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On Cohesive Soils
and Their Flow Properties

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Preface

The report presented here contains as an introduction a general description of the properties of cohesive soils. Further it includes a lecture on the flow properties of soils, given at a meeting of the Rheological Section of the Swedish National Committee for Mechanics in May 1962. The lecture dealt with some results and discussion of new tests, as well as a summary of investigations on the same topic presented earlier (see Reprints and Preliminary Reports No. 4 of the Institute).

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SWEDISH GEOTECHNICAL INSTITUTE

ON COHESIVE SOILS AND THEIR FLOW PROPERTIES

by

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Abstract

This paper first gives a simple description of the composition, structure and certain properties of cohesive soils (in the first place Swedish sedimentary clays) from the aspect of soil mechanics. Then a report is given of a laboratory investigation made at the Swedish Geotechnical Institute, at which the flow properties of a large number of soil samples in remoulded state were studied by applying the fall-cone test and, to some samples, vane tests. The liquid limit according to Casagrande and the plastic limit were determined, and the Casagrande's flow curve was compared with that of the fall-cone method.

The paper is a continuation of an earlier report published by the author (KARLSSON, 1961), certain excerpts from which have been taken for the sake of completeness.

1. Introduction

Natural soils are more or less saturated and may, with respect to their composition, be divided into mineral soils and organic soils. The classification of the mineral soils is usually based on the grain size distribution and, geotechnically, also on the deformation and strength properties. Thus distinction may be made between gravel and sand soils (so-called friction soils), silts (composite soils) and clays (cohesive soils). Some organic soils are also included among the cohesive soils, in the first place muds ("dy" and "gyttja" soils) and amorphous peats ("dytorv").

The water-binding capacity of the soil particles is especially for cohesive soils of dominant importance for their general rheological properties. It is therefore of great value to study this feature and the methods for its determination.

2. Composition, Structure and General Rheological Properties of Clays

21. Composition and Structure

A natural clay may be described as a porous mass, the skeleton of which consists mainly of mineral particles, coupled by forces which are mainly of a, so called, cohesive nature, and whose voids are filled with liquid and often also partly with gas.

The grain skeleton (the solid phase) consists partly of granular particles of minerals (sand and silt), and partly of colloidal particles of clay minerals (mainly particles with grain size $\leq 2 \mu$). In addition, sedimentary clays in particular contain substances of organic origin in varying proportions.

The granular particles consist of fragments of rock and are subspherical or subangular in shape (small specific surface). The clay particles consist of transformed rock minerals (usually by weathering), chiefly feldspar. Bentonite, also included in the tests, consists mainly of volcanic ash. The clay particles are usually flaky in shape (large specific surface). A number of different clay minerals can be distinguished, the principal being illite, kaolinite and montmorillonite. Illite is dominant in most Swedish clays.

The substances of organic origin may be of a colloidal nature, e.g. "gyttja" substance and amorphous sulphides, mainly iron-sulphide. They may be present in such proportions that they become of essential significance for the physical properties of the clays.

For the sedimentary clays the structure of the grain skeleton is highly dependent on the sedimentation environment, chiefly the salinity and pH-value of the water.

How the grain skeleton is built up in detail is at present discussed. So, for instance, two theoretical main types of grain skeleton are referred to, viz., flocculated (loose structure) and oriented (denser structure) (see, e.g. LAMBE, 1960). However, a natural clay might consist of a matrix of chains or packets of colloidal particles interspersed with granular particles (EMERSON, 1959).

The structure of sedimentary clays varies, as a rule, depending on the variation of supply of material and flow conditions during the

sedimentations. This is especially valid for varved clays.

The liquid phase (pore water) contains large or small amounts of dissolved salts, colloiddally dissolved silicas, and often also humic substances, etc. The pore water is more or less closely connected with the solid phase and partly attached to the surface of the clay particles owing to their electrical charge (cf. HOLDRIDGE, 1953). The viscosity of the water is said to decrease with distance from the particle surface and vary between wide limits.

The gaseous phase. There are usually bubbles of air in the top soil layer ("dry crust") subjected to desiccation. Also at great depths clay may contain gas; besides air e.g. methane. The gas may occur as bubbles in the pore water, and may sometimes also form cavities in the clay. Since the gas is highly compressible, and more or less soluble in the pore water, its volume depends on the pore water pressure and temperature.

During the time that has elapsed since the sedimentary clays were deposited, the structure of the skeleton and the composition of the pore water have changed. The latter change is due mainly to e.g. the seepage of the subsoil water owing to pressure gradient and osmosis, diffusion, or chemical reactions. The grain skeleton has, for example, settled vertically by pressure of the overburden or in the dry crust by desiccation. Further, the mineral particles may have become cemented together by e.g. iron oxides, calcium carbonate, silicates or organic substance, often in the form of gels.

22. Consistence

221. Remoulding Characteristics

Natural undisturbed clays may be regarded as elastic solids; when worked, they become plastic as a rule, but, among others, so-called quick clays become liquid.

When an undisturbed clay (at water contents higher than the plastic limit) is remoulded, the clay becomes softer; it is insensitive. After the remoulding, part of the loss in strength is as a rule regained. This phenomenon is called thixotropy and is an long-term effect. Thus the sensitivity can be divided in a reversible and an irreversible component.

The sensitivity, S_t , is dependent on structural factors and water-binding capacity, and is defined as the quotient of the so-called undisturbed and the so-called remoulded shear strength in the undrained case (see Section 232). In Sweden the following classification ¹⁾ is sometimes used with respect to the sensitivity: Soils with $S_t \leq 5$ are called low-sensitive, with $S_t = 5 - 10$ medium-sensitive and with $S_t > 10$ highly sensitive. Quick clays include clays with a sensitivity $> 30-50$. Since the result of the determination of the strength of the remoulded clay depends on the method of testing, the values obtained of the sensitivity vary, particularly for quick clays. Values as high as 500 - 700 have been obtained for natural clays by the fall-cone method.

The thixotropy can be considered as a structural effect (cf MITCHELL, 1960). At the remoulding the flocculated, colloidal particles are - in an erratic manner - directed to a more parallel orientation. After the remoulding the stress conditions are then such that the particles strive to become re-arranged to a flocculated structure. In connection with the re-arrangement the adsorbed water structure and the distribution of ions in the liquid phase is also changed. Furthermore, other effects may have an influence.

By investigating a great number of sedimentary deposits, BOSWELL (1949) found that all the materials tested, with the exceptions of clear sands, exhibited thixotropic characteristics and further that many of the soils were rheopectic, i.e. gentle motion accelerated the rate of thixotropic hardening.

For thixotropic soils at a water content near the liquid limit (see below), DAY (1954, 1955) measured the soil moisture tension (the negative pore pressure or suction) after thixotropic stiffening and after remoulding. The results indicate that the tensions fall to a minimum value at remoulding and then gradually return to nearly the initial value. Also BISHOP (1960) has observed, when testing clay with plastic consistency, an increasing pore water suction in connection with the thixotropic stiffening.

An increase of the suction means that the skeleton tends to expand. The volume increase is counteracted by the capillary tensions, provided

1) Other classifications are used in other countries (see e.g. SKEMPTON & NORTHEY, 1952).

that the soil cannot absorb water.

The irreversible component of the sensitivity depends mainly on certain changes in the grain skeleton and the composition of the pore water which have occurred after the original sedimentation. The sensitivity may thus be dependent on the changes in the pore water and also be associated with rheodestruction, i.e. an irreversible breaking up of the cementation bonds. This is especially valid for quick clays.

Highly sensitive clays can thus be formed by the addition of dispersing agents, e.g. certain humates, carbonates or phosphates, SÖDERBLOM (1960); also changes in the pH-value are of importance. Hence, quick clay has been found, in some cases, below peat deposits (ACKERMANN, 1948). In marine sediments, a quick clay formation should presuppose a salt leaching (ROSENQVIST, 1946; see also G. HOLMSEN, 1938).

222. Consistency Limits

The consistency of a sample of remoulded clay can be varied by a change of the water content. If the content is increased, the distance between the particles is increased, and the strength reduced.

On the basis of these facts ATTERBERG (1916, see also 1911) evolved what he called principles of consistency for the classification of cohesive soils. He distinguished between four principal consistency ranges: firm, soft (doughy), semi-liquid, and liquid. The soft range was subdivided in three groups: semi-soft, dry-plastic ("torrplastisk"), and sticky plastic. Also the liquid ranges were subdivided. The ranges were distinguished by consistency limits. In practice, the following limits are now used:

The shrinkage limit, defined as the water content (in weight percentage of dry substance) at which a sample of clay (originally of soft consistency) stops shrinking when the water content is reduced; air finds its way into the pores at the same time.

The out-roll limit (nowadays usually called the plastic limit) defined as the lowest water content at which a clay specimen can be rolled out into a thread.

The sticky limit is defined as the lowest water content at which the clay adheres to a metal blade.

The percussion liquid limit (usually called the liquid limit) is defined as the lowest water content at which a specimen of remoulded clay cut in two has such a soft consistency that the two halves flow together when the specimen is subjected to percussion according to a particular method.

The difference between the liquid limit and the plastic limit, which was called the plasticity index by Atterberg, is to be a measure of the range of plasticity of the clay.

Atterberg's consistency limits (nowadays as a rule only the plastic limit and the liquid limit) are still commonly used for classifying cohesive soils, though with modifications because the Atterberg procedure was rather subjective and time-consuming (see below).

Instead of the percussion liquid limit, the so-called fineness number is now commonly used in Sweden (see further under Section 42).

The liquid limit, like the plasticity index, is to a certain extent a measure of the water-binding capacity of the soils. A treatise of the relation between this capacity, the hygroscopicity, and the liquid limit as well as the fineness number, has been performed by EKSTRÖM (1927). The liquid limit and plasticity index have become a valuable aid in soil mechanics for the estimate of the strength and compression properties of cohesive soils (e.g. SKEMPTON, 1953; OSTERMAN, 1960).

23. Deformation and Strength Properties

231. Compression Properties

When a sample of saturated clay is subjected to vertical pressure, and lateral expansion is prevented (confined compression), the excess load (over a certain value, called the pre-consolidation load) is first taken mainly by the pore water. A flow of pore water then arises, the magnitude of which depends on the pressure conditions and on the permeability of the soil. The volume of the sample reduces as the pore water squeezes out; the clay consolidates. The pore water pressure decreases successively and the external pressure is transferred to a corresponding degree to the solid phase.

When the pore water pressure is equilibrated, the so-called primary consolidation has been completed. The pressure is then borne wholly by the so-called effective stress (vertical pressure less pore water pressure). The time for primary consolidation is, besides on the pressure increase and the permeability (very small owing to the fine pores in the clay) dependent on the length of the drainage path. The consolidation theory was, as is well known, originally mathematically treated by Terzaghi.

Beside the primary effect, secondary consolidation occurs (cf. e.g. BARBER, 1961), which, as has been intimated in the literature, is not to be connected with measurable changes in the pore water pressure. The course of the secondary consolidation covers, as a rule, a very long time. The causes, however, are not fully known; and chemical changes, especially of the organic substance, may add to these effects. Compared with the primary consolidation, secondary consolidations are many times small, but may be considerable in the case of clays containing a high proportion of matter of organic origin.

The vertical compression seemingly causes an increased parallel orientation of the particles. This is to some degree proved by the fact that the permeability and electric resistance are different in vertical and horizontal directions.

A reduction of the vertical pressure on a confined clay specimen causes a momentaneous suction in the pore water, which is gradually equilibrated during a decrease in the effective stress, if the clay, as is usual, expands. This process is present especially when the clay has a possibility to absorb water. The degree of expansion depends in the first place on the mineralogical composition of the clay and the relative change in pressure, but there are a number of other contributory factors. During the expansion the grains tend to resume a previous orientation.

With reference to the primary time effects and the loading conditions, a clay can be called under-consolidated when consolidation is progressing, normally-consolidated when the clay is consolidated for the existing load, or over-consolidated when consolidated for a higher load than the present load.

If, in a confined compression test, water is prevented from entering the sample at unloading, hardly any expansion occurs, except that equivalent to the development of gas. The pore water under-pressure then chiefly remains and is taken by the capillary tensions. A certain amount of elastic deformation occurs, however. Depending on the fact that under the load conditions, the vertical stress is higher than the horizontal stress, the sample is then elongated vertically and contracted laterally. According to measurements made by BISHOP & HENKEL (1953), this deformation is connected with a decrease of the value of the pore water underpressure.

The above-mentioned circumstances are also valid for so-called undisturbed clay samples when extracted from the ground.

At compression of an unconfined sample of undisturbed clay, the sample is first deformed elastically to some degree. When the load reaches an ultimate value, failure with a distinct slip surface usually occurs in normal Swedish soft clays. In certain cases, especially for highly muddy ("gyttjiga") clays and very stiff clays, the failure can be brittle. For a worked clay there is as a rule no marked failure zone but the sample is plastically deformed.

The elastic properties of clay are dependent on several factors, particularly the organic ("gyttja") content and water content.

232. Shearing Properties

Resistance to shear deformation in a clay is to varying degree due to cohesion between the colloidal particles and friction between the granular particles, in natural clays also to the influence of e.g. cementation, the contributory parts depending on the loading and drainage conditions.

The colloidal interaction is of a complex nature (HOLDRIDGE, 1953) and PHYSICO-CHEMICAL PROPERTIES OF SOILS, A SYMPOSIUM, 1961). The bonds depend, among other things, on the type of the clay mineral and the chemical composition of the pore water.

The interaction between the particles is dependent on the distance between the particles and their orientation. The various forces attract or repel. The smaller the distance between the particles, the greater

will be the shearing resistance, though a parallel orientation of the particles may reduce the resistance.

In case of shearing, the particles tend to become parallelly orientated in the deformed zone. They adapt themselves, in case of failure, to an orientation that is fitted to the prevailing stress and deformation. Also the distances between the particles may be changed. A concept of the orientation at failure has been outlined by TROLLOPE & CHAN, 1960.

A change in density - alteration of the distance between the particles - depends on the normal pressure prevailing, and on drainage. Extreme cases are referred to as deformation in undrained or drained conditions, the latter case assuming that water can escape or be absorbed.

In the so-called undrained shear there should be no pore water drainage, e.g. because the deformation rate is high, and hardly any volume change occurs, assuming that the clay is saturated. At undrained shear of an undisturbed clay, a distinct failure usually occurs in connection with rheodestruction and is in case of quick clays accompanied by a collapse of the grain skeleton in the rupture zone. So-called undrained shear strength is almost entirely independent of the normal pressure, since possible changes in the pore water pressure make no contribution to the strength.

In the so-called drained case the rate of deformation must be so low that possible over- and under-pressures in the pore water will have time to diminish. Shear deformation is connected with an increase in volume (dilatation) if the clay is fairly over-consolidated in relation to the prevailing normal pressure, and a decrease in volume (negative dilatation or contraction) if it is under-consolidated. In cases of dilatation, failure occurs at a critical density, depending on the loading conditions, etc. At contraction the shear stress increases with greater deformation as long as contraction continues.

In intermediate cases, which are common in practice, it is a question of incomplete drainage during shearing, and the deformation is due either to dilatation with accompanying pore water suction, or contraction with pore water pressure.

The short-term shear strength is lower than the long-term for under-consolidated or normally-consolidated clays in the case of usual inorganic Swedish clays; for over-consolidated (in the natural case heavily over-consolidated) clays as well as in organic clays the opposite is common.

The shear strength problems have recently been discussed at the RESEARCH CONFERENCE OF SHEAR STRENGTH OF COHESIVE SOILS (1960), where some valuable contributions to the relevant question are given, e.g. by Hvorslev and by Seed, Mitchell & Chan.

3. Composition of Organic Soils

The organic cohesive soils consist to an important degree of organic matter, essentially decomposed.

Peat soils consist mainly of the remnants of plants that have decomposed on their original site. The organic matter in amorphous peat and "dy" soils is, as a rule, precipitated amorphous humus, while "gyttja" soils consist largely of decomposed algae, and also of substance of animal origin. "Gyttja" and "dy" soils are regarded as sedimentary soils, often containing a considerable amount of mineral particles. The organic colloidal substance, however, has great water-binding capacity, and is therefore of dominant significance for the general rheological characteristics.

What has been said above on the deformation and consistency properties of inorganic clays is, in principle, valid in its main points also for the organic cohesive soils.

Most organic soils have, however, a more or less fibrous structure, due to the fact that they contain vegetable fibres. At a deformation, strains are developed in the fibres. This effect is called "straightening of curled threads" by REINER (1960).

4. Report on Tests Made in Connection with the Determination of Flow Properties.

Atterberg's consistency limits have been determined for a large

number of soil samples, and consistency determinations made by the fall-cone method. Laboratory vane tests have been made on some typical materials.

The samples were chiefly of cohesive soils; in the first place Swedish sedimentary clays of various types, but also laterite, kaolin and bentonite, mud and amorphous peat. In addition, some mineral composite soils have been examined, primarily silt.

The samples of laterite consisted of natural laterite soil from Liberia, while the kaolin and bentonite samples were of normal commercial quality. The samples of kaolin and bentonite were dried when bought; practically all the other samples had from the start their natural water content.

41. Atterberg Consistency Limits

The plastic limit was determined according to Atterberg except that the thread of soil was rolled out to a diameter of 3 mm (according to TERZAGHI, 1926).

The percussion liquid limit was determined according to CASAGRANDE (1932). In Casagrande's method a special apparatus is used, which is shown in principle in Fig. 1. In the test the sample is remoulded in the bowl with the help of a spatula whereupon the bowl is hung up on the pin on the stand. Then the surface of the sample is smoothed and a groove is made through the sample with a special tool. When the crank is turned, the bowl is lifted 10 mm by the eccentric on the axle, after which it falls freely onto the ebonite base plate. When the groove is closed (half an inch), the number of blows is recorded and the moisture content is determined. Tests are made at some different moisture contents of the specimen (both in plastic and liquid state).

The relationship between the moisture contents, w , and the number of blows, N , is plotted semi-logarithmically (N in the log scale). The curve (called the flow curve) may, according to Casagrande, be represented by the following equation:

$$w = F_I \cdot \log N + C \quad (1)$$

where F_I = constant, called the flow index
 C = constant

Thus the flow curve is, when drawn semi-logarithmically, a straight line. The number of blows may, according to Casagrande, be taken as proportional to the shearing resistance ($\bar{\tau}$) of the soil. The liquid limit is defined as that moisture content which corresponds to $N = 25$ in the flow curve. (Nowadays so-called one-point methods are commonly used.)

42. Fall-Cone Method

The fall-cone method was developed by the GEOTECHNICAL COMMISSION OF THE SWEDISH STATE RAILWAYS 1914-1922 (1922). The cone apparatus is shown in principle in Fig..2. --

The fall-cone method (referred to below as the cone method) implies that a metal cone of a certain weight and with a certain apex angle is suspended over a horizontally levelled sample with the point barely touching the surface. The cone is allowed to drop into the sample, and the depth of the impression gives a measure of the cohesion of the soil.

The cone apparatus is provided with test cones weighing 100 grams, 60 grams and 10 grams with cone angles of 30 degrees, 60 degrees and 60 degrees, respectively.

The Commission introduced a strength number. It was assumed that the strength at a constant cone impression is directly proportional to the weight of the cone, i.e. to the external work required to produce the impression. The 60 gr - 60° cone was chosen as the standard cone. A 10 mm deep impression with this standard cone was given the strength number 10.

The symbol H_1 is assigned to the strength number of the remoulded soil, and H_3 indicates that of the undisturbed soil. The quotient $\frac{H_3}{H_1}$ is a measure of the sensitivity of the soil.

For determining the liquid limit the cone method is an objective and accurate method. This limit was called the "fineness number" by the Commission. It is defined as the moisture content (in percentage of the dry weight) at which the strength number of a remoulded soil sample (H_1 -number) is 10.

A new, more theoretical interpretation of the cone method has been made at the Swedish Geotechnical Institute (HANSBO, 1957). It was here found, under the given experimental conditions, that the relation between

the undrained shear strength ($\tau_{f \text{ cone}}$) and the depth of penetration (h) of a cone with the weight Q approximates the following formula:

$$\tau_{f \text{ cone}} = k \cdot \frac{Q}{h^2} \quad (2)$$

where k depends chiefly on the apex angle of the cone. The value of k was determined by calibration with the field vane. ¹⁾

In the tests made by the present author, the cone method has been evaluated according to Eq.(2) for the determination of shear strength.

The cone tests were made at different water contents; the lowest in the vicinity of the plastic limit and the highest at a semi-fluid consistency considerably higher than the liquid limit.

The kaolin and bentonite were mixed with distilled water until the moisture content of the samples approached the liquid limit, and the samples were then tested. After remoulding, the other samples were first tested at their natural moisture content. Each sample was then divided into two parts. The moisture content of one part was reduced successively and that of the other increased. The moisture content was reduced by spreading the sample on a plaster-of-Paris slab, and increased by adding distilled water.

The cone tests were made in a bowl on a partial sample remoulded with a spatula. The water content was determined before and after the tests. In addition to the cones standardized by the Geotechnical Commission, two other cones were employed, one 400 gr - 30°, the other 15 gr - 30°.

4.3. Laboratory Vane Method

Tests have been made using the laboratory vane method on bentonite, postglacial clay, mud ("gyttja") and coarse silt with some organic matter.

The vane apparatus used is shown in principle in Fig. 3. Three different vanes with diameters of 1.5, 3.0 and 4.5 cm, respectively, all with a height of double the diameter, were used in the tests. The internal diameter of the sample container was 5.5 cm and the height 17 cm.

1) Further studies on the relationship between strength of soil and cone penetration have been made at SGI and will be published in another connection.

Every sample was studied at different moisture contents, both lower and higher than the liquid limit. For each moisture content a series of tests was made at different rotational speeds. The speed of rotation could be varied as described in connection with the tests performed (see Section 62).

After the sample had been kneaded in a machine, it was put into the container and remoulded with a spatula. The table was raised until the upper surface of the vane was 3 cm below the upper surface of the sample. The motor was started, and the strain indicator was read at definite intervals of time during about one revolution. In several tests the table was then disconnected and the container rotated about 50 revolutions ("rotating" remoulding) by hand, after which the test was repeated. Before and after the test, cone tests and determinations of moisture content were made. Before the speed was altered, the table was lowered and the sample was remoulded with a spatula ("normal" remoulding).

In Fig. 4 is a photograph of a typical surface of rupture at a vane test. This shows that the deformation are concentrated to the very thin rupture zone.

The shearing resistance τ_f for the vane test has been evaluated according to the following formula:

$$\tau_f = \frac{6}{7} \cdot \frac{M_{\max}}{\pi \cdot D^3} \quad (3)$$

The surface of rupture has thus been assumed to be a cylinder with the same diameter, D , and height, $2D$, as the vane (CADLING & ODENSTAD, 1950). When using the formula (3) a constant value of the shearing resistance over the whole failure surface is assumed. If the resistance is dependent on magnitude and rate of strain, the formula is not fully adequate.

5. Result of Tests with Fall-Cone Method and Casagrande's Apparatus

In the evaluation of the cone tests, the factor k in Eq (2) has been put at 0.03 for 60° cones and 0.08 for 30° cones (see below, Section 7).

The undrained shear strength, $\bar{\tau}_{f \text{ cone}}$ (in log scale) has been plotted against the water content, w (in arithm. scale). Fig. 5 shows such log $\bar{\tau}_{f \text{ cone}}$ versus w curves (consistency curves) for a number of different types of soils. In addition, the fineness number, w_F , the percussion liquid limit, w_L , the plastic limit, w_P , and the natural water content, w_n - where this has been determined - are also shown. w_F has been indicated at the water content which, in the consistency curve, is equivalent to $\bar{\tau}_{f \text{ cone}} = 1.8 \times 10^{-3} \text{ kg/cm}^2$ (10 mm impression with the 60 gr - 60° cone). Starting from the slope of the consistency curve at the fineness number, a one-point method for the determination of the fineness number is developed and described in the earlier paper by the author (KARLSSON, 1961).

The relation log $\bar{\tau}$ versus w is according to Casagrande, as mentioned above, a straight line. The analogous relationship according to the cone method (the consistency curve) however, is for clays more or less bent in the vicinity of the liquid limit, as shown in Fig. 5.

Some typical values of the fineness number are plotted against the percussion liquid limit in Fig. 6. The values correspond well, with certain exceptions.

For originally very sensitive clays, as well as for bentonite and organic soils, the fineness numbers are smaller than the percussion liquid limits. For silts, on the other hand, the fineness numbers are greater, probably due to dilatation and higher permeability than in the case of clay; in determinations made by the percussion method, the surface of the sample is enriched with water and becomes liquid. For silts, dilatancy may have an influence also in the cone test.

If the liquid limit defines a definite strength, it is dependent upon the method used. For comparison, the shearing resistance at the fineness number and at the percussion liquid limit, respectively, has therefore been determined by the laboratory vane. The shear strength according to the vane test at the percussion liquid limit varies widely for the different soil types while the variation at the fineness number is smaller (see Table 1).

6. Results of Vane Tests

61. Stress versus Angular Strain Curves

The stress, τ (in log scale), has been plotted against the angular strain, γ . Some curves for bentonite and postglacial clay are shown in Fig. 7; those in the left part of the figure illustrate "normally" remoulded samples, and those in the right part "rotation" remoulded samples. Curves for kaolin, mud ("gyttja") and coarse silt are reported in the paper by KARLSSON (1961) and are not shown here.

Tests made on "normally" remoulded bentonite, postglacial clay and coarse silt show that τ , after attaining a maximum, declines with γ and approaches an asymptotic value. The tests made on "normally" remoulded kaolin and mud show that τ declines with γ only when the water content is lower than the liquid limit. The lowering of the shearing resistance is probably due to particle orientation as well as to volume changes in the failure zone (cf. Section 62) and to some extent also to thixotropic effects.

After the samples had been "rotation" remoulded, the flow was practically stationary.

62. Shearing Resistance versus Angular Velocity

The maximum shearing resistance for "normally" remoulded material, τ_{fn} , and for "rotation" remoulded material, τ_{fr} , has been plotted as a function of the rate of rotation, ω (in log scale). In Fig. 8 some typical curves on τ_{fn} versus $\log \omega$ and τ_{fr} versus $\log \omega$ are shown for tests at which ω could be varied between about 60°/min. and 30,000°/min. Normally τ_{fn} has a minimum (τ'_{fn}) at rotation rates of 100°/min. to 200°/min. At the same value of ω , τ_{fr} is designated τ'_{fr} . After the minimum the τ_{fn} values are, in some cases only within a limited region, largely clustered round a straight line (the B-B line). Especially at water contents lower than the liquid limit, τ_{fr} deviates still more from τ_{fn} when ω increases within the "B-B range".

Fig. 9 gives curves of the τ_{fn} versus $\log \omega$ for bentonite and kaolin obtained in tests made when the speed could be varied between 1.6°/min. and 800°/min. As shown in the figure, the shearing resistance is changed in a variable manner with speed. This is probably because of t

the complexity of the resistance which is influenced by different components - thixotropic effects (hardening), structural viscosity (particle orientation and collisions etc.), volume changes by dilatancy etc.

To get an idea of the relation between viscosity (at failure) and water content of the materials, the quotient $\Delta \tau'_{fn} / \Delta \log \omega$ (the values taken on the B-B line) has been used as a measure of the "apparent viscosity". The quotient $\Delta \tau'_{fn} / \Delta \log \omega$, as well as τ'_{fn} and τ'_{fr} (all in log scale) have been plotted against the water content, w . In Fig. 10 the curves for kaolin and bentonite are shown. As appears from the figure, the curves are falling with increasing w . With the help of the curves obtained the values are taken for the relationships given below.

Values of $\Delta \tau'_{fn} / \Delta \log \omega$ have been plotted against the plasticity index I_p of the materials, Fig. 11 a. I_p has here been put equal to $w_F - w_P$. The water contents, w , of the materials were chosen as follows (three points):

$$\begin{aligned} w \text{ at } \tau'_{fn} &= 0.05 \text{ kg/cm}^2 \\ w &= w_F \\ w \text{ at } \tau'_{fr} &= 0.005 \text{ kg/cm}^2 \end{aligned}$$

Furthermore, the values of the quotient $\frac{\tau'_{fn}}{\tau'_{fr}}$ at the above water contents have been put in relation to I_p (Fig. 11 b). $\frac{\tau'_{fn}}{\tau'_{fr}}$ can be used as a measure of an "apparent sensitivity" in the range of remoulding of the "normally" remoulded material. It can be seen from the figures that, for water contents lower than the liquid limit, the value of $\Delta \tau'_{fn} / \Delta \log \omega$ as well as τ'_{fn} / τ'_{fr} for the material first decreases with increasing value of I_p . This indicates that the granular part of the material (the friction) has a considerable influence on the "apparent viscosity" and the "apparent sensitivity".

The relation between τ' and ω depends on several factors, discussed previously. However, volume changes seemingly occur in the failure zone during the rotation, indicating a certain water transfer. These changes

may result in changes in friction and water pressure. The occurrence hereof is connected to the state of density of the clay in relation to some critical water content. This phenomenon seems to be present especially at water contents lower than the liquid limit.

7. Comparison between the Cone Method and the Laboratory Vane Method

For the evaluation of the cone method τ'_{fn} , according to the vane tests, has here been assumed to represent the short-term shear strength of the remoulded clay. In Eq. (2), k has thus been determined by plotting τ'_{fn} against the strength parameter $\frac{Q}{h^2}$ for 30° and 60° cones. These calibration diagrams have been given earlier for all the materials investigated by the vane method (KARLSSON, 1961). The values of k obtained have here been collected in Table 2. According to the test results, the value of k varies to some degree with the soil proper (both with the kind of material and with the water content) and also to some degree with the cone weight (cone impression).

To investigate the variation with the cone weight and the kind of material, tests have been made with a series of cones of different weights with 60° and 30° apex angles. Tests were made on a number of type samples (high plasticity to low plasticity), with water contents corresponding approximately to the fineness number and with lower water content.

Fig. 12 shows the results of these tests, whereby the strength parameter, $\frac{Q}{h^2}$ (in log scale), has been plotted against cone impression, h . As the figure shows, the test values for most samples are clustered round a curve, which approximates a horizontal line when h exceeds 6 to 8 mm. As exceptions we note the results from the silt and the kaolin tested with 60° cone (the kaolin sample being of low plasticity). Thus, k is not constant within any part of the range investigated. The following conclusions may be drawn from the results of the tests.

For cohesive soils, Eq. (2) should not be applied using a constant value of k , if the cone impression is less than ca 7 mm. For composite soils the cone method, especially when the 60° cone is used, gives uncertain results.

The variation of k with the water content is usually small for clays in the plastic state, but it is of importance if the water content is so high that the consistency tends to be liquid.

Eq. (2) should in fact be revised for, among other things, the uplift of the displacement of clay. Thus a more adequate value can be reached by reducing the cone weight for the value of the uplift.

Regarding the soil as a suspension, and applying Archimedes' principle, we get

$$U = \frac{0.1 w + 1}{0.1 w + 1/G_s} \cdot \frac{Q}{3} \cdot (\tan \beta / 2)^2 \cdot \left(\frac{h}{10}\right)^3$$

where U = uplift pressure

β = apex angle of cone in degrees

G_s = specific density of dry substance

A rough correction of Eq.(2) can be made by reducing the cone weight Q with the static uplift U , hence

$$\tau_{f \text{ cone corr}} \approx k \frac{Q - U}{h^2} \quad (4)$$

For control, cone tests with the 10 gr - 60° cone and vane tests have been made on a clay with a number of different water contents at very soft consistency. The uncorrected and corrected τ_f -values obtained in this way by the cone tests have been plotted in relation to the τ_f values obtained by the vane test (Fig. 13). As shown by the figure, the uncorrected $\tau_{f \text{ cone}}$ -values, at cone impressions \geq ca 10 mm, deviate from the $\tau_{f \text{ vane}}$ -values. The corrected $\tau_{f \text{ cone}}$ -values on the other hand are in good agreement with the $\tau_{f \text{ vane}}$ -values.

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Table 1. Apparent shear strength for various soils according to laboratory vane test at percussion liquid limit and fineness number.

Sample No.	S o i l	Percussion liquid limit w_L per cent	Fineness number w_F per cent	Min. shearing resistance Laboratory vane test	
				at w_L kg/cm ²	at w_F kg/cm ²
78	Bentonite	320	170	0.005	0.015
79	Kaolin	52.5	55.5	0.025	0.016
80	Coarse silt with some organic matter	30	33.5	0.042	0.021
81	Postglacial clay	70	61.5	0.007	0.016
84	Mud	275	215	0.005	0.015
90	Kaolin	45	43	0.015	0.020

Table 2. Values of k for various soils when using the equation $\tau_f = k \frac{Q}{h^2}$ and 30° and 60° cones.

Sample No.	S o i l	$k^{1)}$	
		30° cone	60° cone
78	Bentonite	0.025	0.086
79	Kaolin	0.027	0.078
80	Coarse silt with some organic matter	0.035	0.070
81	Postglacial clay	0.027	0.080
84	Mud	0.026	0.078
90	Kaolin	0.033	0.080

1/ For τ in kg/cm²

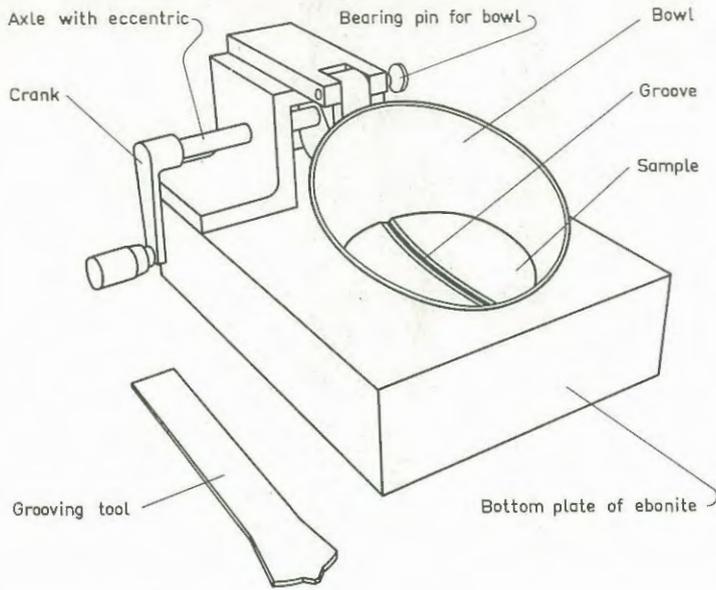


Fig. 1. Casagrande's apparatus for determining the liquid limit, in principle.

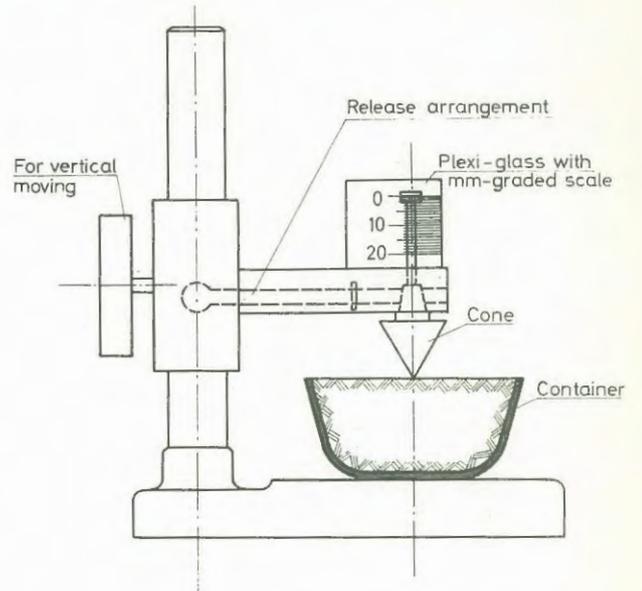


Fig. 2. Fall-cone test apparatus, in principle.

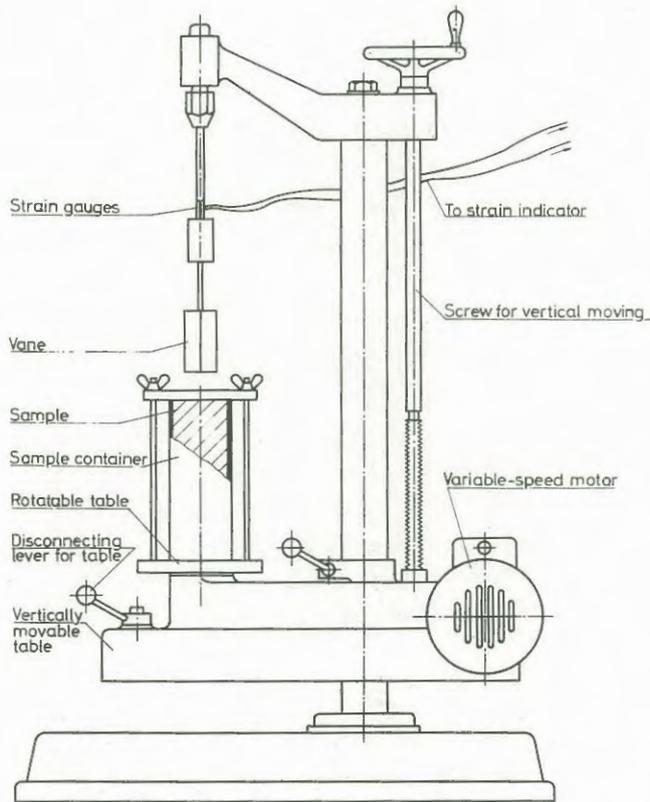


Fig. 3. Laboratory vane test apparatus, in principle.

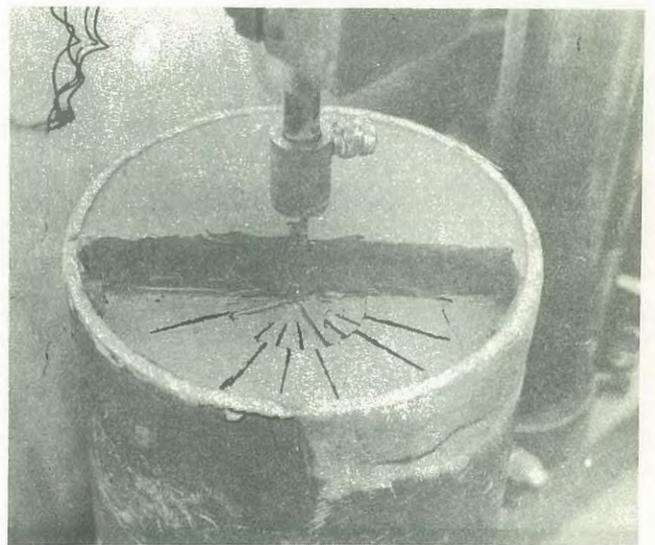


Fig. 4. Surface of rupture at laboratory vane test. Front part of the sample cut out to the level of the upper part of the vane.

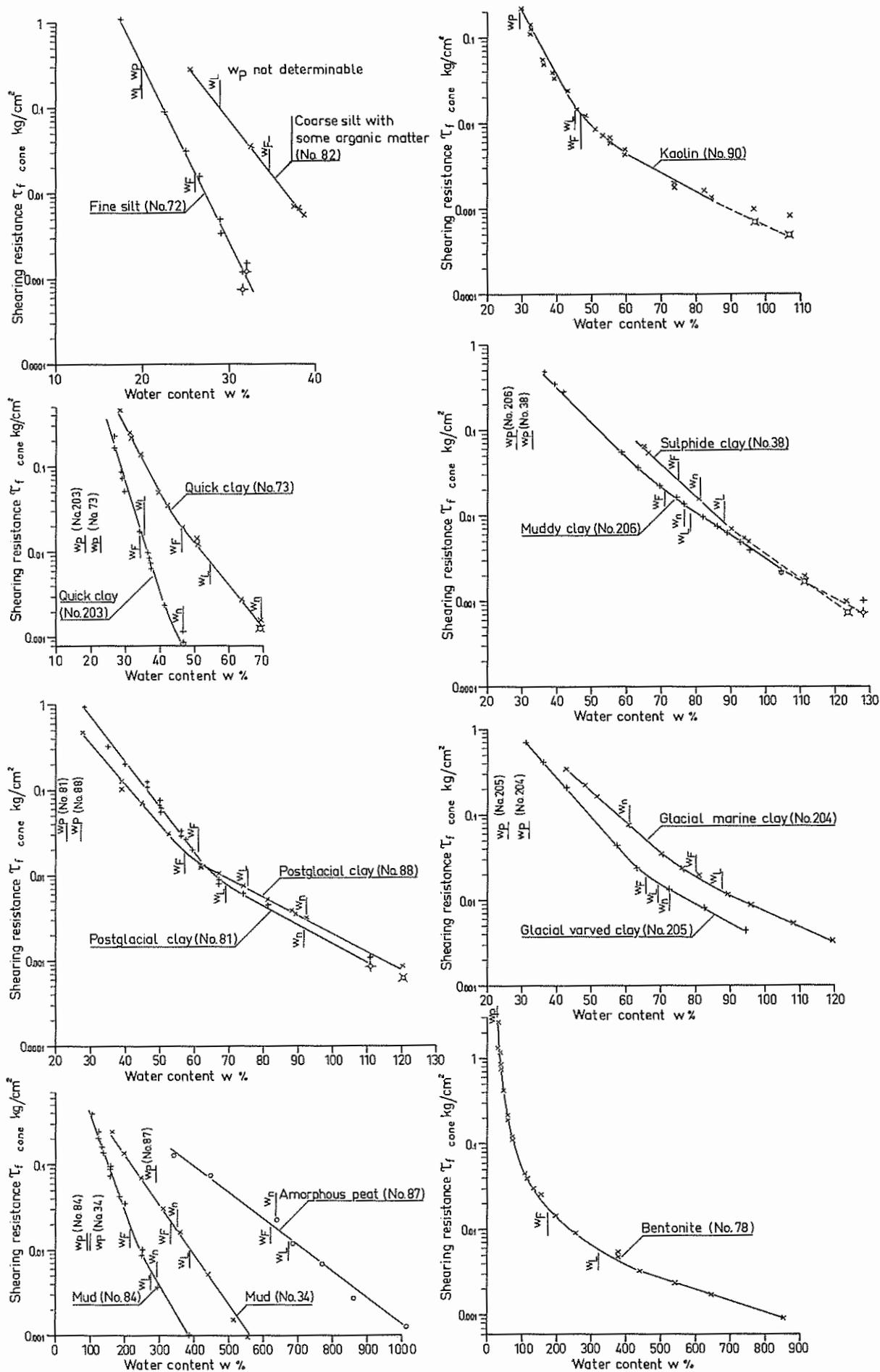


Fig. 5. Consistency curves for various soils (crosses with circles indicate corrected values according to Eq. (4)).

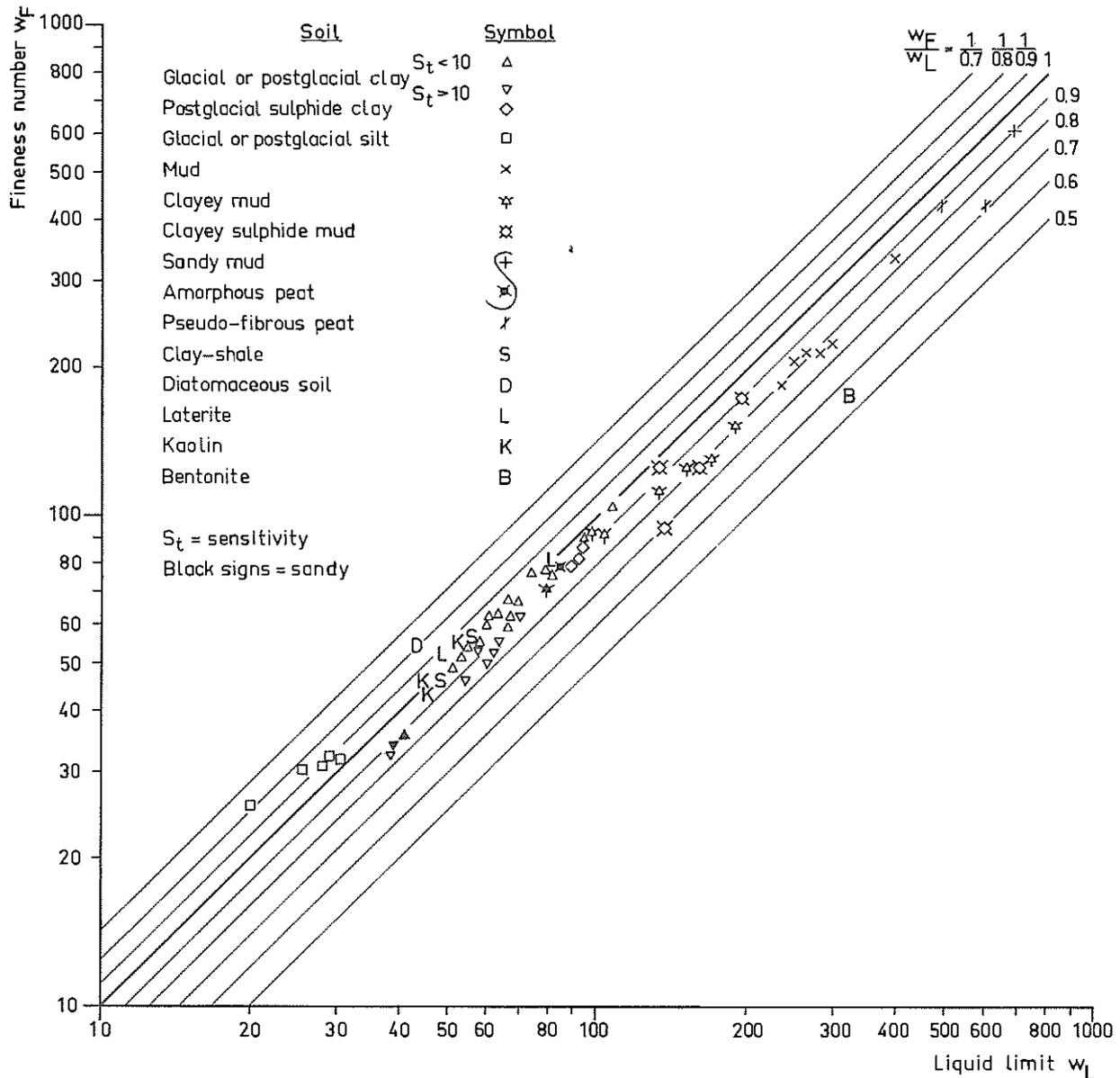
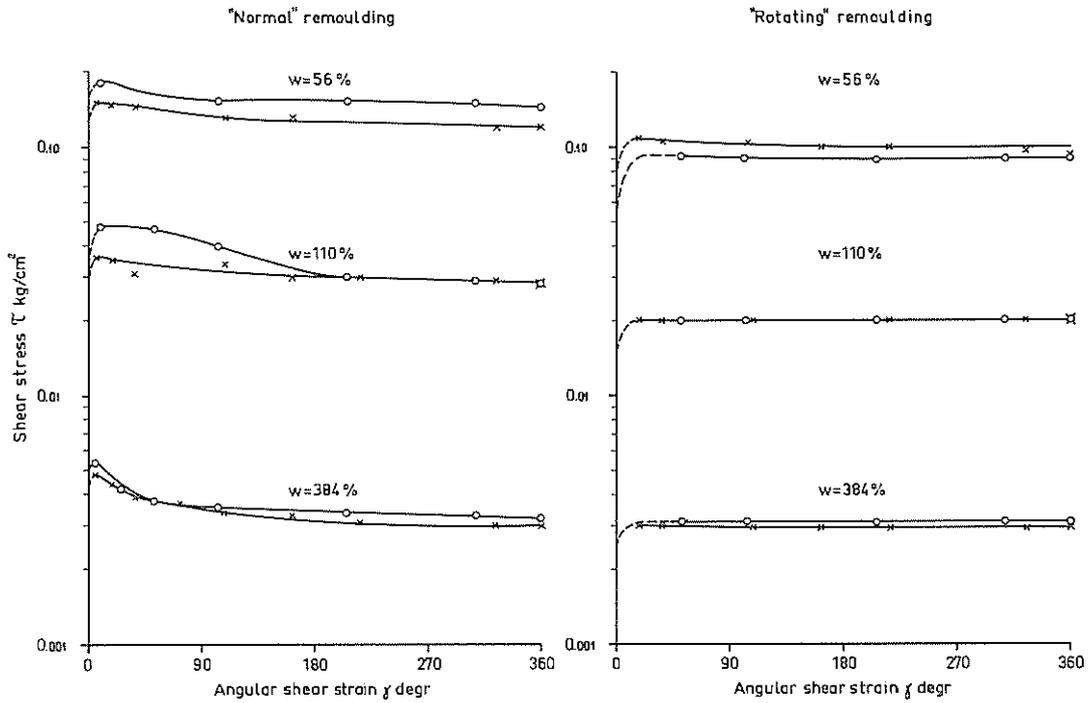


Fig. 6.
Comparison between the liquid limit according to Casagrande and the fineness number.

Bentonite (No.78)

$w_F = 175\%$

Angular velocity	Symbol
210 degr/min	x
610 degr/min	o



Postglacial clay (No.81)

$w_F = 61\%$

Angular velocity	Symbol
120 degr/min	+
610 degr/min	o
1080 degr/min	Δ

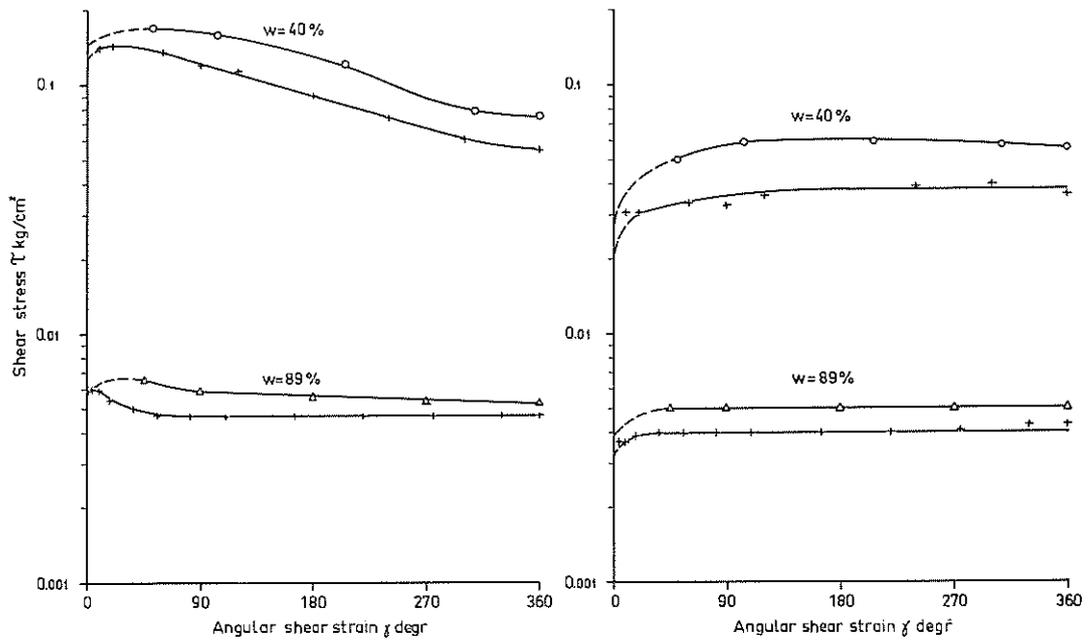


Fig. 7.
Typical stress vs. angular strain curves for bentonite and post-glacial clay.

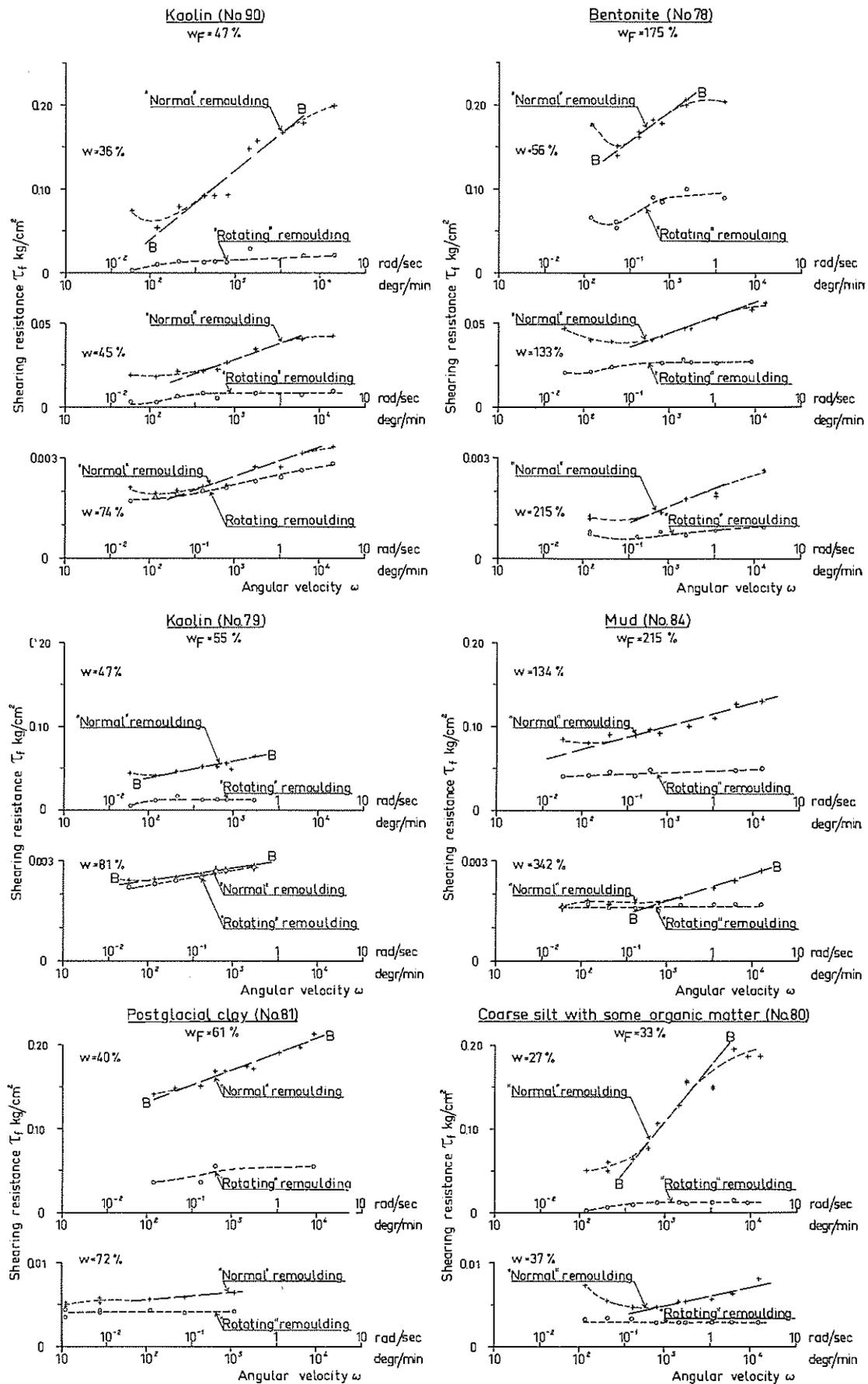


Fig. 8.
Typical shearing resistance vs. angular velocity curves for various soils at «normal» remoulding and «rotating» remoulding and at angular velocities between about 60°/min and 30,000°/min.

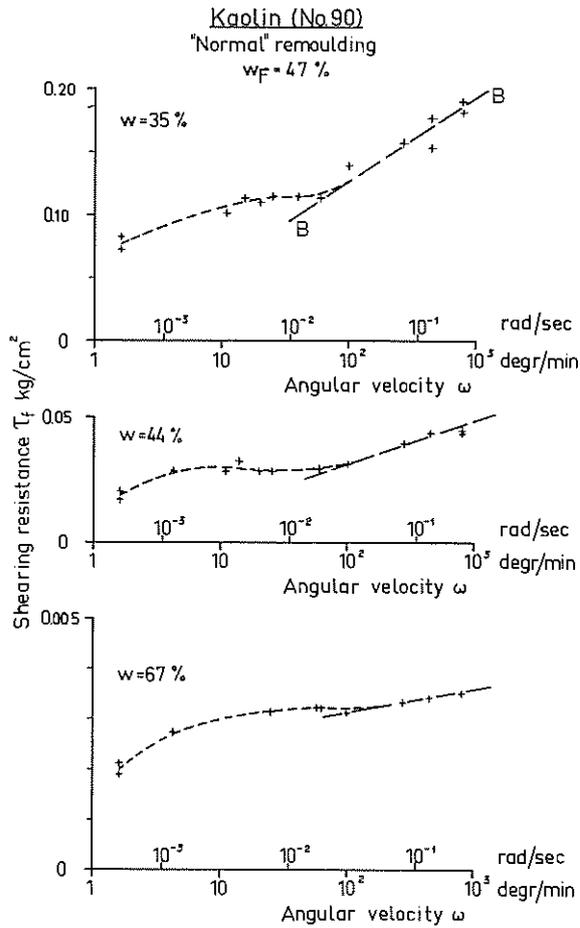


Fig. 9. Typical shearing resistance vs. angular velocity curves for kaolin and bentonite at normal remoulding and at angular velocities between $1.6^\circ/\text{min}$ and $800^\circ/\text{min}$.

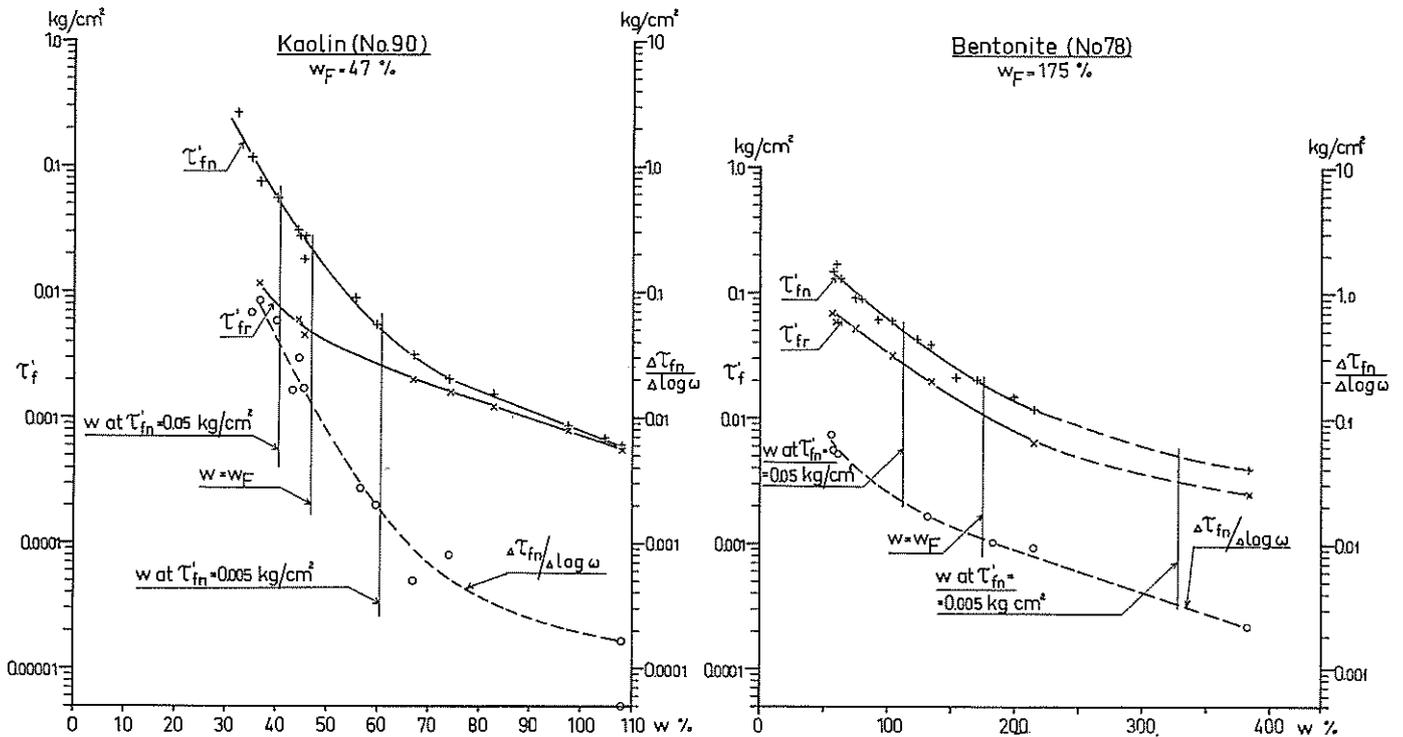


Fig. 10. The quotient $\tau'_{fn}/\Delta \log \omega$ and τ'_{fn} and τ'_{fr} versus water content for kaolin and bentonite.

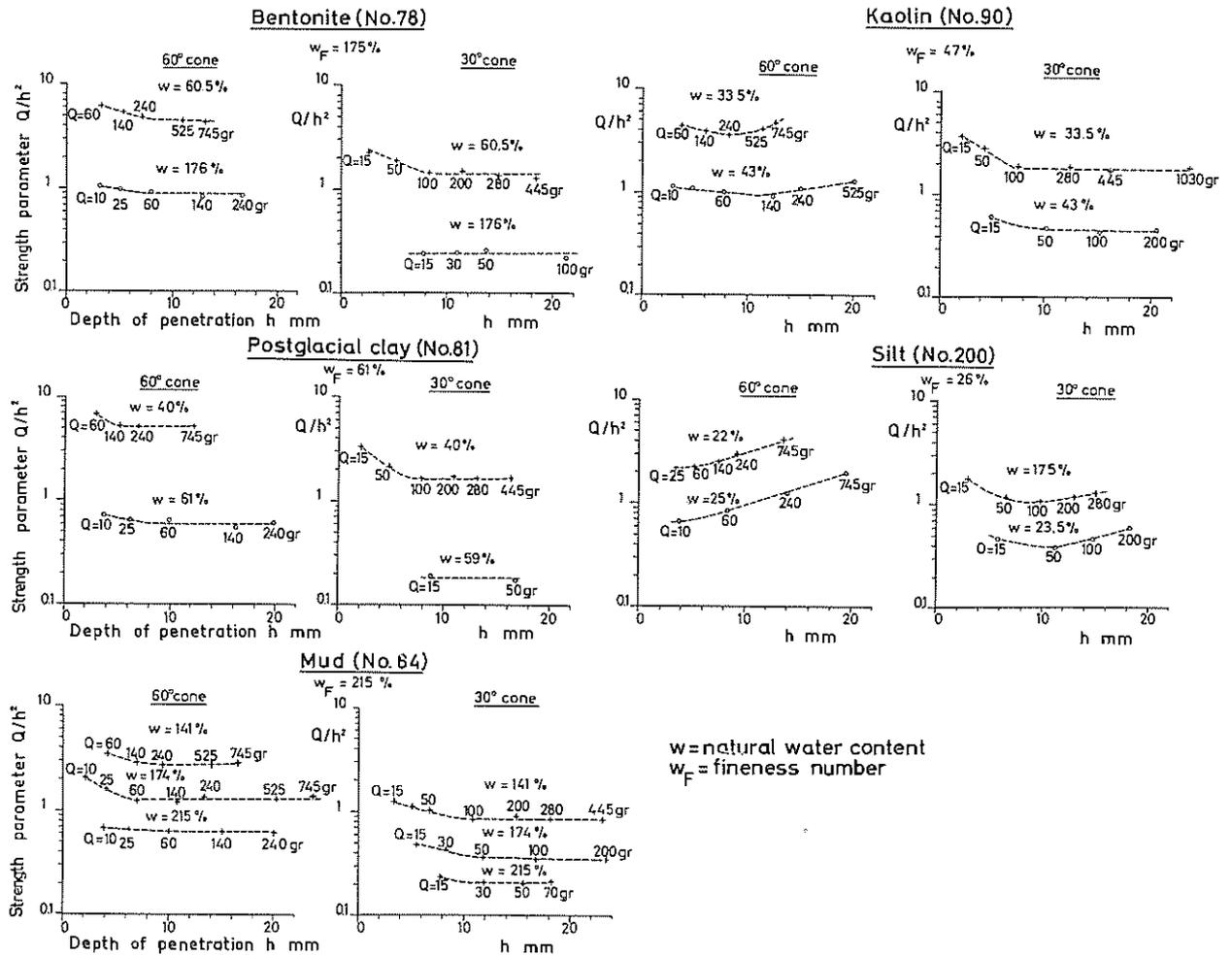


Fig. 12. Tests on remoulded samples with fall-cones of varying weights, Q , for determination of the relationship between the strength parameter Q/h^2 and the depth of penetration, h .

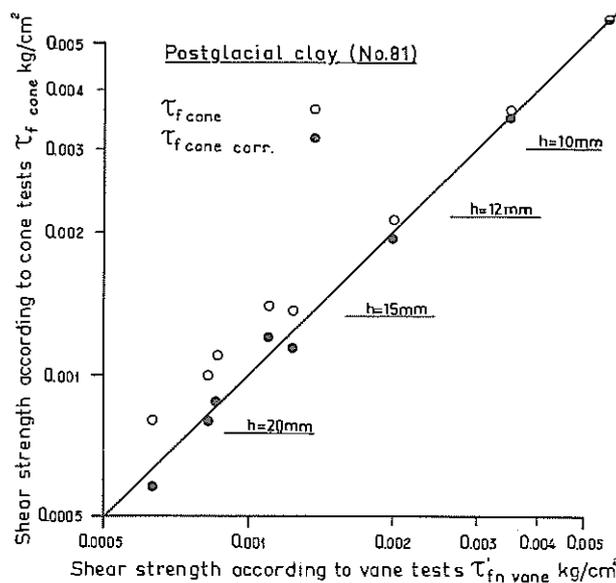


Fig. 13. Relation between shear strength values from cone tests and from laboratory vane tests for a clay at high water contents. For the cone tests, normal values and values corrected for the uplift pressure are given.