly disturbed by the advance of the vane while disturbances caused by sampling and transport are eliminated. For these reasons it was considered suitable to determine the coefficient $K$ from values of $\tau_f$ obtained by the field vane test. By doing this it was possible to include in $K$ a correction for the average disturbance of a particular sampler. This disturbance varies widely for different types of samplers (JAKOBSON, 1954) which is a great disadvantage in the interpretation of laboratory shear tests.

Details of the soils investigated are set out in Table 3. Plotting values of shear strength $\tau_f$ given in Table 3 against the corresponding depths of penetration $h$ on double-logarithmic paper, Plate I, we find good agreement with Eq. (16), provided the constant $K$ is properly chosen. Plate I gives the values of $K$ for the 100 gm.—30° cone when using the ordinary piston sampler SGI IV, Fig. 15, and the pneumatic piston sampler SGI VI, Fig. 16. Thus for the ordinary piston sampler, $K \approx 1.0$, and for the pneumatic piston sampler, $K \approx 0.8$.

In order to determine the values of $K$ for the 60° cones, clays were tested with both the 100 gm.—30° cone and the 60 gm.—60° cone in the course of routine work in the Consulting Department of the Institute. A linear relation was found between the depths of penetration for the two cones as shown in Fig. 17. Unfortunately, it was not possible to use the corresponding values of $h$ given for these two cones in the $H$-tables (Stat. Järnv. Geot. Komm., 1922, p. 51) as these values do not seem to hold for "undisturbed" clay.

[1] This relation does not always hold. It is possible that its applicability is confined to a particular group of clays and stress distribution.

## *Table 3.*

| Site | Depth m | Field vane test $\tau_f$ t/m² | Fall-cone test 100 gm—30° $h$ mm Sampler IV | Fall-cone test 100 gm—30° $h$ mm Sampler VI | Natural water content $w$ % | Liquid limit $w_L$ % | Plastic limit $w_P$ % | Unit weight $\gamma$ t/m³ | Type of soil |
|---|---|---|---|---|---|---|---|---|---|
| Enköping | 1.2 | 1.50 | 7.8 | — | — | — | — | — | grey muddy clay |
| | 2 | 1.25 | 9.6 | 8.6 | 114 | 131 | 36 | 1.37 | » » » |
| | 3 | 1.10 | 9.9 | 8.7 | 113 | 125 | 32 | 1.37 | dark-grey muddy clay |
| | 4 | 1.10 | 9.9 | 8.6 | 102 | 113 | 31 | 1.41 | dark-grey clay |
| | 5 | 1.15 | 9.2 | 8.6 | 105 | 115 | 31 | 1.40 | black clay |
| | 6 | 1.30 | 8.7 | 7.9 | 103 | 117 | 31 | 1.42 | black clay with thin organic layers |
| | 7 | 1.50 | 8.1 | 7.6 | 100 | 113 | 31 | 1.43 | black clay |
| | 8 | 1.85 | 7.3 | 6.6 | 100 | 123 | 32 | 1.44 | dark-grey clay |
| | 9 | 2.15 | 6.8 | 6.2 | 97 | 119 | 33 | 1.47 | » » |
| | 10 | 2.55 | 6.5 | 5.2 | 98 | 127 | 39 | 1.46 | » » |
| | 11 | 2.80 | 6.0 | 5.2 | 90 | 122 | 38 | 1.48 | » » |
| | 12 | 2.85 | 6.0 | 5.0 | 81 | 117 | 34 | 1.51 | dark-grey clay with thin organic layers |
| | 13 | 2.85 | 5.8 | 5.1 | 76 | 101 | 32 | 1.57 | dark-grey clay |
| | 14 | 2.85 | 5.8 | 5.1 | 80 | 115 | 33 | 1.52 | dark clay |
| | 15 | 3.05 | 5.7 | 4.9 | 72 | 106 | 29 | 1.58 | dark-grey clay |
| | 17 | 2.80 | 6.2 | 5.4 | 69 | 92 | 27 | 1.60 | grey clay with stains of iron sulphide |
| | 19 | 2.70 | 7.0 | 6.4 | 77 | 88 | 27 | 1.58 | disturbed stratification of grey varved clay with layers of sand |
| | 21 | 2.60 | 6.0 | 6.2 | 53 | 61 | 22 | 1.70 | |
| | 23 | 2.80 | 5.4 | 6.3 | 49 | 58 | 22 | 1.78 | |
| | 25 | 3.40 | 6.3 | 6.1 | 51 | 62 | 24 | 1.77 | |
| | 27 | 3.80 | 5.7 | 5.2 | 53 | 62 | 21 | 1.72 | brown-grey varved clay |
| | 29 | 4.05 | 5.3 | 4.9 | 43 | 50 | 19 | 1.80 | grey varved clay with thin layers of sand |
| Kungsängen, Uppsala | 5 | 1.90 | 6.2 | 6.3 | 100 | 130 | 48 | 1.46 | dark-grey muddy clay with shells |
| | 7.5 | 2.15 | 6.1 | 6.0 | 77 | 97 | 37 | 1.51 | dark-grey clay |
| | 10 | 2.30 | 6.1 | 5.6 | 68 | 86 | 31 | 1.58 | » » |
| | 12.5 | 2.60 | 5.9 | 5.1 | 68 | 88 | 29 | 1.59 | » » |
| | 15 | 2.55 | 6.0 | 5.5 | 54 | 60 | 23 | 1.64 | grey clay with stains of iron sulphide |
| | 17.5 | 3.00 | 5.1 | 5.0 | 65 | 69 | 26 | 1.62 | — » — |

*Tabe 3 (Continued)*

| Site | Depth m | Field vane test $\tau_f$ t/m² | Fall-cone test 100 gm—30° $h$ mm Sampler IV | Fall-cone test 100 gm—30° $h$ mm Sampler VI | Natural water content $w$ % | Liquid limit $w_L$ % | Plastic limit $w_P$ % | Unit weight $\gamma$ t/m³ | Type of soil |
|---|---|---|---|---|---|---|---|---|---|
| Torslanda, Göteborg | 3 | 1.50 | 8.9 | — | — | — | — | 1.58 | brown-grey clay |
| | 4 | 2.20 | 7.3 | — | 73 | 71 | 26 | 1.57 | » » with seams of sand |
| | 5 | 2 35 | 6.5 | — | 73 | 72 | 25 | 1.57 | grey clay |
| | 6 | 2.50 | 6.2 | — | 66 | 67 | 24 | 1.60 | dark-grey clay with stains of iron sulphide |
| | 7 | 2.80 | 5.9 | — | 64 | 65 | 24 | 1.62 | grey clay with bands of iron sulphide |
| | 8 | 2.80 | 5.6 | — | 62 | 65 | 24 | 1.64 | dark-grey clay with bands of iron sulphide |
| | 9 | 2.95 | 5.7 | — | 60 | 61 | 25 | 1.66 | — » — |
| | 10 | 2.90 | 5.8 | — | 59 | 61 | 24 | 1.66 | — » — |
| | 11 | 2.85 | 6.0 | — | 54 | 56 | 22 | 1.67 | dark-grey clay |
| | 12 | 2.90 | 5.9 | — | 53 | 52 | 21 | 1.68 | dark-grey clay with bands of iron sulphide |
| | 13 | 2.50 | 7.1 | — | 45 | 42 | 20 | 1.75 | — » — |
| Lake Ullnasjön | 0.75 | 0.30 | 15.9 | — | 188 | 173 | 41 | 1.20 | green-grey clayey nekron mud |
| | 1.30 | 0.50 | 13.2 | — | 188 | 187 | 52 | 1.25 | brown-grey » » » |
| | 1.75 | 0.45 | 12.9 | — | 178 | 166 | 51 | 1.25 | dark-brown » » » |
| | 2.35 | 0.65 | 11.4 | — | 162 | 161 | 37 | 1.27 | » » » » with shells |
| | 2.75 | 0.70 | 10.8 | — | 151 | 140 | 28 | 1.29 | » » » » |
| | 3.35 | 0.90 | 9.4 | — | 155 | 155 | 41 | 1.30 | black » » » |
| The River Göta älv H 29.4/180 | 3.5 | 2.10 | 6.1 | — | 69 | 62 | 27 | 1.59 | grey clay |
| | 4.5 | 2.55 | 6.5 | — | 68 | 64 | 28 | 1.61 | » » |
| | 5.5 | 2.70 | 6.1 | — | 72 | 64 | 27 | 1.59 | » » |
| | 6.5 | 2.95 | 6.0 | — | 66 | 63 | 27 | 1.59 | » » |
| | 7.5 | 3.10 | 5.9 | — | 72 | 68 | 28 | 1.56 | » » |
| | 8.5 | 3.15 | 5.3 | — | 72 | 69 | 29 | 1.57 | » » coloured by iron sulphide |
| | 9.5 | 3.90 | 6.2 | — | 78 | 74 | 30 | 1.53 | dark-grey clay with shells |
| | 11.5 | 4.05 | 5.3 | — | 78 | 74 | 31 | 1.54 | dark-grey clay |
| | 12.5 | 4.00 | 5.0 | — | 76 | 70 | 30 | 1 55 | » » |
| | 13.5 | 4.05 | 5.4 | — | 78 | 75 | 30 | 1.54 | » » |

*Table 3 (Continued)*

| Site | Depth m | Field vane test $\tau_f$ t/m² | Fall-cone test 100 gm—30° $h$ mm Sampler IV | Fall-cone test 100 gm—30° $h$ mm Sampler VI | Natural water content $w$ % | Liquid limit $w_L$ % | Plastic limit $w_P$ % | Unit weight $\gamma$ t/m³ | Type of soil |
|---|---|---|---|---|---|---|---|---|---|
| The River Göta älv H 71.9/40 | 2 | 1.20 | 9.9 | — | 96 | 88 | 29 | 1.48 | grey clay |
| | 3 | 1.40 | 8.3 | — | 89 | 88 | 30 | 1.51 | » » |
| | 4 | 1.45 | 8.9 | — | 79 | 81 | 28 | 1.54 | » » |
| | 5 | 1.35 | 7.5 | — | 77 | 78 | 27 | 1.58 | » » with shells |
| | 6 | 1.40 | 8.7 | — | 80 | 77 | 28 | 1.53 | » » |
| | 7 | 1.40 | 9.3 | — | 83 | 78 | 27 | 1.52 | grey clay |
| | 8 | 1.75 | 7.7 | — | 75 | 76 | 27 | 1.53 | dark-grey clay |
| | 9 | 2.00 | 7.9 | — | 75 | 78 | 28 | 1.58 | » » with shells |
| | 10 | 2.15 | 6.6 | — | 74 | 78 | 29 | 1.55 | » » » » |
| | 11 | 2.45 | 7.2 | — | 70 | 74 | 27 | 1.56 | » » » » |
| | 12 | 2.50 | 6.8 | — | 78 | 82 | 29 | 1.53 | » » » » |
| | 13 | 3.00 | 6.3 | — | 78 | 81 | 30 | 1.54 | dark-grey clay |
| | 14 | 2.80 | 6.6 | — | 87 | 81 | 30 | 1.51 | » » |
| | 15 | 2.70 | 6.5 | — | 77 | 72 | 29 | 1.53 | blue-grey » |
| Highway No. 10 Örebro—Lindesberg | 3 | 0.30 | 17.8 | — | — | — | — | 1.05 | brown non-fibrous peat |
| | 5 | 0.65 | 13.0 | — | — | — | — | 1.05 | » » » |
| | 6 | 0.70 | 15.4 | — | — | — | — | 1.06 | brown gel mud *(dy)* |
| | 7 | 0.80 | 12.5 | — | 354 | 310 | 110 | 1.07 | » » » » |
| | 8 | 0.90 | 10.7 | — | — | — | — | 1.11 | dark-green nekron mud *(gyttja)* |
| | 9 | 0.95 | 11.6 | — | — | — | — | 1.41 | dark-grey clay with thin layers of fine sand |
| | 10 | 1.10 | 10.5 | — | 74 | 61 | 23 | 1.52 | grey varved clay |
| | 12 | 1.40 | 8.8 | — | — | — | — | 1.67 | brown-grey varved clay |
| | 14 | 1.65 | 7.5 | — | 50 | 41 | 19 | 1.74 | » » » |
| | 16 | 1.80 | 7.4 | — | — | — | — | 1.86 | brown-grey varved clay with thin layers of fine sand |

*Table 3 (Continued)*

| Site | Depth m | Field vane test $\tau_f$ t/m² | Fall-cone test 100 gm—30° $h$ mm Sampler IV | Fall-cone test 100 gm—30° $h$ mm Sampler VI | Natural water content $w$ % | Liquid limit $w_L$ % | Plastic limit $w_P$ % | Unit weight $\gamma$ t/m³ | Type of soil |
|---|---|---|---|---|---|---|---|---|---|
| Stora Värtan | 2.6 | 0.50 | 15.0 | — | 103 | 62 | 26 | 1.44 | grey clay |
| Lake Erken | 3.3 | 1.05 | 10.8 | — | 105 | 88 | 30 | 1.33 | dark-grey muddy clay with shells |
| Väsby | 2 | 1.00 | 9.9 | — | — | 132 | 44 | 1.37 | dark-grey muddy clay |
| | 3 | 1.10 | 9.0 | — | 131 | 126 | 42 | 1.31 | » » » |
| | 4 | 1.20 | 8.5 | — | 112 | — | — | 1.39 | » » » |
| | 5 | 1.20 | 8.9 | — | 97 | — | — | 1.40 | dark-grey clay |
| | 6 | 1.20 | 9.7 | — | 93 | 105 | 30 | 1.45 | » » |
| | 7 | 1.40 | 9.5 | — | 96 | — | — | 1.45 | » » |
| | 8 | 1.50 | 8.7 | — | 83 | — | — | 1.48 | grey clay |
| | 9 | 1.60 | 8.2 | — | 77 | — | — | 1.52 | » » |
| | 10 | 1.50 | 8.2 | — | 80 | 72 | 23 | 1.54 | » » |
| | 11 | 2.10 | 7.5 | — | 92 | — | — | 1.55 | » » |
| | 12 | 2.10 | 7.4 | — | 87 | 79 | 25 | 1.51 | grey varved clay |
| | 2 | 0.90 | 11.0 | — | 131 | — | — | 1.35 | dark-grey muddy clay |
| | 3 | 1.10 | 9.2 | — | 114 | — | — | 1.37 | » » » |
| | 4 | 1.00 | 8.7 | — | 107 | — | — | 1.41 | » » » |
| | 5 | 1.30 | 8.4 | — | 100 | — | — | 1.45 | dark-grey clay |
| | 6 | 1.20 | 9.0 | — | 101 | — | — | 1.46 | » » |
| | 7 | 1.20 | 8.8 | — | 81 | — | — | 1.50 | grey clay |
| | 8 | 1.40 | 9.0 | — | 83 | — | — | 1.52 | » » |
| | 9 | 1.50 | 8.0 | — | 76 | — | — | 1.54 | » » |
| | 10 | 1.90 | 8.1 | — | 73 | — | — | 1.49 | » » |
| | 11 | 2.60 | 7.9 | — | 96 | — | — | 1.46 | grey varved clay |
| | 12 | 1.70 | 7.9 | — | 64 | — | — | 1.66 | » » |

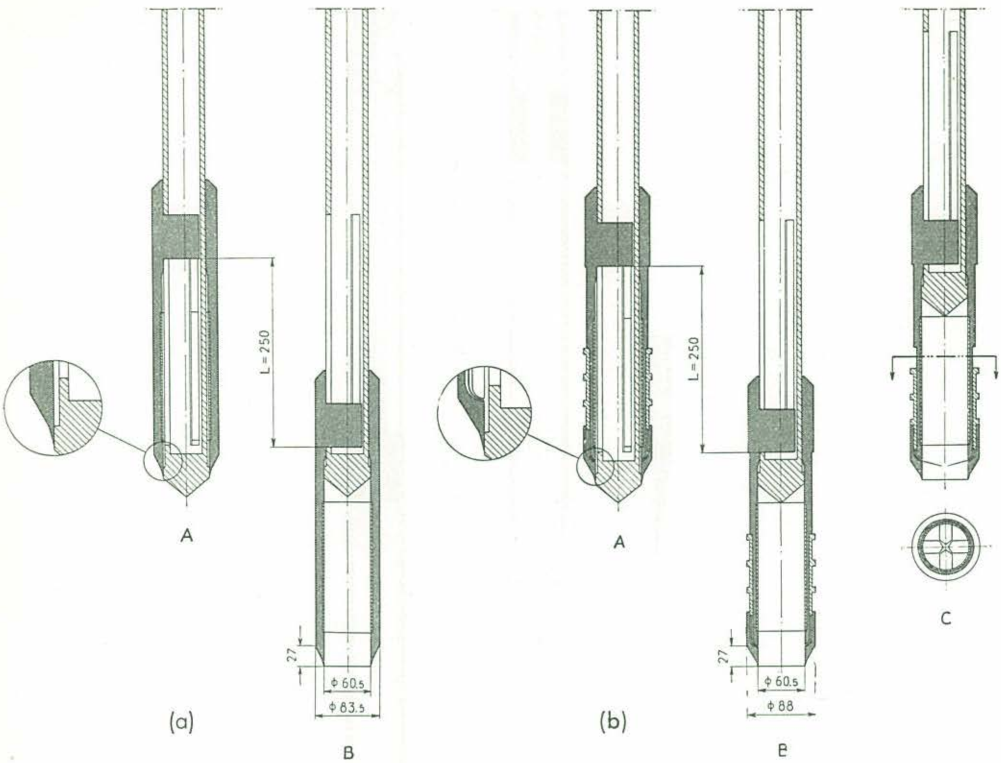



*Fig. 15. Sampler SGI IV, a) without shutter, b) with delayed shutter.*

Using the relation in Fig. 17, it is possible to give in the diagrams, Plate I, the relation between $\tau_f$ and $h$ for the 60gm.—60° cone. We find $K \approx 0.25$ for sampler SGI IV and $\approx 0.20$ for sampler SGI VI. For the 10gm.—60° cone, $K$ has the same value, and the relation between $\tau_f$ and $h$ for the 10gm.—60° cone is thus given as shown in Plate I.

In the original interpretation of the cone test the values of $\tau_f$, calculated from the $H_3$-numbers, were multiplied by a certain factor whose magnitude depended on the content of organic matter (CALDENIUS, 1938, p. 142). At the Institute, for instance, the following reduction factors have been used

| | |
|---|---|
| for gel mud *(dy)*, nekron mud *(gyttja)*, and peat .... | 0.6 |
| clayey mud .................................... | 0.7 |
| muddy clay .................................... | 0.8 to 0.9 |
| clay .......................................... | 1.0 |

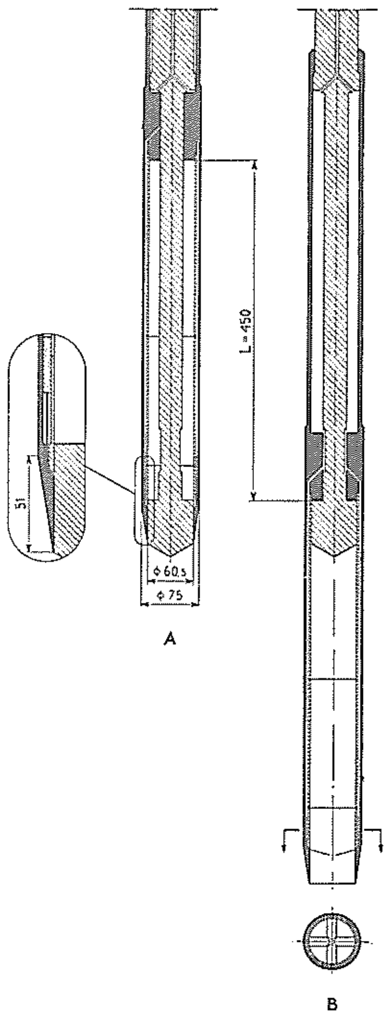



*Fig. 16. Sampler SGI VI.*

On the basis of the theory presented in this report it has not been possible to find an experimental justification of such reduction factors although all the above types of clays were investigated. These reduction factors seem not to be inherent in the cone test itself but to have been related to other phenomena, *e.g.* a varying disturbance between the above types of clay by the sampler or incorrect interpretation of $H_3$ or both.

*Determination of K for Remoulded Clay*

For remoulded clay the value of $K$ in Eq. (16) will be different from the values of $K$ applicable to "undisturbed" clay. Naturally, in this case no importance

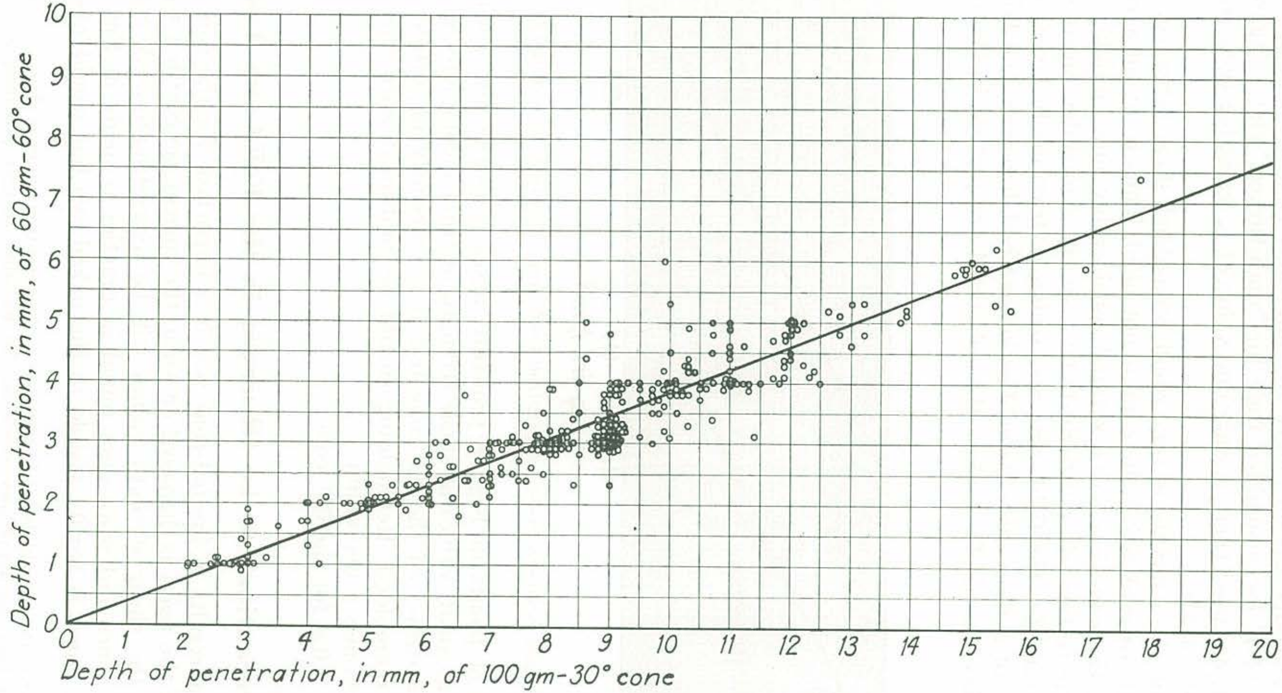



*Fig. 17. Relation between depths of penetration of 100 gm—30° cone and 60 gm—60° cone.*

attaches to disturbances which may occur during transport or sampling.[1] In addition, the stress distribution along the surface of the cone will be quite different as the sensitivity has no longer any influence. It is accordingly easier to give the values of $K$ for remoulded than for "undisturbed" clay. Regardless of the type of sampler used there is only one value of $K$ valid for each particular value of the cone angle.

In the case of clays having a very small remoulded shear strength, the use of the vane test for calibrating the cone is not ideal. Thus to avoid a regain of shear strength, the vane has to be rotated considerably more quickly than in the routine vane testing of undisturbed clay and the remoulded shear strength may therefore (and for the field vane also owing to friction) be overestimated. In the absence of a more suitable calibration method however, the laboratory vane test has been used. The values thus obtained are plotted in the diagram, Fig. 18.

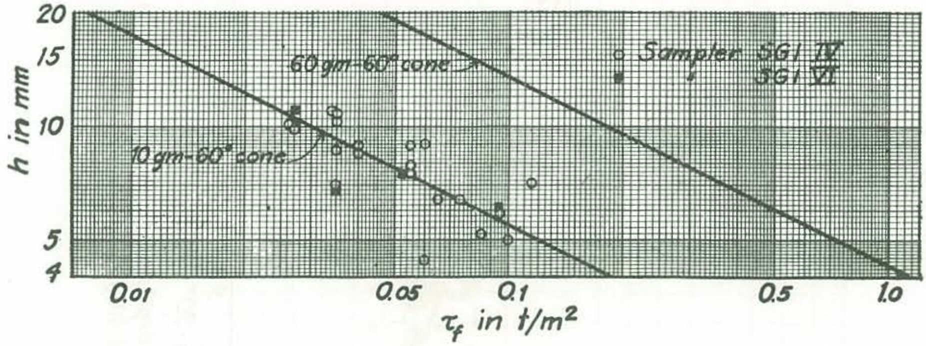



*Fig. 18. Relation between cone penetration and shear strength of remoulded clay.*

The scatter of points in Fig. 18 probably arises from the low sensitivity of the vane test at small values of shear strength or from incomplete remoulding. Thus for the accuracy of the fall-cone test it is extremely important that the clay be fully remoulded. Remoulding should be continued until the depth of penetration becomes a maximum.

We find $K \approx 0.30$ for the 60° cone.

## 6. Precautions to be Observed in the Fall-Cone Test

For accurate results in the fall-cone test it is important for the cone to be in good order. If the point is damaged or worn, the correct penetration will not be attained and the shear strength values will be overestimated. A comparison between new cones and worn ones found in practice is made in Fig. 19. The wear of these cones is hardly visible to the naked eye but would nevertheless

[1] The only factor that would be of importance is the moisture content. The current sampling methods however have no appreciable effect on the moisture content provided the samples are properly stored.

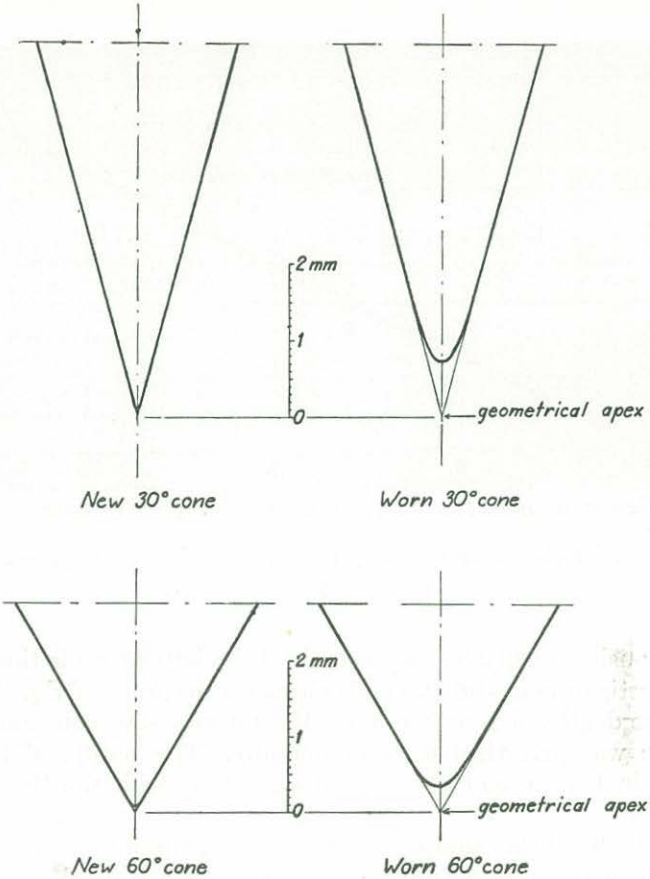



*Fig. 19. Comparison between new and worn cones.*

cause considerable errors in the estimation of $\tau_f$. For example, at $h \approx 7$ mm., experiment has shown the worn 30° cone to give values of $\tau_f$ approximately 25 to 30 % higher than those obtained with the new 30° cone.

A fairly accurate result may be obtained even with a worn cone if it is adjusted so that its geometrical apex is coincident with the surface of the clay. It would seem however more practical to renew the cone.

In the case of medium or stiff clays the depth of penetration is often less than 3 to 4 mm. when the 100gm.—30° cone is used. When the penetration is so small the accuracy of the shear strength values obtained will be influenced very much by the accuracy of the readings. Thus an error of one tenth of a millimetre at $h = 2$ mm. will give an error in $\tau_f$ of approximately 2 t/m$^2$. Obviously, not only the influence of the human factor but also the possible existence of shells and grains of sand *etc.* will play an important role.

In order to reduce the influence of the above factors a heavier cone should be used. According to Eq. (16), the penetration $h$ for a 100 gm.—30° cone is

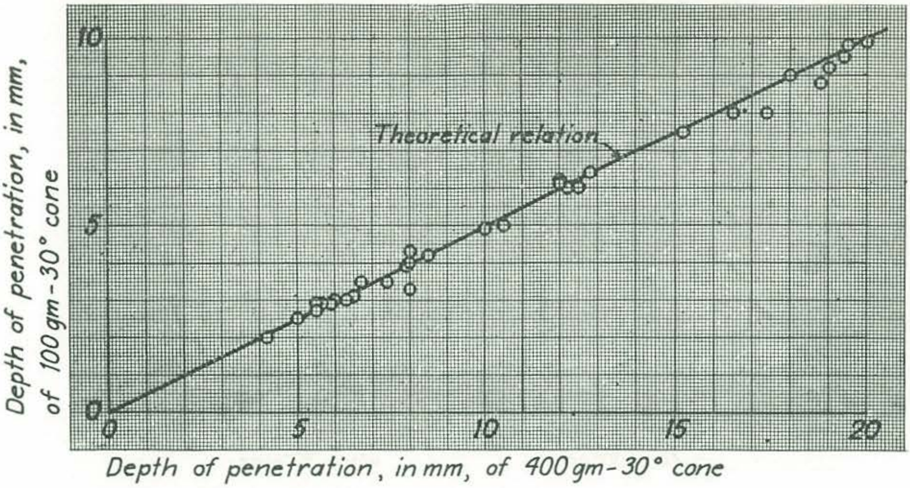



*Fig. 20. Relation between depths of penetration of 100 gm—30° cone and 400 gm—30° cone.*

doubled by using a 400 gm.—30° cone, and the latter cone is thus recommended for the investigation of stiff clays. To check the accuracy of Eq. (16), the relation between the depths of penetration for the 100 gm.—30° cone and the 400 gm.——30° cone was investigated experimentally. The results of the experiments are shown in Fig. 20 and are in good agreement with the theoretical relation, Eq. (16).

If the cylindrical clay sample is not confined around its periphery, the reading $h$ obtained might be misleading due to the elastic vertical compression caused by the impact of the cone. This effect is minimized however, if the sample is retained in the sampling tube during the test.

## 7. Comparison between the Fall-Cone Test and Other Shear Strength Tests

Figs. 21 to 32 offer an opportunity of comparing the shear strength values obtained from the fall-cone test with those obtained from other laboratory tests and from the field vane test.

The fall-cone test gives values of $\tau_f$ which agree remarkably well with the values of $\tau_f$ obtained from the field vane test except in the case of the River Lidan, Fig. 32 (*cf.* Cadling and Odenstad, 1950, p. 60 and Plate 3), where the clay was of extremely high sensitivity. In this case therefore, the samples were no doubt considerably disturbed, and this probably accounts for the divergent results obtained. The high sensitivity may also have affected the

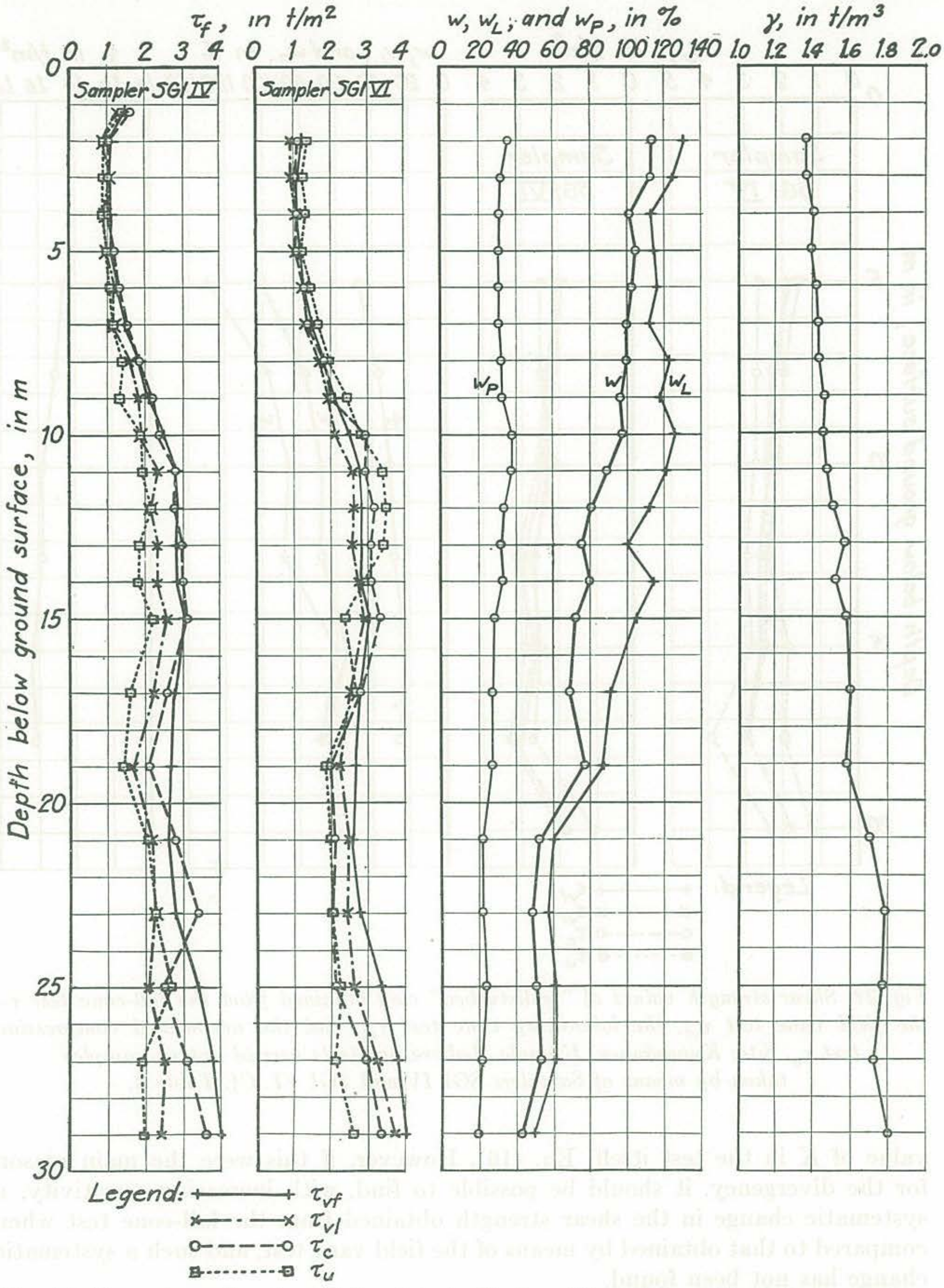



*Fig. 21. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, the laboratory vane test $\tau_{vl}$, and the unconfined compression test $\tau_u$. Site: Enköping. Laboratory tests carried out on samples taken by means of Samplers SGI IV and SGI VI. Cf. Table 3.*

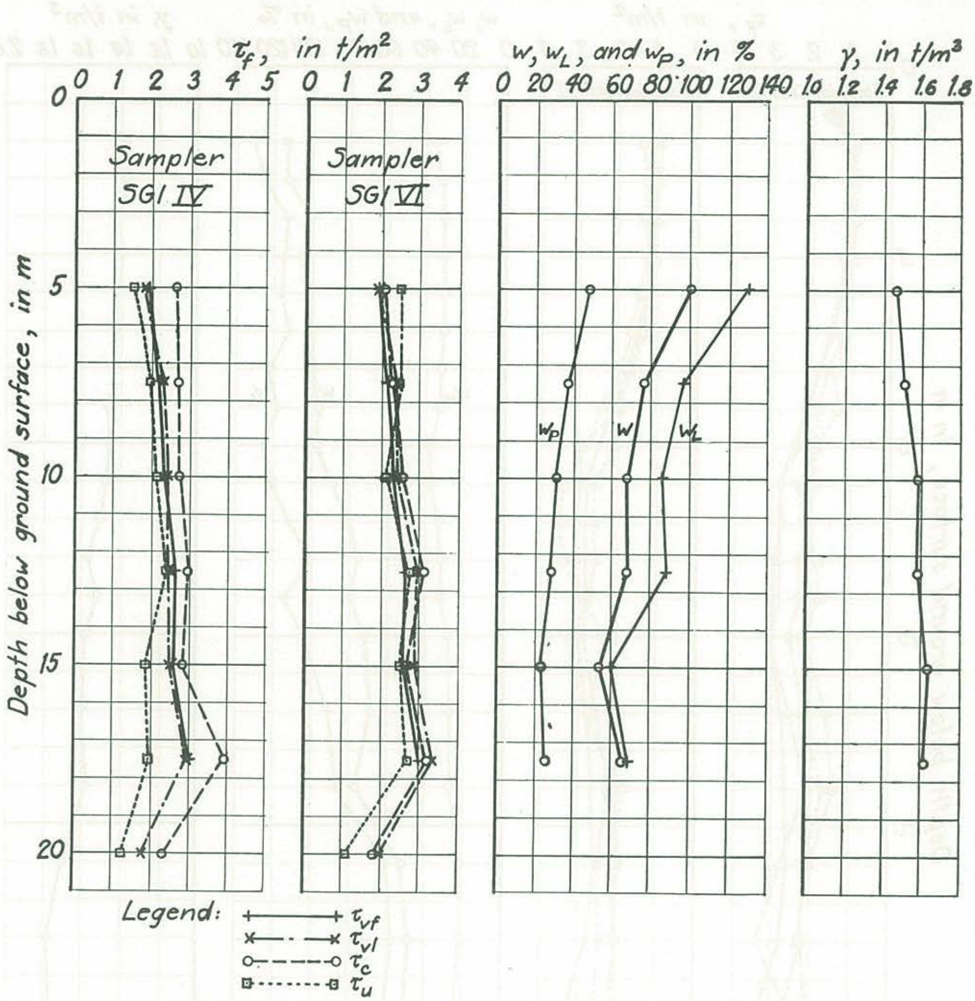



*Fig. 22. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, the laboratory vane test $\tau_{vl}$, and the unconfined compression test $\tau_u$. Site: Kungsängen, Uppsala. Laboratory tests carried out on samples taken by means of Samplers SGI IV and SGI VI. Cf. Table 3.*

value of $K$ in the test itself, Eq. (16). However, if this were the main reason for the divergency, it should be possible to find, with increasing sensitivity, a systematic change in the shear strength obtained from the fall-cone test when compared to that obtained by means of the field vane test, and such a systematic change has not been found.

In the case of quick clays the laboratory shear strength tests must obviously be regarded with suspicion, and whenever possible, field vane tests should be made.

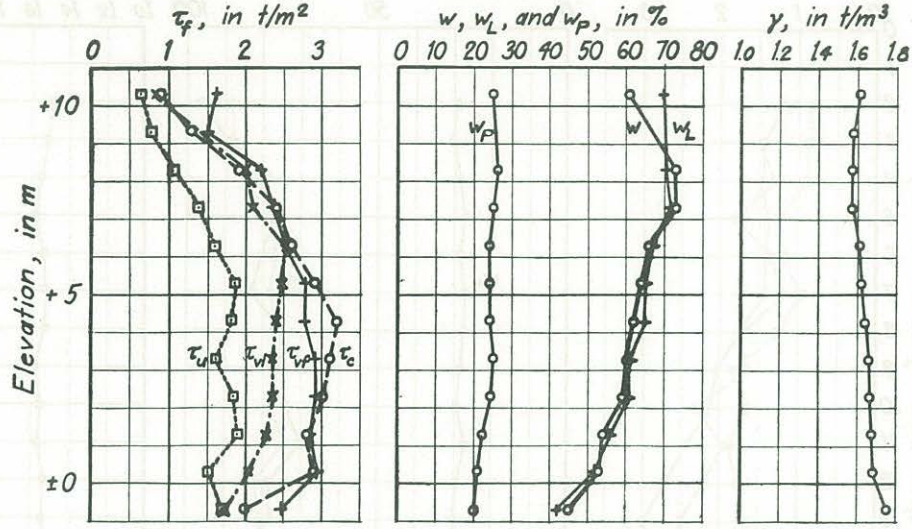



*Fig. 23. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, the laboratory vane test $\tau_{vl}$, and the unconfined compression test $\tau_u$. Site: Torslanda, Gothenburg. Laboratory tests carried out on samples taken by means of Samplers SGI IV. Cf. Table 3.*

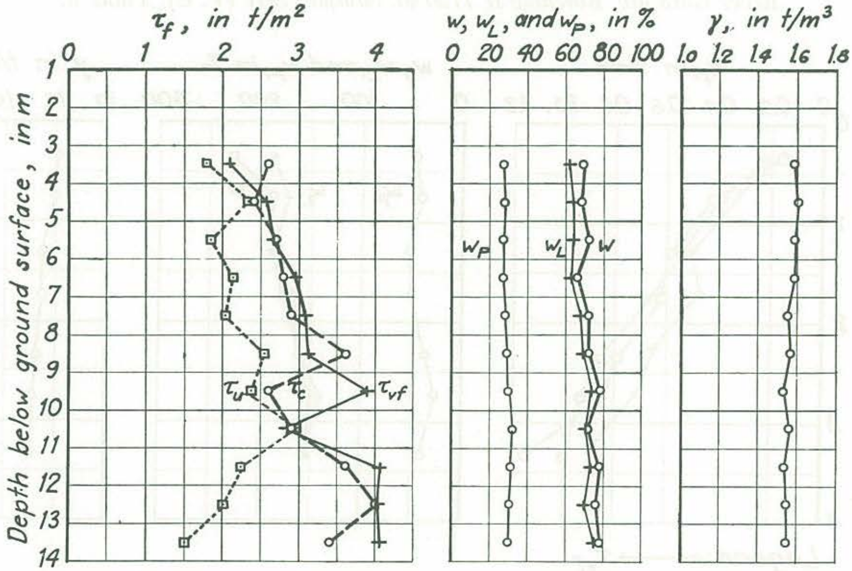



*Fig. 24. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, and the unconfined compression test $\tau_u$. Site: The River Göta älv, Borehole H 29.4/180. Sampler SGI IV. Cf. Table 3.*

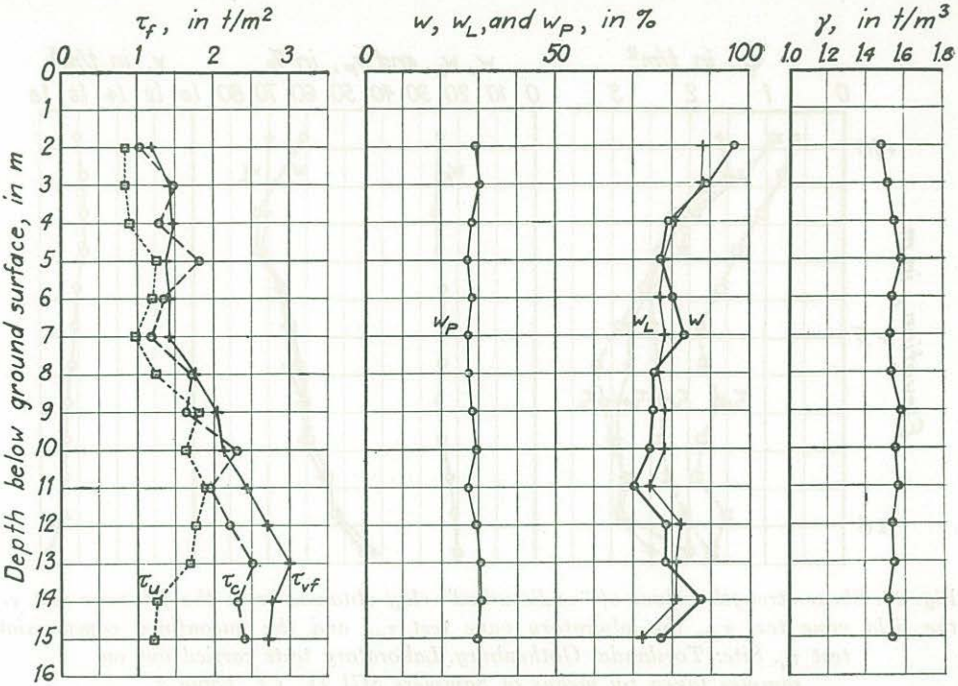



*Fig. 25. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, and the unconfined compression test $\tau_u$. Site: The River Göta älv, Borehole H 71.9/40. Sampler SGI IV. Cf. Table 3.*

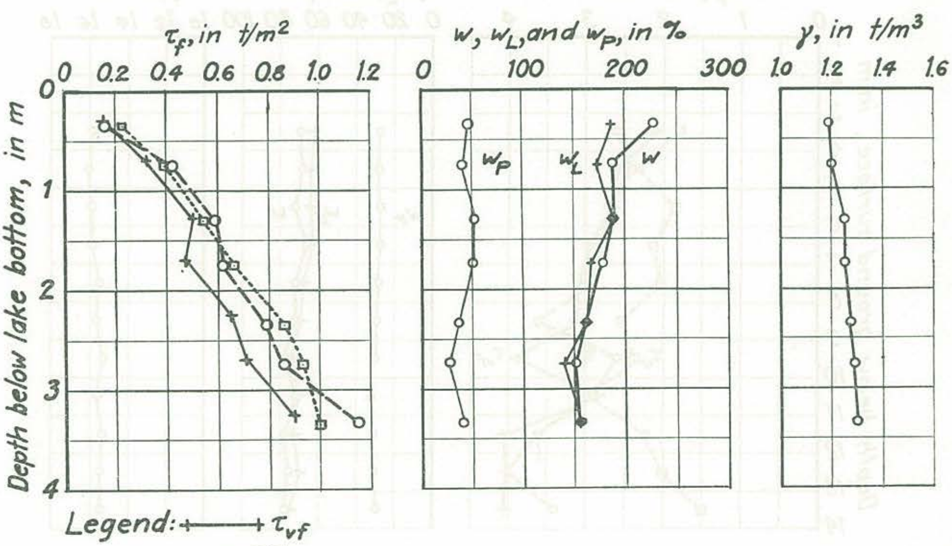



*Fig. 26. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, and the unconfined compression test $\tau_u$. Site: Lake Ullnasjön. Sampler SGI IV. Cf. Table 3.*

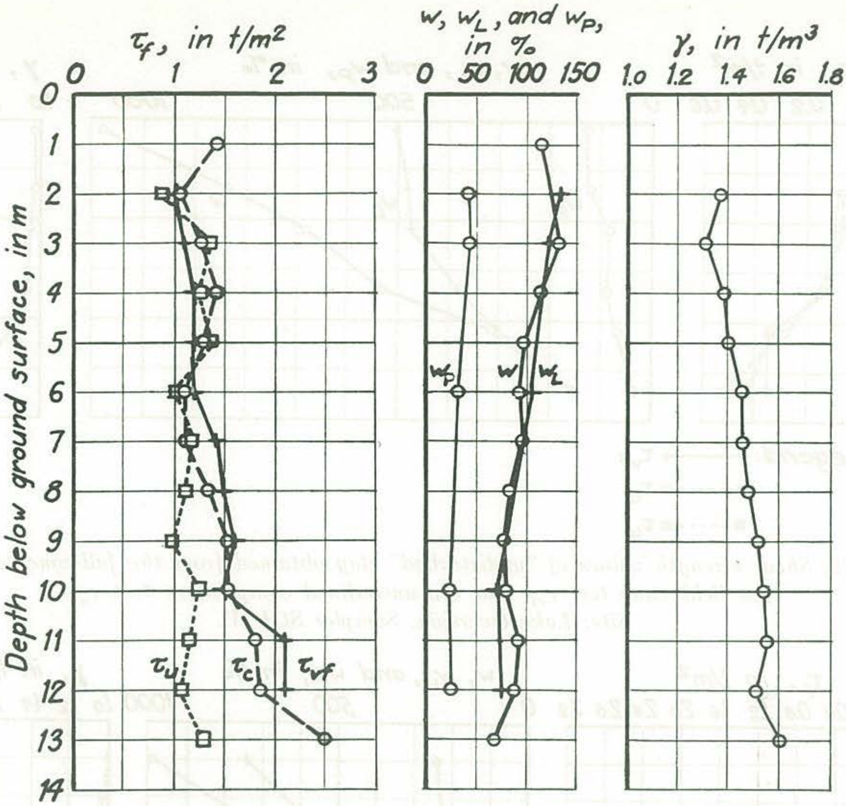



*Fig. 27. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, and the unconfined compression test $\tau_u$. Site: Väsby. Sampler SGI IV. Cf. Table 3.*

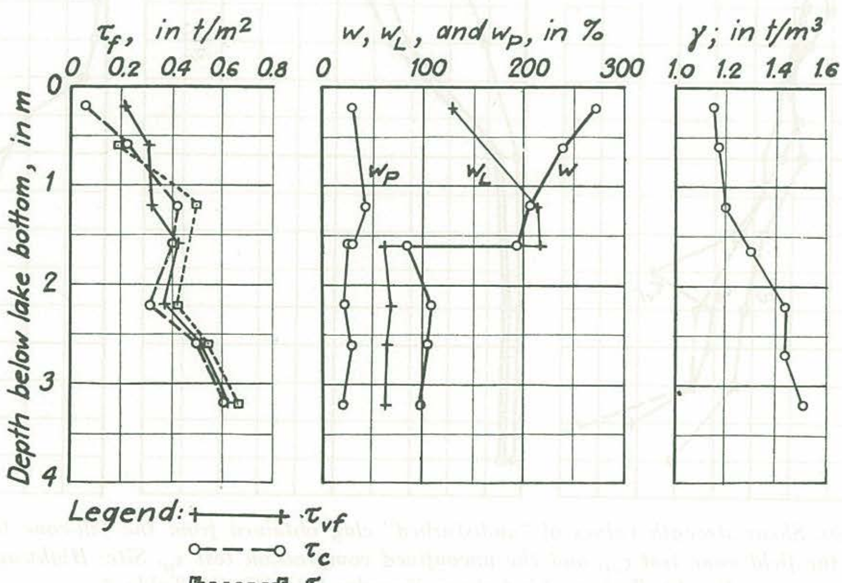



*Fig. 28. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, and the unconfined compression test $\tau_u$. Site: Lake Fågelnäsviken. Sampler SGI IV.*

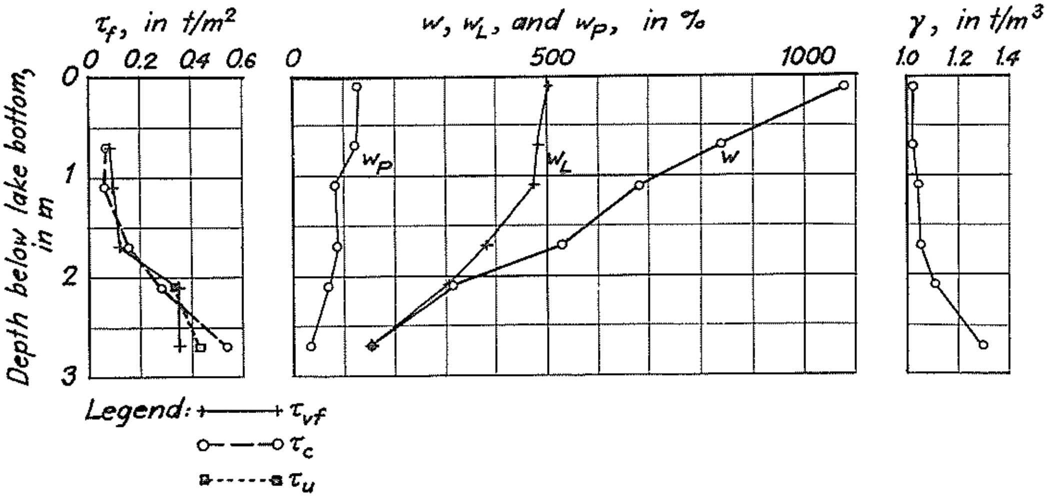



*Fig. 29. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, and the unconfined compression test $\tau_u$. Site: Lake Översjön. Sampler SGI IV.*

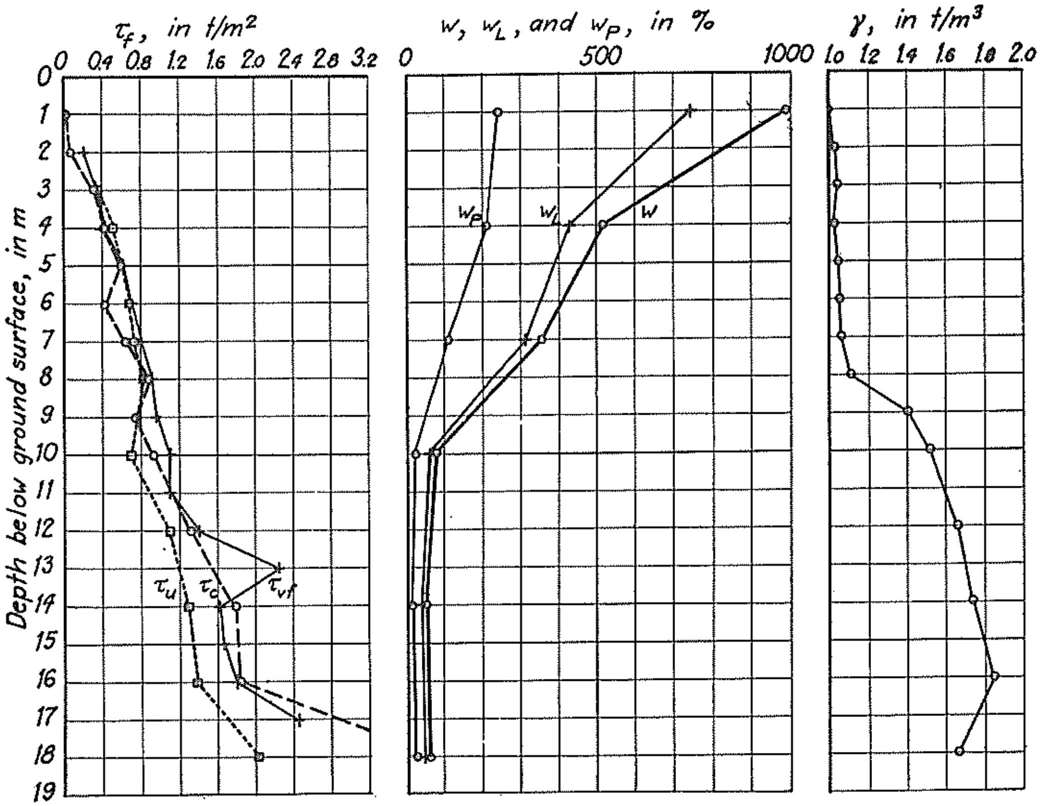



*Fig. 30. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, and the unconfined compression test $\tau_u$. Site: Highway No. 10, Örebro—Lindesberg. Sampler SGI IV. Cf. Table 3.*

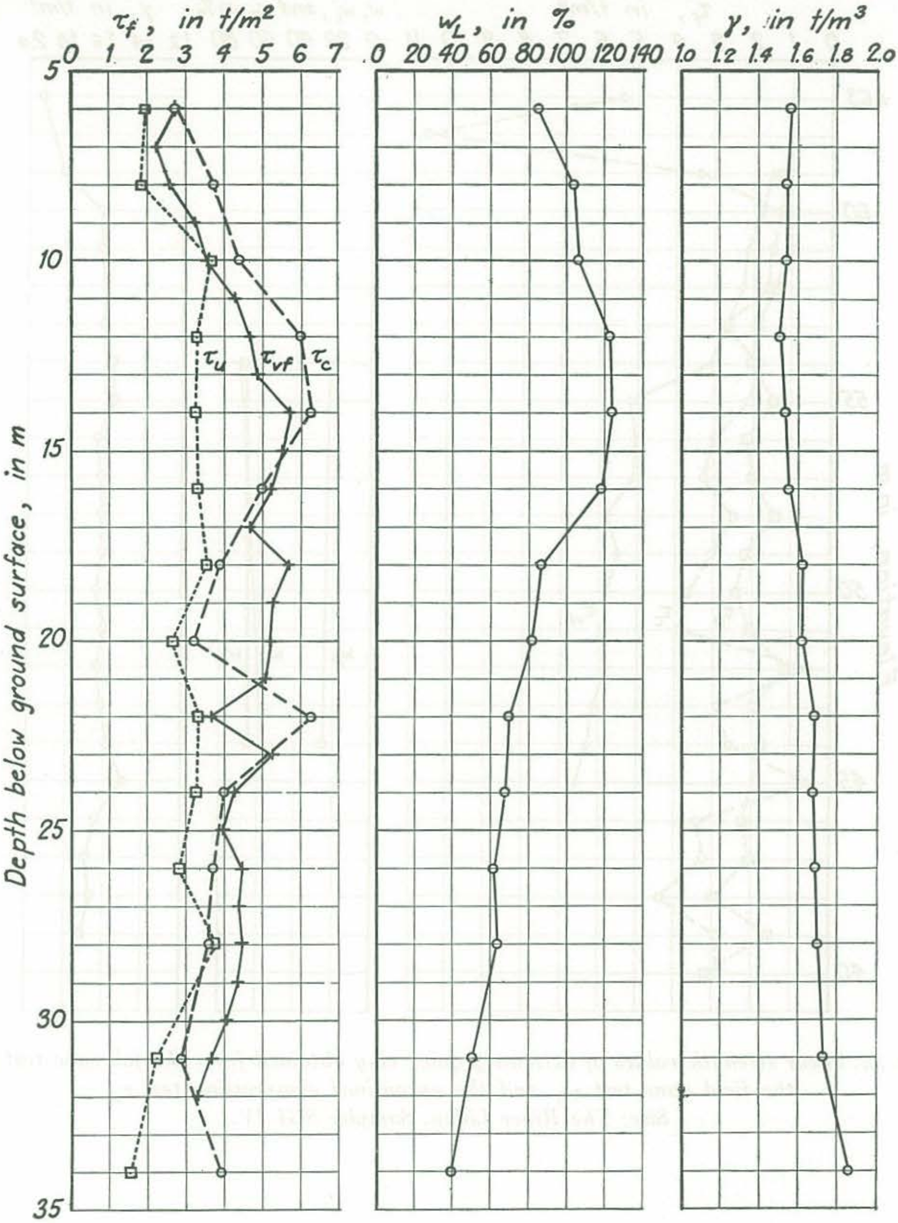



*Fig. 31. Shear strength values of "undisturbed" clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, and the unconfined compression test $\tau_u$. Site: Uppsala. Sampler SGI IV.*

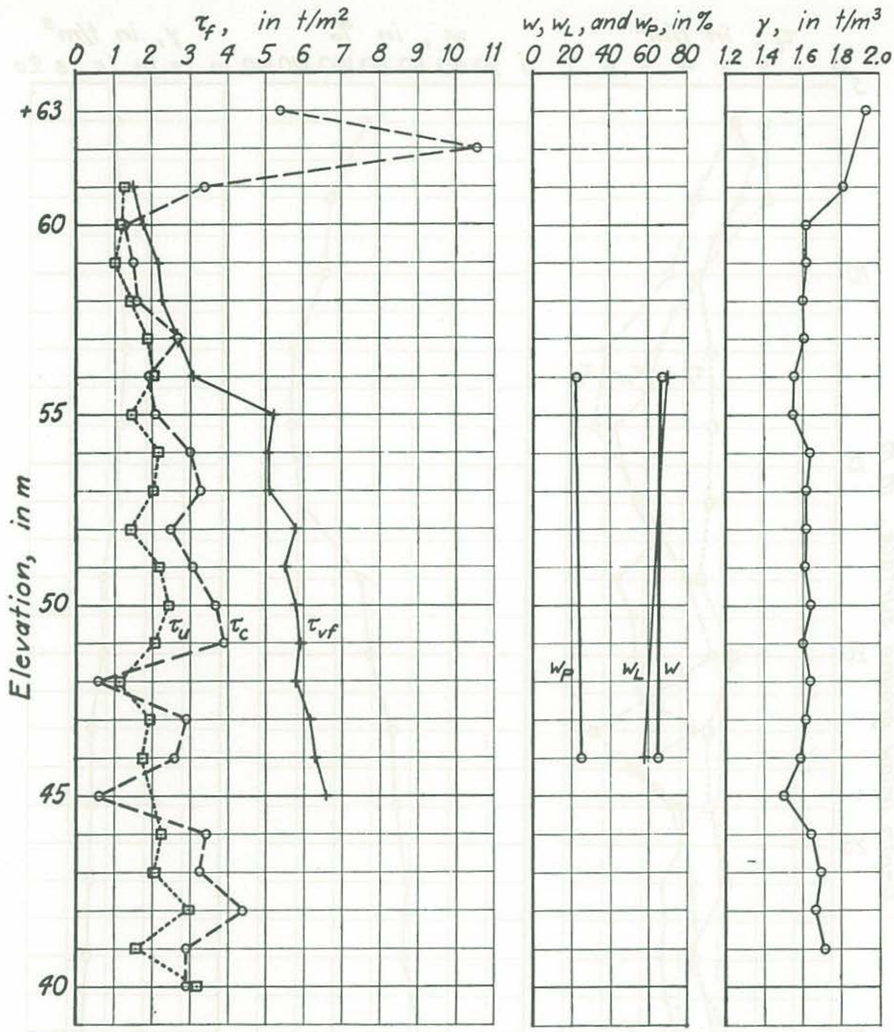



*Fig. 32. Shear strength values of extremely quick clay obtained from the fall-cone test $\tau_c$, the field vane test $\tau_{vf}$, and the unconfined compression test $\tau_u$. Site: The River Lidan. Sampler SGI IV.*

## 8. Conclusions

This paper shows the possibility of finding by the fall-cone test a direct relation between the depth $h$ of cone penetration and the undrained shear strength $\tau_f$ of a clay. The values of $\tau_f$ thus obtained are in close agreement with those obtained by the field vane test provided that the appropriate angle and sampler factor $K$ is chosen. Thus the $\tau_f$ values given by the new interpretation of the fall-cone test differ from those obtaining hitherto, and a re-appraisal of the safety factors to be applied may be necessary.

The investigation indicates the importance of relating the fall-cone test to the type of sampler used. The difference in disturbance caused by the two types of samplers mentioned in this paper, namely SGI IV and SGI VI, is shown by the fall-cone test results to be approximately 25 %, and this of course must be allowed for in deciding the $K$-value.

In the interpretation of the fall-cone test presented here, the use of reduction factors in the case of organic clays (gel mud, nekron mud, peat, *etc.*) is avoided. The need for reduction factors has been due to the hitherto existing interpretation of the $H$-numbers and to poor sampling methods, organic clays being less susceptible to disturbance than inorganic clays.

To obtain correct results by means of the fall-cone test it is necessary that the apex of the cone is not damaged or worn.

The 100 gm.—30° cone is too light to use in the case of stiff clays. If however, a 400 gm.—30° cone is added to the present standard series of cones, the fall-cone test can be used with satisfactory accuracy on clays with shear strengths up to about 20 t/m$^2$.

*Table I. Relation between the depth of cone penetration h and the undrained shear strength $\tau_f$ of undisturbed clay according to Eq. (16) Samples takes with Sampler SGI IV.*

| h mm | $\tau_f$ in t/m$^2$ by use of 400 gm—30° cone | 100 gm—30° cone | 60 gm—60° cone | 10 gm—60° cone |
|---|---|---|---|---|
| 4.0 | 25 | 6.3 | 0.94 | 0.16 |
| 4.1 | 24 | 6.0 | 0.90 | 0.15 |
| 4.2 | 23 | 5.7 | 0.85 | 0.14 |
| 4.3 | 22 | 5.4 | 0.81 | 0.13 |
| 4.4 | 21 | 5.2 | 0.77 | 0.13 |
| 4.5 | 20 | 5.0 | 0.74 | 0.12 |
| 4.6 | 19.1 | 4.8 | 0.71 | 0.12 |
| 4.7 | 18.3 | 4.6 | 0.68 | 0.11 |
| 4.8 | 17.6 | 4.4 | 0.65 | 0.11 |
| 4.9 | 16.9 | 4.2 | 0.62 | 0.10 |
| 5.0 | 16.2 | 4.0 | 0.60 | 0.10 |
| 5.1 | 15.6 | 3.9 | 0.58 | 0.096 |
| 5.2 | 15.0 | 3.7 | 0.56 | 0.092 |
| 5.3 | 14.4 | 3.6 | 0.53 | 0.089 |
| 5.4 | 13.9 | 3.4 | 0.51 | 0.086 |
| 5.5 | 13.4 | 3.3 | 0.50 | 0.083 |
| 5.6 | 12.9 | 3.2 | 0.48 | 0.080 |
| 5.7 | 12.5 | 3.1 | 0.46 | 0.077 |
| 5.8 | 12.0 | 3.0 | 0.45 | 0.074 |
| 5.9 | 11.6 | 2.9 | 0.43 | 0.072 |
| 6.0 | 11.2 | 2.8 | 0.42 | 0.069 |
| 6.1 | 10.9 | 2.7 | 0.40 | 0.067 |
| 6.2 | 10.6 | 2.6 | 0.39 | 0.065 |
| 6.3 | 10.3 | 2.5 | 0.38 | 0.063 |
| 6.4 | 10.0 | 2.5 | 0.37 | 0.061 |
| 6.5 | 9.7 | 2.4 | 0.35 | 0.059 |
| 6.6 | 9.4 | 2.3 | 0.34 | 0.057 |
| 6.7 | 9.1 | 2.3 | 0.33 | 0.056 |
| 6.8 | 8.8 | 2.2 | 0.32 | 0.054 |
| 6.9 | 8.5 | 2.1 | 0.32 | 0.052 |
| 7.0 | 8.3 | 2.1 | 0.31 | 0.051 |
| 7.1 | 8.1 | 2.0 | 0.30 | 0.050 |
| 7.2 | 7.9 | 1.95 | 0.29 | 0.048 |
| 7.3 | 7.7 | 1.90 | 0.28 | 0.047 |
| 7.4 | 7.5 | 1.85 | 0.27 | 0.045 |
| 7.5 | 7.3 | 1.80 | 0.27 | 0.044 |
| 7.6 | 7.1 | 1.75 | 0.26 | 0.043 |
| 7.7 | 6.9 | 1.70 | 0.25 | 0.042 |
| 7.8 | 6.7 | 1.66 | 0.25 | 0.041 |
| 7.9 | 6.5 | 1.62 | 0.24 | 0.040 |
| 8 0 | 6.3 | 1.58 | 0.23 | 0.039 |

*Table I (Continued)*

| h mm | $\tau_f$ in $t/m^2$ by use of | | | |
|---|---|---|---|---|
| | 400 gm—30° cone | 100 gm—30° cone | 60 gm—60° cone | 10 gm—60° cone |
| 8.1 | 6.2 | 1.54 | 0.23 | 0.038 |
| 8.2 | 6.0 | 1.51 | 0.22 | 0.037 |
| 8.3 | 5.9 | 1.47 | 0.22 | 0.036 |
| 8.4 | 5.7 | 1.44 | 0.21 | 0.035 |
| 8.5 | 5.6 | 1.40 | 0.21 | 0.035 |
| 8.6 | 5.5 | 1.37 | 0.20 | 0.034 |
| 8.7 | 5.3 | 1.34 | 0.20 | 0.033 |
| 8.8 | 5.2 | 1.31 | 0.19 | 0.032 |
| 8.9 | 5.1 | 1.28 | 0.19 | 0.032 |
| 9 0 | 5.0 | 1.25 | 0.18 | 0.031 |
| 9.1 | 4.9 | 1.22 | 0.18 | 0.030 |
| 9.2 | 4.8 | 1.20 | 0.18 | 0.030 |
| 9.3 | 4.7 | 1.17 | 0.17 | 0.029 |
| 9.4 | 4.6 | 1.15 | 0.17 | 0.028 |
| 9.5 | 4.5 | 1.12 | 0.17 | 0.028 |
| 9.6 | 4.4 | 1.10 | 0.16 | 0.027 |
| 9.7 | 4.3 | 1.08 | 0.16 | 0.027 |
| 9.8 | 4.2 | 1.05 | 0.16 | 0.026 |
| 9.9 | 4.1 | 1.03 | 0.15 | 0.026 |
| 10.0 | 4.0 | 1.01 | 0.15 | 0.025 |
| 10.1 | 4.0 | 0.99 | 0.15 | 0.025 |
| 10.2 | 3.9 | 0.97 | 0.14 | 0.024 |
| 10.3 | 3.8 | 0.96 | 0.14 | 0.024 |
| 10.4 | 3.7 | 0.94 | 0.14 | 0.023 |
| 10.5 | 3.7 | 0.92 | 0.14 | 0.023 |
| 10.6 | 3.6 | 0.90 | 0.13 | 0.022 |
| 10.7 | 3.5 | 0.88 | 0.13 | 0.022 |
| 10.8 | 3.5 | 0.87 | 0.13 | 0.021 |
| 10.9 | 3.4 | 0.85 | 0.13 | 0.021 |
| 11.0 | 3.3 | 0.84 | 0.12 | 0.021 |
| 11.1 | 3.3 | 0.82 | 0.12 | 0.020 |
| 11.2 | 3.2 | 0.81 | 0.12 | 0.020 |
| 11.3 | 3.2 | 0.79 | 0.12 | 0.020 |
| 11.4 | 3.1 | 0.78 | 0.11 | 0.019 |
| 11.5 | 3.1 | 0.77 | 0.11 | 0.019 |
| 11.6 | 3.0 | 0.75 | 0.11 | 0.019 |
| 11.7 | 3.0 | 0.74 | 0.11 | 0.018 |
| 11.8 | 2.9 | 0.73 | 0.11 | 0.018 |
| 11.9 | 2.9 | 0.71 | 0.11 | 0.018 |
| 12.0 | 2.8 | 0.70 | 0.10 | 0.017 |
| 12.1 | 2.8 | 0.69 | 0.10 | 0.017 |
| 12.2 | 2.7 | 0.68 | 0.10 | 0.017 |
| 12.3 | 2.7 | 0.67 | 0.099 | 0.017 |
| 12.4 | 2.6 | 0.66 | 0.097 | 0.016 |

*Table I (Continued)*

| h mm | $\tau_f$ in t/m² by use of | | | |
|---|---|---|---|---|
| | 400 gm—30° cone | 100 gm—30° cone | 60 gm—60° cone | 10 gm—60° cone |
| 12.5 | 2.6 | 0.65 | 0.096 | 0.016 |
| 12.6 | 2.5 | 0.64 | 0.095 | 0.016 |
| 12.7 | 2.5 | 0.63 | 0.093 | 0.016 |
| 12.8 | 2.5 | 0.62 | 0.092 | 0.015 |
| 12.9 | 2.4 | 0.61 | 0.091 | 0.015 |
| 13.0 | 2.4 | 0.60 | 0.089 | 0.015 |
| 13.1 | 2.4 | 0.59 | 0.088 | 0.015 |
| 13.2 | 2.3 | 0.58 | 0.086 | 0.014 |
| 13.3 | 2.3 | 0.57 | 0.085 | 0.014 |
| 13.4 | 2 3 | 0.56 | 0.084 | 0.014 |
| 13.5 | 2.2 | 0.56 | 0.082 | 0.014 |
| 13.6 | 2.2 | 0.55 | 0.081 | 0.013 |
| 13.7 | 2.2 | 0.54 | 0.080 | 0.013 |
| 13.8 | 2.1 | 0.53 | 0.079 | 0.013 |
| 13.9 | 2.1 | 0.52 | 0 078 | 0.013 |
| 14.0 | 2.1 | 0.52 | 0.077 | 0.013 |
| 14.2 | 2.0 | 0.50 | 0.074 | 0.012 |
| 14.4 | 1.95 | 0.49 | 0.072 | 0.012 |
| 14.6 | 1.90 | 0.48 | 0.070 | 0.012 |
| 14.8 | 1.85 | 0.47 | 0.069 | 0.011 |
| 15.0 | 1.80 | 0.45 | 0.067 | 0.011 |
| 15.2 | 1.75 | 0.44 | 0.065 | 0.011 |
| 15.4 | 1.70 | 0.43 | 0.063 | 0.011 |
| 15.6 | 1.66 | 0.42 | 0.062 | 0.010 |
| 15.8 | 1.62 | 0.41 | 0.060 | 0.010 |
| 16.0 | 1.58 | 0.40 | 0.058 | 0.0098 |
| 16.2 | 1.54 | 0.39 | 0.057 | 0.0095 |
| 16.4 | 1.51 | 0.38 | 0.056 | 0.0093 |
| 16.6 | 1.47 | 0.37 | 0.055 | 0.0091 |
| 16.8 | 1.44 | 0.36 | 0.053 | 0.0089 |
| 17.0 | 1.40 | 0.35 | 0.052 | 0.0087 |
| 17.2 | 1.37 | 0.34 | 0.051 | 0.0085 |
| 17.4 | 1.34 | 0.33 | 0.050 | 0.0083 |
| 17.6 | 1.31 | 0.33 | 0.049 | 0.0081 |
| 17.8 | 1.28 | 0.32 | 0.047 | 0.0079 |
| 18.0 | 1.25 | 0.31 | 0.046 | 0.0078 |
| 18.5 | 1.18 | 0.30 | 0.044 | 0.0073 |
| 19.0 | 1.12 | 0.28 | 0.042 | 0.0069 |
| 19.5 | 1.07 | 0.27 | 0.040 | 0.0066 |
| 20.0 | 1.01 | 0.25 | 0.038 | 0.0063 |

*Table II. Relation between the depth of cone penetration h and the undrained shear strength $\tau_f$ of remoulded clay according to Eq. (16).*

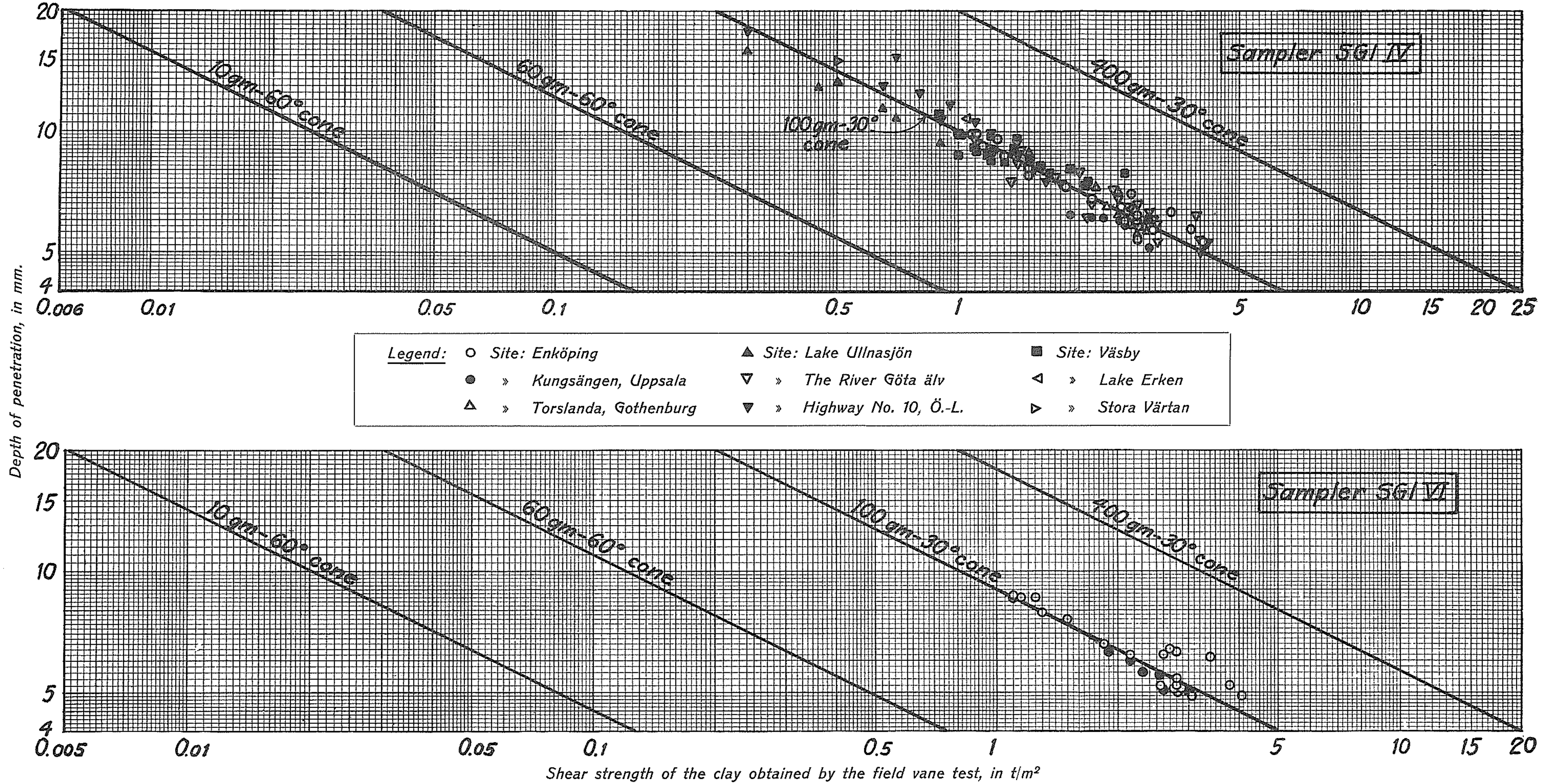
| $h$ mm | $\tau_f$ in t/m² by use of | | $h$ mm | $\tau_f$ in t/m² by use of | |
|---|---|---|---|---|---|
| | 60 gm—60° cone | 10 gm—60° cone | | 60 gm—60° cone | 10 gm—60° cone |
| 4.0 | 1.13 | 0.19 | 8.0 | 0.28 | 0.047 |
| 4.1 | 1.07 | 0.18 | 8.1 | 0.27 | 0.046 |
| 4.2 | 1.02 | 0.17 | 8.2 | 0.27 | 0.045 |
| 4.3 | 0.97 | 0.16 | 8.3 | 0.26 | 0.044 |
| 4.4 | 0.93 | 0.15 | 8.4 | 0.26 | 0.042 |
| 4.5 | 0.89 | 0.15 | 8.5 | 0.25 | 0.041 |
| 4.6 | 0.85 | 0.14 | 8.6 | 0.25 | 0.040 |
| 4.7 | 0.81 | 0.14 | 8.7 | 0.24 | 0.039 |
| 4.8 | 0.78 | 0.13 | 8.8 | 0.23 | 0.038 |
| 4.9 | 0.75 | 0.12 | 8.9 | 0.23 | 0.038 |
| 5.0 | 0.72 | 0.12 | 9.0 | 0.22 | 0.037 |
| 5.1 | 0.69 | 0.11 | 9.1 | 0.22 | 0.036 |
| 5.2 | 0.66 | 0.11 | 9.2 | 0.21 | 0.035 |
| 5.3 | 0.64 | 0.11 | 9.3 | 0.21 | 0.035 |
| 5.4 | 0.62 | 0.10 | 9.4 | 0.20 | 0.034 |
| 5.5 | 0.60 | 0.099 | 9.5 | 0.20 | 0.033 |
| 5.6 | 0.57 | 0.096 | 9.6 | 0.19 | 0.033 |
| 5.7 | 0.55 | 0.092 | 9.7 | 0.19 | 0.032 |
| 5.8 | 0.53 | 0.089 | 9.8 | 0.19 | 0.031 |
| 5.9 | 0.51 | 0.086 | 9.9 | 0.18 | 0.031 |
| 6.0 | 0.50 | 0.083 | 10.0 | 0.18 | 0.030 |
| 6.1 | 0.48 | 0.081 | 10.1 | 0.18 | 0.029 |
| 6.2 | 0.47 | 0.078 | 10.2 | 0.17 | 0.029 |
| 6.3 | 0.45 | 0.076 | 10.3 | 0.17 | 0.028 |
| 6.4 | 0.44 | 0.073 | 10.4 | 0.17 | 0.028 |
| 6.5 | 0.43 | 0.071 | 10.5 | 0.16 | 0.027 |
| 6.6 | 0.41 | 0.069 | 10.6 | 0.16 | 0.027 |
| 6.7 | 0.40 | 0.067 | 10.7 | 0.16 | 0.026 |
| 6.8 | 0.39 | 0.065 | 10.8 | 0.15 | 0.026 |
| 6.9 | 0.38 | 0.063 | 10.9 | 0.15 | 0.025 |
| 7.0 | 0.37 | 0.061 | 11.0 | 0.15 | 0.025 |
| 7.1 | 0.36 | 0.060 | 11.1 | 0.15 | 0.024 |
| 7.2 | 0.35 | 0.058 | 11.2 | 0.14 | 0.024 |
| 7.3 | 0.34 | 0.056 | 11.3 | 0.14 | 0.023 |
| 7.4 | 0.33 | 0.055 | 11.4 | 0.14 | 0.023 |
| 7.5 | 0.32 | 0.053 | 11.5 | 0.14 | 0.023 |
| 7.6 | 0.31 | 0.052 | 11.6 | 0.13 | 0.022 |
| 7.7 | 0.30 | 0.051 | 11.7 | 0.13 | 0.022 |
| 7.8 | 0.29 | 0.049 | 11.8 | 0.13 | 0.021 |
| 7.9 | 0.29 | 0.048 | 11.9 | 0.13 | 0.021 |

*Table II (Continued)*

| h mm | $\tau_f$ in t/m² by use of | | h mm | $\tau_f$ in t/m² by use of | |
|---|---|---|---|---|---|
| | 60 gm—60° cone | 10 gm—60° cone | | 60 gm—60° cone | 10 gm—60° cone |
| 12.0 | 0.12 | 0.021 | 14.6 | 0.084 | 0.014 |
| 12.1 | 0.12 | 0.020 | 14.8 | 0.082 | 0.014 |
| 12.2 | 0.12 | 0.020 | 15.0 | 0.080 | 0.013 |
| 12.3 | 0.12 | 0.020 | 15.2 | 0.078 | 0.013 |
| 12.4 | 0.12 | 0.019 | 15.4 | 0.076 | 0.013 |
| 12.5 | 0.11 | 0.019 | 15.6 | 0.074 | 0.012 |
| 12.6 | 0.11 | 0.019 | 15.8 | 0.072 | 0.012 |
| 12.7 | 0.11 | 0.019 | 16.0 | 0.070 | 0.012 |
| 12.8 | 0.11 | 0.018 | 16.2 | 0.069 | 0.011 |
| 12.9 | 0.11 | 0.018 | 16.4 | 0.067 | 0.011 |
| 13.0 | 0.11 | 0.018 | 16.6 | 0.065 | 0.011 |
| 13.1 | 0.10 | 0.017 | 16.8 | 0.064 | 0.011 |
| 13.2 | 0.10 | 0.017 | 17.0 | 0.062 | 0.010 |
| 13.3 | 0.10 | 0.017 | 17.2 | 0.061 | 0.010 |
| 13.4 | 0.10 | 0.017 | 17.4 | 0.059 | 0.0099 |
| 13.5 | 0.099 | 0.016 | 17.6 | 0.058 | 0.0097 |
| 13.6 | 0.097 | 0.016 | 17.8 | 0.057 | 0.0095 |
| 13.7 | 0.096 | 0.016 | 18.0 | 0.055 | 0.0093 |
| 13.8 | 0.095 | 0.016 | 18.5 | 0.053 | 0.0088 |
| 13.9 | 0.093 | 0.015 | 19.0 | 0.050 | 0.0083 |
| 14.0 | 0.092 | 0.015 | 19.5 | 0.047 | 0.0079 |
| 14.2 | 0.089 | 0.015 | 20.0 | 0.045 | 0.0075 |
| 14.4 | 0.087 | 0.014 | | | |

*Bibliography*


1. BJERRUM, L., 1954. Geotechnical Properties of Norwegian Clays. (Norw. Geot. Inst., Publ. No. 4). Oslo.
2. CADLING, L. and ODENSTAD, S., 1950. The Vane Borer. (R. Swed. Geot. Inst., Proc. No. 2). Stockholm.
3. CALDENIUS, C., 1938. Några rön från grundundersökningar i Göteborg rörande fasthetens variation inom lerorna. Tekn. Tidskr. Vol. 68 H. 51.
4. CALDENIUS, C., and LUNDSTRÖM, R., 1956. The Landslide at Surte on the River Göta Älv. (Sv. Geol. Unders. Ser. Ca. No. 27). Stockholm.
5. CASAGRANDE, A., 1936. The Determination of the Pre-Consolidation Load and its Practical Significance, Discussion. Proc. 1. Internat. Conf. Soil Mech. Found. Engng., Vol. 3.
6. CASAGRANDE, A. and SHANNON, W. L., 1948. Stress-Deformation and Strength Characteristics of Soils under Dynamic Loads. Proc. 2. Internat. Conf. Soil Mech. Found. Engng., Vol. 5.
7. HILL, R., 1950. The Mathematical Theory of Plasticity. Oxford.
8. HOFFMAN, O. and SACHS, G., 1953. Theory of Plasticity for Engineers. New York.
9. HULTIN, T., 1937. Försök till bestämning av Göteborgslerans hållfasthet. (Tekn. Samf. Handl. No. 2). Göteborg.
10. HVORSLEV, J., 1937. Über die Festigkeitseigenschaften gestörter bindiger Boden. (Ing.-vidensk. Skr. A No. 45). Kopenhagen.
11. JAKOBSON, B., 1954. Influence of Sampler Type and Testing Method on Shear Strength of Clay Samples. (R. Swed. Geot. Inst. Proc. No. 8). Stockholm.
12. NÁDAI, A., 1927. Der bildsame Zustand der Werkstoffe. Berlin.
13. NÁDAI, A., 1931. Plasticity. New York.
14. SKAVEN HAUG, S., 1931. Skjærfasthetsforsøk med leire. Medd. Norg. Statsb. Vol. 6 Nr. 6.
15. SKEMPTON, A. W. and BISHOP, A. W., 1950. The Measurement of the Shear Strength of Soils. Géotechnique, Vol. 2, Nr. 2.
16. Statens Järnvägar: Geotekniska Kommissionen 1914—22. Slutbetänkande. (Stat. Järnv. Geot. Medd. Nr. 2). Stockholm.


Sampler SGI IV
10gm-60° cone
60gm-60° cone
100gm-30° cone
400gm-30° cone
20
15
10
5
4
0.006
0.01
0.05
0.1
0.5
1
5
10
15
20
25
Depth of penetration, in mm.
Legend: ○ Site: Enköping
● » Kungsängen, Uppsala
△ » Torslanda, Gothenburg
▲ Site: Lake Ullnasjön
▽ » The River Göta älv
▼ » Highway No. 10, Ö.-L.
■ Site: Väsby
◁ » Lake Erken
▷ » Stora Värtan
Sampler SGI VI
10gm-60° cone
60gm-60° cone
100gm-30° cone
400gm-30° cone
20
15
10
5
4
0.005
0.01
0.05
0.1
0.5
1
5
10
15
20
Shear strength of the clay obtained by the field vane test, in t/m²

# LIST OF PUBLICATIONS

# OF THE ROYAL SWEDISH GEOTECHNICAL

# INSTITUTE

## *Proceedings*



## *Meddelanden*