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IN SITU SOIL-INVESTIGATION METHODS FOR SOIL MECHANICS AND FOUNDATION ENGINEERING IN DELTA REGIONS

P. LUBKING[1]

ABSTRACT

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There now exists a great variety of investigation methods and measurements which can be applied in the field to derive appropriate and reliable information from the soil strata with respect to their behaviour as a foundation for civil engineering constructions.

The basic principles of in situ measurements in deltaic areas and especially in The Netherlands are given and some of the most interesting and recently developed devices are described.

Although every apparatus causes a certain disturbance of the soil strata, in practice the measuring results often appear to be sufficient and serviceable.

INTRODUCTION

From the earliest times when man settled in delta regions he was confronted with soil mechanics or, rather, with foundation-engineering problems. The passability of the freshly deposited sedimentary soil, together with the adverse foundation conditions for the constructions of his dwellings and dykes, compelled him to gather essential knowledge concerning the mechanical properties of these soils, often consisting of fine sand and of compressible clay and peat deposits with poor loadbearing capacity.

If it is furthermore considered that these soils are in many instances situated below the water level, either permanently or during a substantial part of the year, and that, even if they are above water level, their water table is at most only a few metres below the surface of the ground, it will be obvious that development of soil-mechanical and foundation-engineering knowledge was indeed essential.

The central aim of such efforts was to obtain information on the structure and properties of the subsoil. And as there was no technically advanced apparatus available for soil exploration in those early days, it was endeavoured to obtain the required information by simple and readily applicable methods:
(1) a hole was formed in the ground in some way, and the material excavated from it was tested and/or examined, as was also the soil profile exposed on the sides of the hole;
(2) or the soil was subjected to direct testing as regards its foundation-engineering properties, e.g. by the application of test loads to particular larger or smaller areas or by forcing or driving rods or poles into the ground until they encountered more resistant strata, i.e. possessing better bearing capacity.

As a result of gradually improving these primary and primitive exploratory techniques and by replacing the qualitative and relative methods by others that could yield measurable quantitative results, a considerable number of serviceable *in situ* exploration methods were developed in course of time and applied in actual practice. This process of development has been especially vigorous in recent decades (ANONYMOUS, 1975, 1977).

All these methods are based on the above-mentioned simple and obvious procedures and are aimed more particularly at obtaining insight into the mechanico-physical properties of soil, either by direct measurement or, via sampling, by testing

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[1] Laboratorium voor Grondmechanica, Postbus 69, 2600 AB DELFT, The Netherlands.

in the laboratory. By mechanico-physical properties is to be understood more particularly the stress-strain behaviour of soil, whether or not related to the water present in the pores, since any change in the volume of water-saturated soil brings about an inflow or outflow of pore water, this flow being governed by the permeability properties of the soil.

The *in situ* soil investigation methods envisaged here possess, however, two rather unattractive properties: they yield strictly *local* soil mechanics *information,* and to a greater or less extent they *disturb* the original *suboil* with regard to its structure and state of stress.

Besides these techniques, so-called geophysical methods have been developed in the last few decades. These are much less localized in their exploratory character and do not cause disturbance of the subsoil under investigation. On the other hand they yield only very approximate information in terms of soil mechanics and foundation engineering as compared with the soil-mechanical methods of exploration. The geophysical methods will not be considered in this article. Currently used *in situ* exploratory techniques in soil mechanics can be subdivided into three basic types:

(1) Type I, in which a shaft is formed in the ground in such a way that the column of material in the shaft is representative of the *in situ* condition of the soil. Examples of this principle are, essentially, all boring or drilling methods which extract acceptable samples from the soil.

(2) Type II, in which a shaft is formed in the ground in such a way that the environment of the shaft reveals the *in situ* condition of the soil. The shaft wall then serves as the object of measurement for determining the stress-strain behaviour, the density and the permeability properties.

(3) Type III, in which the necessary forces and/or moments are measured in the course of progressive movement of a test body driven or thrust or rotated into the ground; the most obvious and most widely employed form of test body is the rod with a conical tip.

The forms in which the three above-mentioned types of exploratory techniques are applied for soil mechanical and foundation engineering purposes in delta regions are of course largely governed by the relevant boundary conditions:

(1) Soil mechanics and foundation engineering require the most accurate possible information on the soil properties. This involves optimum measurement *in situ* of stress-strain behaviour, porewater behaviour and/or density and state of stress *or* the extraction of soil samples of optimum quality for determining the stress-strain behaviour and porewater behaviour subject to boundary conditions as regards density and state of stress.

(2) The depth to which information as envisaged above is required or desired is relatively small. Depending on the dimensions of the structure to be founded and especially depending on the nature and magnitude of the loads to be transmitted to the subsoil, this depth will in most cases not exceed a few tens of metres.

(3) The soil to be investigated generally consists of recently deposited sand, clay and/or peat strata and can be designated as soft soil.

(4) The water table is located at a depth of at most only a few metres below ground level. Also, it is necessary to take due account of the possibility that the foundation soil to be investigated is under water. Besides water depths ranging up to some tens of metres, additional obstacles exist in the form of tidal movements, currents and wave action.

TYPE I: THE REMOVED COLUMN AS TEST MATERIAL

The column of soil which is extracted in one way or another from the shaft formed in the ground gives, in the first place, general information on the composition of the soil strata concerned. In order to judge the soil with regard to its soil-mechanical and foundation-engineering properties, it will be necessary to carry out laboratory testing of samples obtained from this column. Obviously, the degree of disturbance, or absence of disturbance, will significantly affect the result of such sample testing.

The boring or drilling system employed, and the manner of execution of any particular boring technique, will in general determine the quality or degree of disturbance of the samples.

Of the numerous existing methods of boring only a limited number are somewhat frequently applied in The Netherlands' delta region. They can be subdivided into three categories:

(1) Methods in which a hole is first formed to the depth from which the sample is to be obtained, while the borehole serves as a transport passage for the extraction of the sample. Examples of this principle are afforded by shell and auger borings and by Ackermann borings (bailer borings with continuous sampling).

(2) Methods in which flushing with water is an essential feature of the system: water is jetted into the borehole under considerable pressure and carries soil particles to the surface of the ground, where it flows out of the borehole casing. Examples are wash borings, core borings and rotary borings.

(3) Methods in which no hole is first formed, but in which a sample of predeterminable length is extracted in a single operation. Examples are Begemann (continuous) borings, vibratory borings and piston sampler borings.

In the methods envisaged in category (1), which are very extensively used in The Netherlands' delta region, the manner of execution of the boring is of decisive influence on the quality or the degree of disturbance of the samples obtained. BEGEMANN (1977-a, 1979) showed that if, as often occurred, the water level in the borehole casing was lower than the surrounding groundwater level, the quality of the borehole profile and the undisturbed state of the samples would be ad-

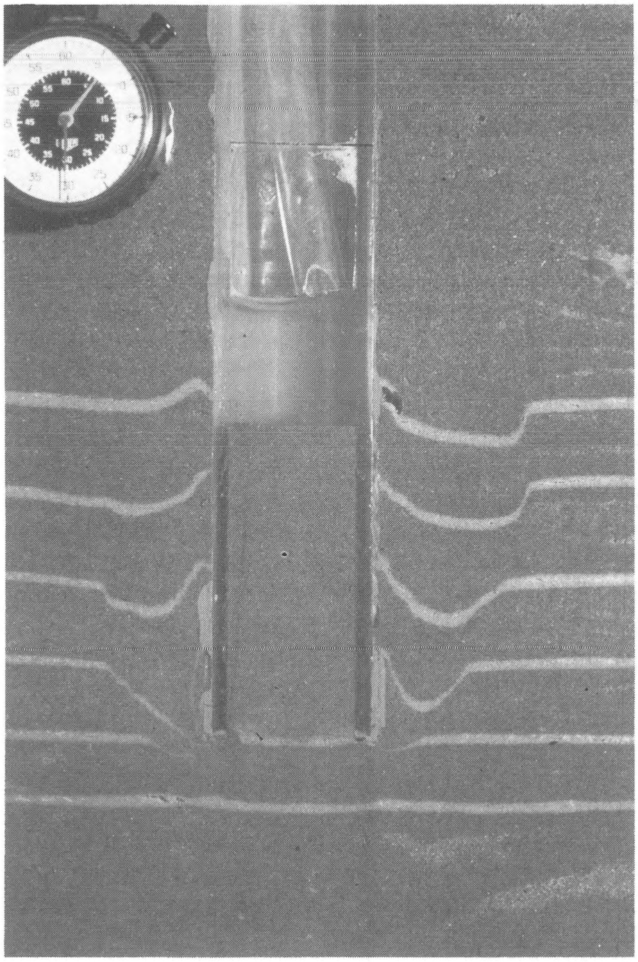

Fig. 1
Bailer boring in clean sand.

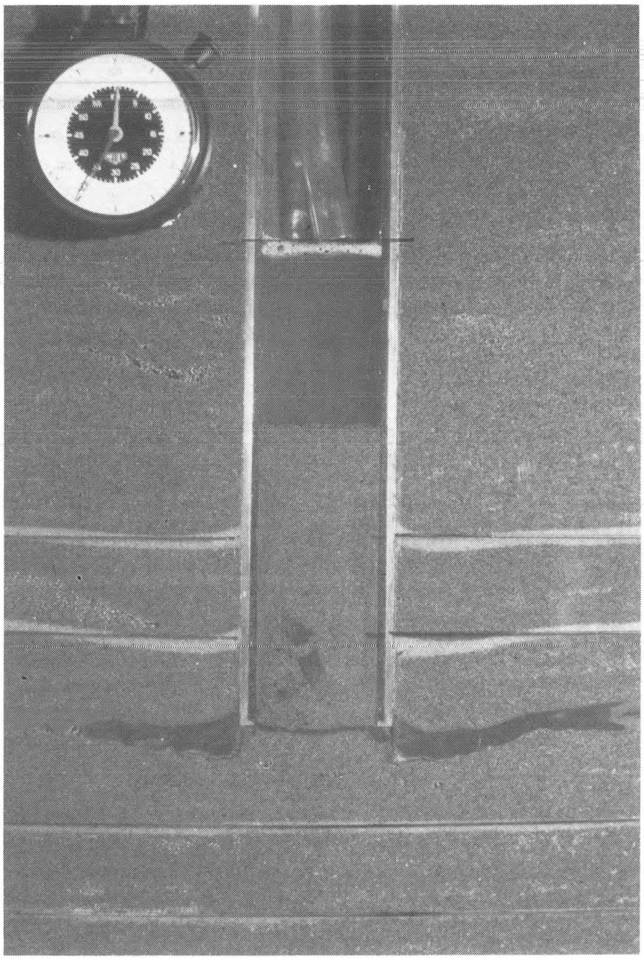

Fig. 2
Bailer boring in sand with clay layers.

versely affected. Basing himself on the streamline pattern associated with such a difference in water level he calculated what amount of lowering of the water level in the casing is critical with regard to the formation of a 'blow' at the bottom of a borehole in sand or clay. He also showed what phenomena occur on exceeding this calculated critical difference in level in a homogeneous sand stratum and in a sand stratum intersected by thin clay strata. In addition, he proved that more particularly in those instances where the borehole casing is in a clay stratum overlying a sand stratum the progressively increasing hydraulic gradient brings about the rise of the clay in the borehole casing. This phenomenon, which is further intensified by the additional negative pressure due to the rapid pulling up of the shell or bailer as well as by the subsequent lowering of pressure when the full shell emerges from the water in the casing, is the cause of the often quite considerable inaccuracies in the determination of the level of the clay/sand interface (Figs. 1 and 2).

Furthermore, in consequence of the very large decrease in intergranular pressure in the mass of soil around the end of the casing, increases in the porosity of the samples obtained are liable to occur. When tested in the laboratory, such samples yield unreliable values for the soil mechanical properties.

Only by means of very careful, and therefore relatively slow, execution of the exploratory borings under close supervision can errors be avoided in obtaining undisturbed samples, and their consequences in the laboratory, as well as in the 3-dimensional interpretation of the soil profile.

In the methods envisaged in category (2) the consequences of excess water pressures in relation to the groundwater level emerge in the zone around the bottom edge of the borehole casing (BEGEMANN, 1977). Both as a result of the flushing-water pressures employed and as a result of the considerable excess height of the column of soil-and-water mixture in the casing in relation to the groundwater level the effective horizontal intergranular pressure in this zone may be exceeded. This in turn will cause vertical radially directed hair cracks which are detrimental to the quality of the samples to be taken from that zone. For soil mechanics and foundation-engineering investigations these methods are, however, only sporadically employed in The Netherlands.

The adverse effects of water overpressure or underpressure that are liable to occur in the methods belonging to the

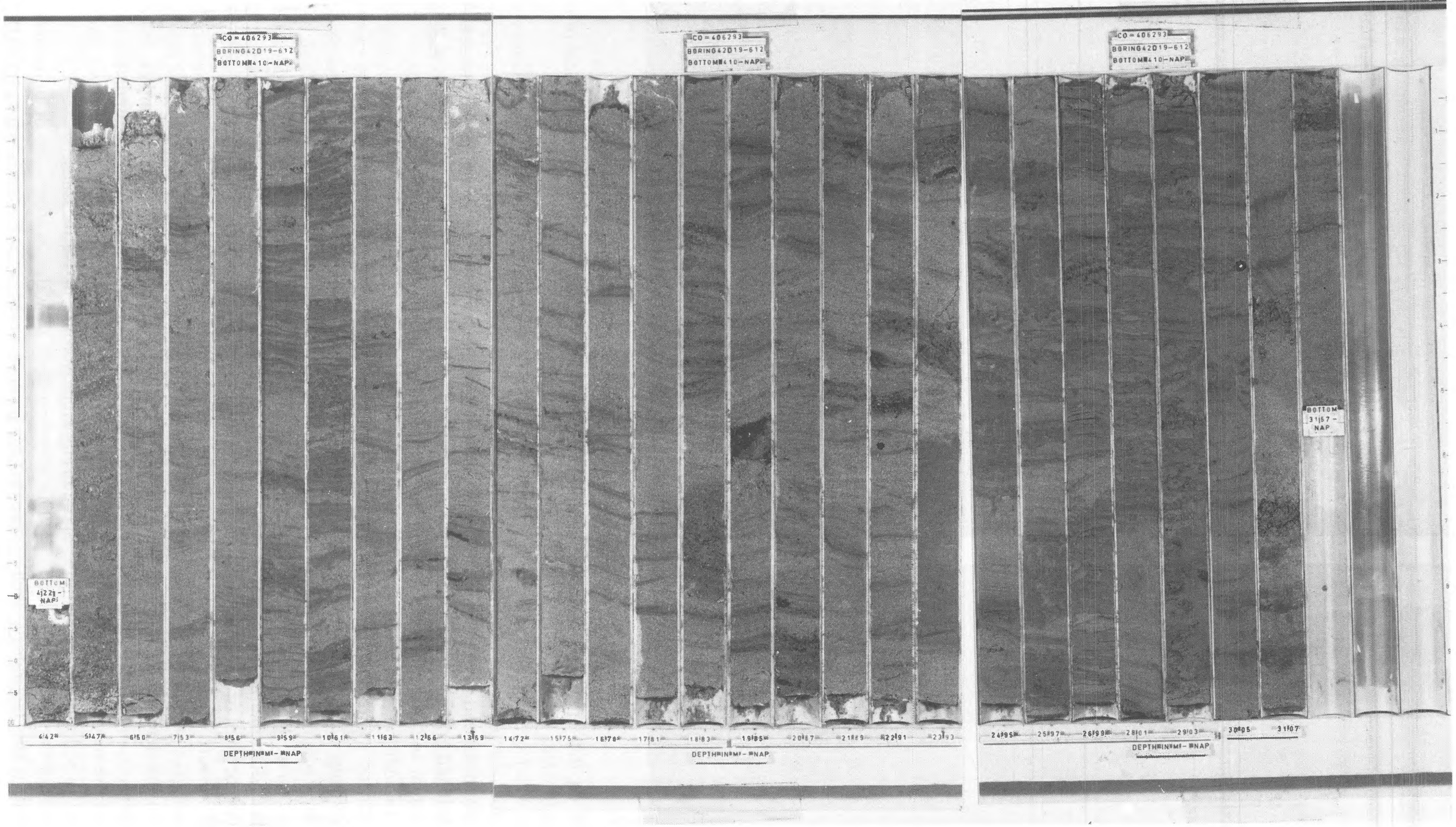

Fig. 3
Result of 66 mm Begemann boring and penetration test at the same spot.

categories (1) and (2) cannot arise in the case of category (3), for the simple reason that no borehole has to be formed in advance.

By the application of pressure or vibration a tube or a set of tubes is introduced into the ground, by means of which the sample of predeterminable length is cut at the same time.

For the determination of a practically undisturbed soil profile one of the methods in this category is therefore mostly employed, more particularly the vibratory boring or the Begemann boring with small-diameter (29 mm) equipment (BEGEMANN, 1961).

The last-mentioned technique yields samples which are practically undisturbed as regards stratification, structure of the soil, etc. For laboratory investigation of stress behaviour these samples are not very suitable, however: not only is their diameter rather small, but also, especially in the case of soft soil samples, the horizontal supporting pressures during the sample cutting operation are often found to have been too high.

For practical reasons the vibratory boring technique is mostly employed for exploratory sampling under water. When applied to sandy soils, corrections for the lengths of the samples obtained are necessary in order to compensate for compaction caused by the vibration.

Since vibrations are moreover liable to have an adverse effect on the quality of clay samples, the Begemann boring with 66 mm diameter equipment is often preferred as a means of obtaining undisturbed samples for laboratory testing (ANONYMOUS, 1977). With this method it is now possible to obtain a continuous sample up to 30 m in length in the relatively short time of 4 to 5 hours. In this way not only a complete and detailed soil profile is obtained, but in addition it yields virtually undisturbed samples of acceptable diameter of which structure and density have not, or hardly, been affected by the manner of boring. As the boring tubes are thrust into the ground to the required depth by pressure, the depth attainable with the Begemann system is dependent on the magnitude of the reaction force that can be mobilized (Fig. 3).

TYPE II: SHAFT WALL AS TEST MATERIAL

With regard to the methods of soil exploration in which a shaft formed in the ground is utilized for performing *in situ* measurements for determining the stress-strain behaviour, the existing state of stress, the density or the permeability properties there are three types, according to the manner in which the shaft is formed:

(1) as a conventional borehole;
(2) a carefully executed 'undisturbed' borehole;
(3) a borehole formed by soil displacement by means of a penetration body.

In so far as type (1) is concerned, the method most widely used in The Netherlands' delta region is the hole formed by shell and auger boring. In general, the boreholes are used for carrying out geophysical measurements.

In soil mechanics and foundation engineering the *in situ* permeability is often determined by means of the so-called falling-head test or the recovery test. Both these methods start with a water level in the borehole casing which corresponds to the groundwater level existing in the soil layer at the lower end of the casing; in this situation no water flows into or out of the casing. In the falling-head test the water level in the casing is raised, so that water flows out of it into the soil. In the recovery test the water level in the casing is lowered, so that water flows into it from the surrounding soil. The rate of water inflow or outflow is an approximate measure of the permeability of the soil in the vicinity of the lower end of the casing. Since the soil permeability in the horizontal direction often differs from that in the vertical direction, these methods merely yield an average value.

Over the years, many efforts have been made to refine these somewhat crude measuring methods. One of the most highly perfected variants, but technically difficult to perform, was successfully applied in connection with the damming of the East Scheldt (Oosterschelde) (VERMEIDEN & LUBKING, 1978).

A borehole can also be used for obtaining information on the stress-strain behaviour of the soil in the horizontal direction at any particular depth. In practice this is done with the so-called pressure-meter test. For this purpose an uncased borehole is sunk by means of rotary mud flush boring and its sides are supported by a fluid of high specific gravity. A probe which is radially deformable in the horizontal direction is then inserted into the hole. When the probe has reached the desired depth, the pressure within it is increased step by step, each step being maintained for a certain length of time. At the same time the horizontal volume changes of the probe are measured (BAGUELIN ET AL., 1978).

In scale tests or prototype measurements boreholes are used in some cases for the installation of devices measuring strain, inclination or deformation in the subsoil; in most cases horizontal deformations are registrated (ANONYMOUS, 1975), but also vertical deformations are measured (HEINS & DE LEEUW, 1977).

Since the borehole walls are, for various reasons connected with the execution of the boring operation, generally far from 'undisturbed' and are not to be regarded as representative of the *in situ* condition of the soil strata, in terms of either their structure or their state of stress, it soon became evident that the methods of type (1) can yield much better results that are attainable by forming a supposedly 'undisturbed' borehole (2).

After a good deal of experimenting it proved possible, in so far as the pressure-meter principle is concerned, to achieve very good results with the so-called retrojet method. In this system the measuring cell is accommodated inside a thin-walled steel tube which is open at its lower end and

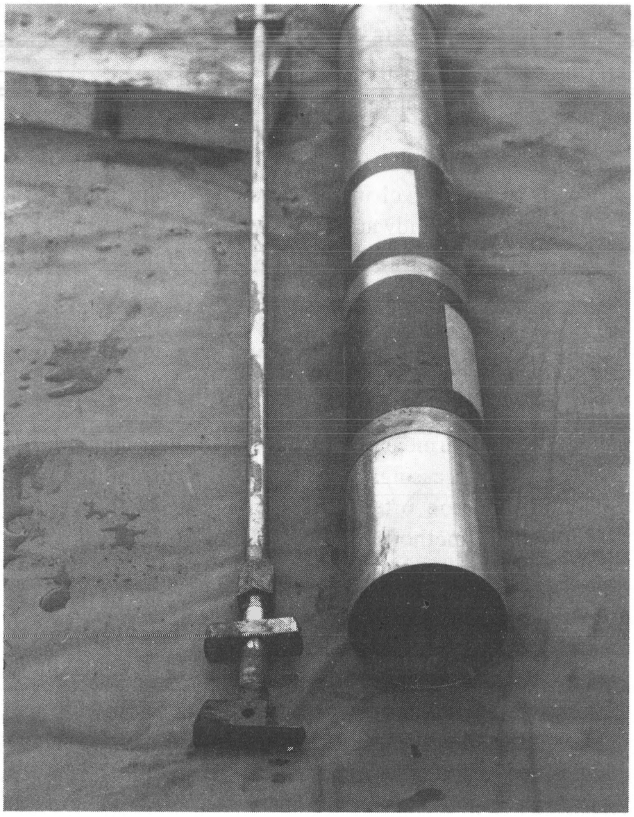

Fig. 4
Camkometer with pressure cells in side-wall.

which is forced into the ground to the desired depth by lightly applying pressure to it. The soil that enters the tube is carried away up the tube by a jet of fluid directed obliquely upwards. Both in sand and in clay soils good contact between the soil and the measuring cell is obtained with this equipment, while the surrounding soil is only slightly disturbed. In the 1960s and 1970s a variety of specialized versions of the pressure-meter test were developed (BAGUELING ET AL., 1978). On the one hand, these developments aimed at obtaining even more accurate observation of the stress-strain behaviour in the horizontal direction and, on the other hand, they were concerned with determining the existing horizontal soil stress. The best known devices of this kind are the PAF (Pressiomètre Autoforeur) and the Camkometer (Fig. 4). Both consist of a hollow cylinder provided on its lateral surface with one or more load cells for pressure measurement. The cylinder is jacked into the ground; the soil entering it is loosened by the action of a cutter mounted in the tube and is brought to the surface by water flushing. Once it has been introduced into the ground to the desired depth, the PAF functions in principle like an ordinary pressure meter. The Camkometer has primarily been developed for measuring the actually existing horizontal soil stress by making the pressure in the measuring cells equal to the horizontal soil pressure; when this has been achieved, the outer diaphragm of the load cell is stress-free.

A borehole formed in the same manner as for the PAF or the Camkometer test also provides an ideal starting position for other measurements. In connection with this, various devices for the determination of optimally reliable values of, for example, the density, the stress-strain properties and possibly the permeability of the soil are at present under development. In scale tests or prototype measurements the undisturbed borehole could also be used for the accommodation of equipment for the recording of rapidly or slowly varying stresses and/or strains in the soil under the influence of external factors such as foundation pressures or earth-moving operations.

With reference to the methods of type (2) there has been a trend towards increasingly sophisticated techniques of execution in order to keep the disturbance of the subsoil to the absolute minimum, enabling increasingly reliable measurements of the soil-mechanical parameters to be obtained. On the other hand, there has been an evolution of techniques in which the measuring body is introduced into the ground by displacement (instead of insertion into a bored hole), more particularly by driving or thrusting it in (3). With a procedure of this kind the density of the soil in the immediate vicinity of the measuring body will undergo certain changes, and in granular soils crushing of grains may occur. Hence the extent of the disturbed zone and/or its influence on the final measured result must be limited as much as possible. The first-mentioned aim can be achieved by giving the penetrating body a 'streamlined' shape as far as practicable, by keeping its diameter as small as possible and by carrying out the penetration with all possible care.

In practice, a set of tubes terminating in a conical point at the lower end and introduced into the ground by thrusting action is found to give the most satisfactory results. The standardized diameter of 36 mm adopted for cone penetration testing (see BEGEMANN, 1979) has proved to be mechanically strong enough to penetrate even relatively firm sand strata, sufficiently large to accommodate the requisite measuring equipment in terms of dimensions, and furthermore to create a disturbed zone of such a small size that its effect on the results of the measurements remain within acceptable limits for most practical purposes.

For these reasons, in recent decades a number of measuring devices have been developed also for routine investigation work which are accommodated in a 36 mm diameter probe that can be thrust into the ground: the density probe, the permeability probe, the friction probe, the water-pressure gauge and the soil-pressure gauge (ANONYMOUS, 1977).

Density measurements may be performed with the nuclear-radiation scatter method or by means of the electric-dipole probe. In the first-mentioned method the measurements based on the backscatter principle are obtained inside a tube which has been thrust into the ground. The principle of the electric-dipole probe (which is suitable only for use in sand) is based on the fact that the granular material of the soil is not electrically conductive (Fig. 5). Hence the electric resistance

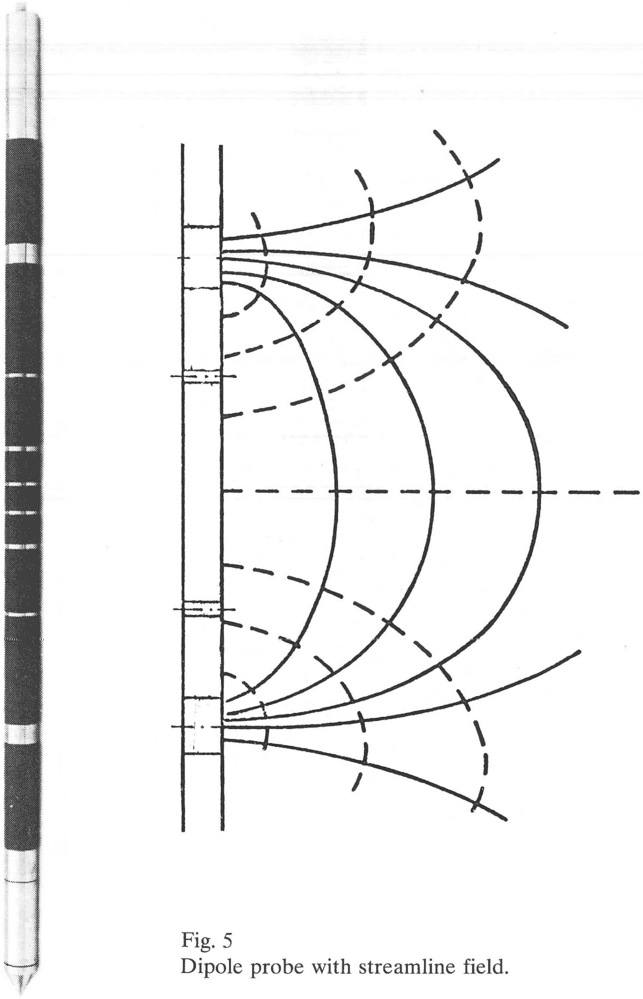

Fig. 5
Dipole probe with streamline field.

of the soil is determined entirely by the amount of groundwater in the pores and by the composition of this water. The higher the porosity of the saturated sand and the higher the salt concentration in the water, the lower will be the specific electric resistance measured.

The soil-conductivity probe consists of a steel core sheathed in plastic. Two pairs of steel rings encircle it at different distances from its lower end. When the probe has been forced so deep into the ground that the centre of the pairs of rings is at the level at which the density is to be measured, an electric potential difference is applied between the two outermost rings, the current electrodes. With the aid of the two inner rings, which are the measuring electrodes, the electric resistance ϱ_{gr} of the 'spherical' mass of soil between the two current electrodes is measured.

Next, the specific resistance of the water is determined at the same level with the aid of a second probe, the so-called water-conductivity probe. This device is provided with a filter through which a small quantity of water is drawn into the measuring cell and measured for electric resistance ϱ_w. The quotient of water resistance to soil resistance (ϱ_w/ϱ_{gr}) has a direct relationship with the porosity of the sand. As the accuracy of this relationship may to a greater or less extent be affected by the granulometric distribution, the grain shape and the presence of silt or shells, the relationship should be determined afresh for each geological formation and/or each region in which measurements are to be performed (VERMEIDEN & LUBKING, 1978).

In view of the drawbacks associated with permeability measurements with the so-called open-borehole method, as already mentioned with reference to point (1), in recent decades various permeability probes have been devised which in most cases are based on the principle of the recovery test. At its lower end the probe designed to be thrust into the ground is provided with a filter, while the system is closed at the top by special measuring and control equipment which enables measurements to be obtained relatively quickly even in soil of low permeability, e.g. clay (ANONYMOUS, 1977). In medium and compact sands it is not possible to use this system because in such soils the probe is surrounded by a zone of crushed grains (caused by the thrusting of the probe into the ground) which affects the filter resistance and thus has a considerable effect on the results of the measurements. Satisfactory results were, however, obtained by using a probe of smaller diameter, but in many instances this probe proved to be mechanically too weak to penetrate deeply enough into sand strata.

In order to overcome these difficulties, a new permeability probe has been developed for use in sand, called the hydraulic-dipole probe, which is closely related to the electric-dipole probe (RIETSEMA & VIERGEVER, 1979). In this permeability-measuring device the two current electrodes of the electric-dipole probe have been replaced by filters which function as a source and a well respectively, i.e. water is forced into the surrounding soil through one filter, while the other filter draws water into the probe, and vice versa (Fig. 5). When a streamline field has been established in this way the hydraulic potential difference between the two inner filters is measured with the aid of a differential-pressure gauge. If furthermore the rate of flow circulated through the soil is known, the permeability of the spherical mass of sand between the two extreme filters can be calculated.

It can theoretically be shown that in this case, for a point-shaped source or point-shaped well, the measured potential difference is independent of the vertical permeability; in other words, only the horizontal permeability of the soil is measured.

Because of the actual dimensions of the filters, the vertical permeability does have some effect on the measured result, but this effect is relatively slight and generally negligible in practice. If it is desired also to measure the vertical permeability, a potential measurement will have to be performed at the same time in a known position in the streamline field beside the probe.

The great advantage of this method over all the permeability measuring techniques mentioned above is the fact

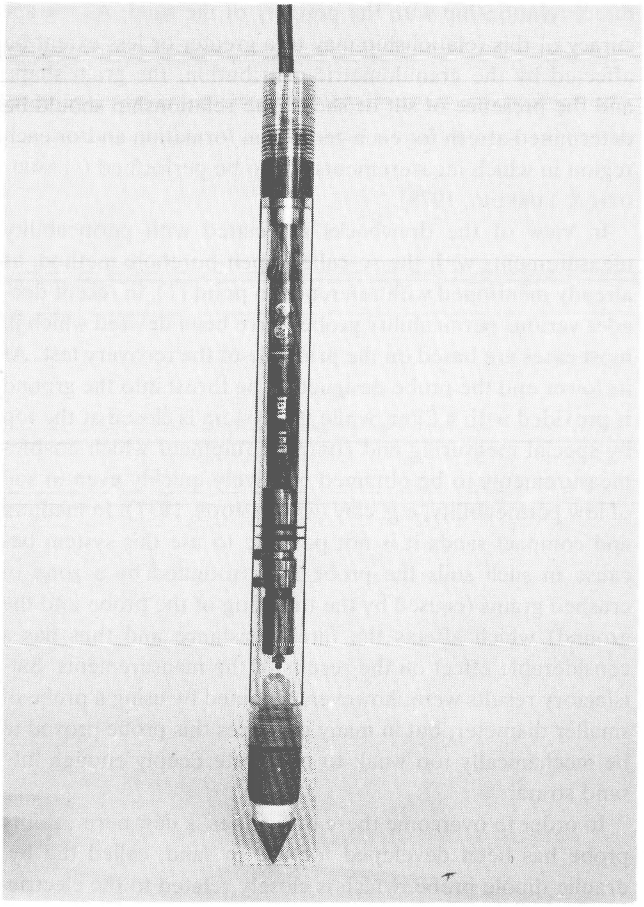

Fig. 6
Water-pressure gauge (transparant model).

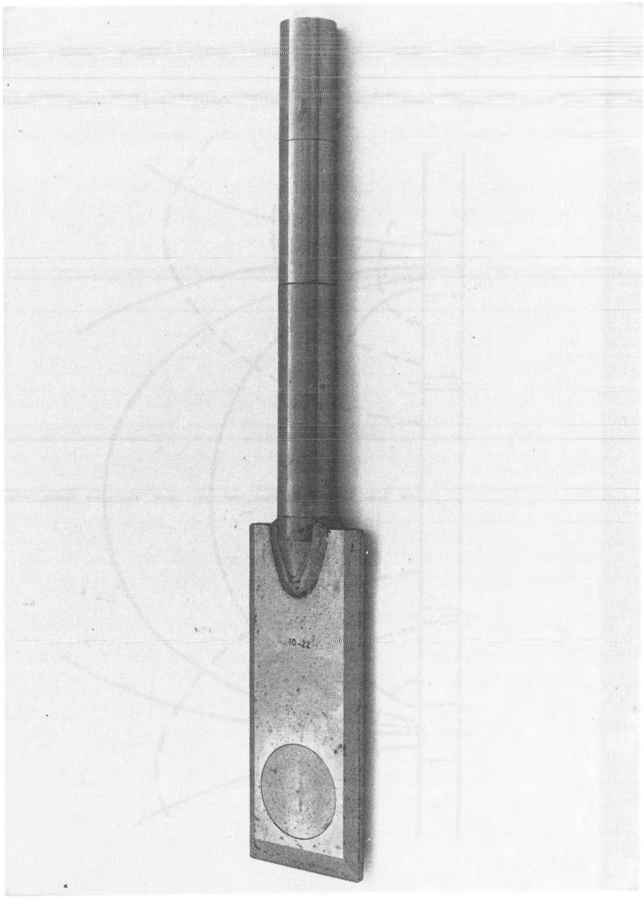

Fig. 7
Lance-type pressure cell.

that the magnitude of the filter resistance is unimportant; the only condition is that a flow of water from source to well, or vice versa, has to be established.

Another measuring method, belonging to category (3), is the so-called friction sleeve. With this device, which has been developed in connection with the evolution of cone penetration equipment, it is possible to obtain some idea of the steel-to-soil adhesion or, with a rugged sleeve, of the cohesion.

The water-pressure gauge with which the magnitude or the change in magnitude of the water pressure at a certain level in the ground can be recorded is likewise assignable to category (3). Evolved from the standpipe, which is in fact merely a borehole casing in which the water level is recorded, the device is essentially a tube which is forced into the ground and is provided with a filter at its lower end, while the upper end of the tube is closed and provided with a pressure gauge.

In more recent versions of this device the recording system has been replaced by electric or pneumatic pressure sensors installed directly behind the filter (Fig. 6). Although it is mostly used for the recording of very gradually changing phenomena, such as the variation in water pressure during consolidation, it is also possible, by using special sensors, to detect and record very rapid changes in water pressure at any particular level in the ground (HEINS & DE LEEUW, 1977).

The earth pressure existing at a certain depth, or the changes in pressure occurring there, can be measured with the aid of an earth-pressure sensor. For this purpose the sensor is accommodated inside a spear-shaped element fitted to the lower end of a set of tubes. Earth pressures in other than horizontal directions can be measured by introducing the tubes into the ground at a certain angle of inclination instead of vertically (Fig. 7).

The pressure meter, too, has evolved in the direction envisaged in category (3). In order to obtain better contact of the probe with the soil than is possible by using the device in an uncased borehole with a heavy fluid-supporting medium, the probe is incorporated in a steel tube which is driven into the ground to the desired depth. This tube is of low rigidity in the radial direction because it is provided with longitudinal slots (VERMEIDEN & LUBKING, 1978). As an alternative to driving, this probe may be installed in the ground by vibration, this being done more particularly in fine sands in which the slots are liable to become clogged, so that difficulties are encountered if the probe is installed by driving.

Furthermore, developments are currently in progress with a view to limiting the disturbances of the wall of the shaft which without doubt occur when the probe is driven or vibrated into the ground, by pushing the probe into the soil (HENDERSON ET AL., 1979). In order also to limit the displacement of the soil that occurs under such conditions, the lower end of the probe is sometimes constructed as the cutting edge of a soil sampler.

TYPE III: TESTING DURING PENETRATION

The soil-mechanical and foundation-engineering investigation methods involving the measurement of forces or moments necessary for bringing about a certain movement of a measuring body driven or thrust into the ground have long been, and still are, extensively used for reason of their simplicity, especially in soft-soil regions (MELZER, 1968; ZWECK, 1969; SANGLERAT, 1972).

The number of forms of construction indeed is virtually unlimited. Every country, and even every region, has gone through a period of development of varying length, in so far as such equipment is concerned, so that each has its own more or less individualized measuring device which is geared to the locally encountered subsoil conditions – particularly as regards the technical execution of the test – and whose measured results can, on the basis of experience, easily be correlated with purely soil-mechanical parameters or with the bearing capacity of the actual foundation structures.

These methods can most suitably be classified with regard to the manner in which the device is introduced into the ground:

(1) The so-called dynamic methods, in which the measuring body is taken down to the required depth by driving, for which, as a rule, only a drop-weight is needed. Examples are the standard penetration test (SPT) and the dynamic cone penetration test (CPT).

(2) The so-called static methods, in which the measuring body is taken down to the required depth by thrusting, for which it is nearly always essential to develop a reaction force of sufficient magnitude. Examples are the static cone penetration test (CPT) and the vane test.

In actual practice the dynamic methods comprise percussive and driving penetration tests. In principle, these are all methods in which a rod, provided with a conical tip at its lower end, is driven into the ground by means of a free-falling 'hammer'. The various versions of such equipment differ from one another in the size of the drop-weight ('hammer'), the height of fall, and the shape and dimensions of the penetration cone. These tests may be performed from ground level or alternatively from the bottom of a borehole formed in advance. In all cases the number of blows needed for increasing the penetration depth by a certain amount is determined.

The most widely known method in this category is the so-called standard penetration test (SPT), applied more particularly in the English-speaking countries. In principle, the measuring body which is driven into the ground from the bottom of a borehole is not a cone but a split spoon sampler. In this standardized method the number of blows (N) by a hammer weighing 622.9 N with a 0.76 m height of fall which are needed to drive the sampler a distance of 0.305 m into the ground is the SPT value. If the test is performed in gravel or coarse sand, the bottom part of the sampler with its cutting edge is allowed to be replaced by a solid cone with a vertex angle of 60°. The SPT or N values then obtained are found to differ little from those obtained with the unmodified split spoon sampler.

An additional advantage of the SPT is that a soil sample is obtained of each section of depth in which a value of N has been determined. Obviously, this technique must be applied with considerable caution when used below the water table. As a result of creating in the borehole a water level which is higher or lower than the surrounding groundwater level it may easily occur that too high or too low values of N are obtained, this being due to the softening or the compression of the soil in the borehole casing.

The dynamic cone penetration test is usually not performed from the bottom of a borehole, but direct from ground level. That is why this type of driving penetration test is quicker and simpler to perform. Against this, the test results are, especially with increasing depth, affected by the friction of the soil against the rod. In order to reduce these objectionable frictional influences, various counter-measures can be taken:

- using a rod of smaller diameter than the cone;
- using a bentonite suspension which is pumped through the hollow rod and, on emerging from holes at the top of the cone, flows upwards in the hole along the outside of the rod;
- enclosure of the rod within tubes as employed in static penetration testing (see below).

The values of N obtained from the SPT or from a driving penetration test serve as a basis for foundation engineering considerations and calculations. By virtue of experience and tests over a good many years, methods of interpretation have been developed with which, aided by the N-values, a fair idea of the stratification and composition of the soil can be obtained. Regional knowledge of the geological features or at least data derived from borings carried out in the vicinity are essential to such an interpretation. Furthermore, it is not possible to draw direct inferences as to the physico-mechanical soil properties from theoretical considerations. However, the N-value is often used, though usually in combination with the estimated existing soil stress, for obtaining some guidance as to the density of non-cohesive soils. If at the same time, as is indeed the case with the SPT, information on the composition of the soil is available, the density can be estimated even more accurately. From soil-mechanical investigation it is known that the angle of internal friction of non-cohesive material is largely dependent on the density. Since this angle moreover plays a major role in the theoretical bearing capacity formulas, it is obvious that many investigators have given correlations between the N-value and the bearing-capacity coefficients in the formulas for determining the ultimate bearing capacity of foundations. Furthermore, many investigators have derived a direct relation between the N-value and the deformation behaviour of non-cohesive soil; in these cases, too, the estimated existing soil stress is always introduced as a variable (TERZAGHI & PECK, 1938; GIBBS & HOLZ, 1957; ZWECK, 1969; SANGLERAT, 1972).

For cohesive soils it is possible, on the basis of the SPT, to derive a very approximate relation between the N-value and the consistency. Since consistency is also bound up with the stress-strain behaviour a tentative idea of the mobilizable bearing capacity and of the probable deformations of foundations can be obtained in this way.

In the so-called static methods (2) the subsoil is explored by means of a usually slow-moving penetration body. The most familiar example is the cone penetration test, first developed in The Netherlands in the early part of this century (BARENTSEN, 1936) and, especially in the last few decades, widely adopted throughout the world.

Numerous variants have emerged over the years with regard to the shape and dimensions of the cones and rods, the manner of thrusting the cone into the ground, and the method of recording the pressures. There is nevertheless, thanks to the standardization achieved in 1977, a greater measure of uniformity than in the driving penetration techniques.

A penetration cone with a vertex angle of 60° and a diameter of 36 mm is fairly generally employed, while the extension tubes mostly have an external diameter of 36 mm; the penetration speed is about 2 cm/s and, what is actually the most important feature, the cone resistance is measured separately.

Up to the 1960s the cone resistance was measured solely by mechanical means. In this method the pressure that the soil exerts on the cone is transmitted via the inner rods to the measuring unit mounted above ground (BROUG, 1972). In present-day practice, electric measurement is increasingly being used; the pressure exerted on the cone by the soil is measured by means of an electric device mounted directly over the cone, the measuring signals being transmitted via a cable to a recording instrument installed above ground (VAN REE, 1972).

Mechanical cone penetration testing is usually performed as a discontinuous procedure: the cone pressure is measured by forcing the cone a certain distance into the ground by means of the inner rods, the casing tubes being lowered later. In both operations the magnitude of the pressure needed for performing the downward movement is measured: first the cone pressure, the magnitude of which is determined by the bearing capacity of the soil, then the pressure applied to the string of casing tubes, the magnitude here being determined by the soil friction on the tubes. In order to obtain more detailed information on the friction at various depths, the so-called friction jacket cone was developed in the 1950s. For this purpose a loose cylindrical sleeve, of the same external diameter as the casing tubes (36 mm), was interposed between these and the penetration cone, which was of the so-called mantle type then normally employed. First the cone alone is pushed down a certain distance, then it is pushed a further distance, but now followed by the sleeve, while the casing tubes remain at rest; in that situation the cone resistance *and* the local friction are measured as one combined value. The casing tubes are next pushed down.

Electric measurement is usually performed continuously, i.e. the tubes are thrust continuously into the ground for a length of 1 or 2 m, while the pressure exerted on the cone is measured continuously with the aid of special transducers and often directly displayed by means of an X-Y recorder. With electric penetration testing it is likewise possible to measure the local friction, using a cylindrical sleeve similar to that used in mechanical testing. This sleeve is mounted directly over the cone or some distance above it.

The simplest version of the electric penetration testing cone is the so-called straigth cone or constant-diameter cone, which is now of a standardized form (HEIJNEN & JOUSTRA, 1977). In addition, the type known as the DSML (Delft Soil Mechanics Laboratory) cone is also used in The Netherlands; its characteristic feature is that it is constricted for some distance above the actual cone, the object of the constriction being to make the results more directly comparable with those yielded by the discontinuous measurements with the mechanical cone, for it was upon these latter measurements that the interpretations and calculation rules developed at the DSML were based (HEIJNEN, 1973) (Fig. 8).

In thrusting the cone into the ground, the ultimate bearing capacity of the soil into which the cone penetrates is constantly being exceeded. Hence the cone penetration test is primarily a test for bearing capacity or strength. Obviously, sand will have higher cone resistance values than clay or peat, and densely packed sand or firm clay will have higher resistances than loosely packed sand or soft clay. For this reason, it is often possible, on the basis of the penetration test results, to make a rapid and reliable prognosis of the composition and structure of the foundation soil strata. One important condition for such an interpretation, however, is that certain reference data are available for the region concerned, more particularly in the form of geological information, results of exploratory borings and/or the results of geophysical investigation.

By comparing the cone resistance with the local friction encountered at various depths it is, however, possible to obtain a good indication of the soil types even only on the basis of penetration resistance plus friction measurement. This relationship, first found by BEGEMANN (1965), yields very serviceable results in practice. The presence of preconsolidated strata can also be detected by means of this relationship.

Many investigators have pointed out that the magnitude of the cone resistance is determined by a combination of factors: the type of soil, the existing state of stress and the

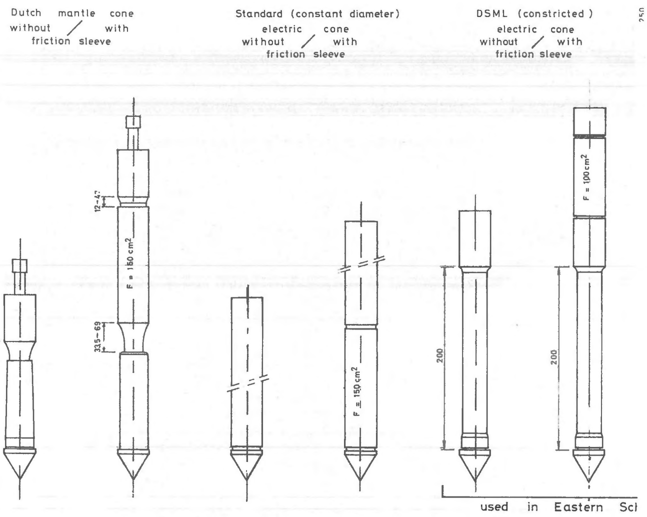

Fig. 8
Cone types: mantle cone, standard cone and DSML cone.

density encountered. Therefore it will in general not be possible to determine the soil-mechanical parameters separately from the cone penetration test results. Yet, over the years, very useful correlations between cone resistance and density or between cone resistance and stress-strain parameters have in many instances been established (ANONYMOUS, 1977; ZWECK, 1969; SANGLERAT, 1972; BEGEMANN, 1977-b; SCHMERTMAN, 1977).

As in the case of the methods envisaged in category (1), it is only possible to obtain approximate guiding information on the density of non-cohesive soils by cone penetration testing. Knowledge of the composition of the soil and possibly of the geology of the region concerned can be helpful in improving the accuracy of the information thus obtained. In establishing the correlation between density and penetration results the estimated soil stress at the relevant depth is often introduced as a variable. In scaled-down tests in the laboratory, where both the composition of the non-cohesive material and the state of stress are known, the density is often determined by penetration tests in combination with the experimentally predetermined relation between cone resistance and density.

For non-cohesive soils there exist, in most countries or regions, very serviceable correlations between the cone resistance and the angle of internal friction. In such correlations, too, the soil stress at the relevant depth is nearly always introduced as a variable.

For cohesive soils a simple relationship between cone resistance and undrained shearing resistance (cohesion) is usually adopted. In general the agreement between this cohesion and the local friction is reasonably good. The stress-strain behaviour of cohesive and of non-cohesive material can also be deduced from the cone penetration test results by means of simple correlations.

In view of the proved serviceability of the above-mentioned correlations, it is an obvious step also to establish correlations between penetration test results and the bearing capacity of foundations, just as has been done for the dynamic methods (1). Here again the existing state of stress and the composition of the soil are often introduced as variables (SANGLERAT, 1972; SCHMERTMAN, 1977). A much more obvious approach, however, is to use the cone penetration test results in the design of pile foundations, since the cone and the extension tubes above it represent something closely resembling a scale model of a foundation pile.

In The Netherlands, and also in its neighbouring countries, there exist various methods of determining the permissible pile point bearing pressure and the anticipated positive or negative friction on the pile, basing oneself on the penetration test results. All these methods are empirical or semi-empirical; some are based entirely on comparing the results of penetration tests with those of loading tests on actual piles, others are based in part on soil-mechanical theories as to deformation around a penetrating pile point (SANGLERAT, 1972; SCHMERTMAN, 1977).

The cone penetration technique using the principle of thrusting (forcing the cone into the ground under steady pressure) has the advantage over driving (hammering) that it enables the point resistance to be recorded separately and that it produces only relatively minor disturbance of the soil, so that it generally yields more reliable and more accurate results. In respect of its technical execution, however, the thrusting technique often presents more difficulties. For instance, it is always necessary to mobilize a reaction force for the thrusting equipment; furthermore, incidental discontinuities in the soil such as stones, thin 'cemented' strata and layers of shell material often prove too resistant to penetrate. The reaction force is generally obtained with the aid of kentledge (dead weight), by anchorage or a combination of the two. The weight of the kentledge or the strength of the anchorage is in practice adjusted to suit the soil conditions, the probable composition and resistance of the strata to be penetrated, and the desired penetration depth. This depth can be increased by a number of special techniques, all of which in principle are measures for reducing the friction of the penetration tubes. Such measures are:

- the use of constricted penetration tubes;
- friction-inhibiting or friction-reducing devices in the form of projections on the tubes (ridges or rings);
- the use of a flushing liquid which is injected above the cone and upwards along the tubes;
- boreholes: after the cone has penetrated to its maximum normally attainable depth, a borehole is sunk to that depth, and penetration of the cone is then resumed;
- the use of a borehole casing which is introduced into the ground with the aid of rotary motion and which removes the soil around the penetration tubes and itself then serves as a guiding tube;
- so-called telescopic cone penetration testing, a technique

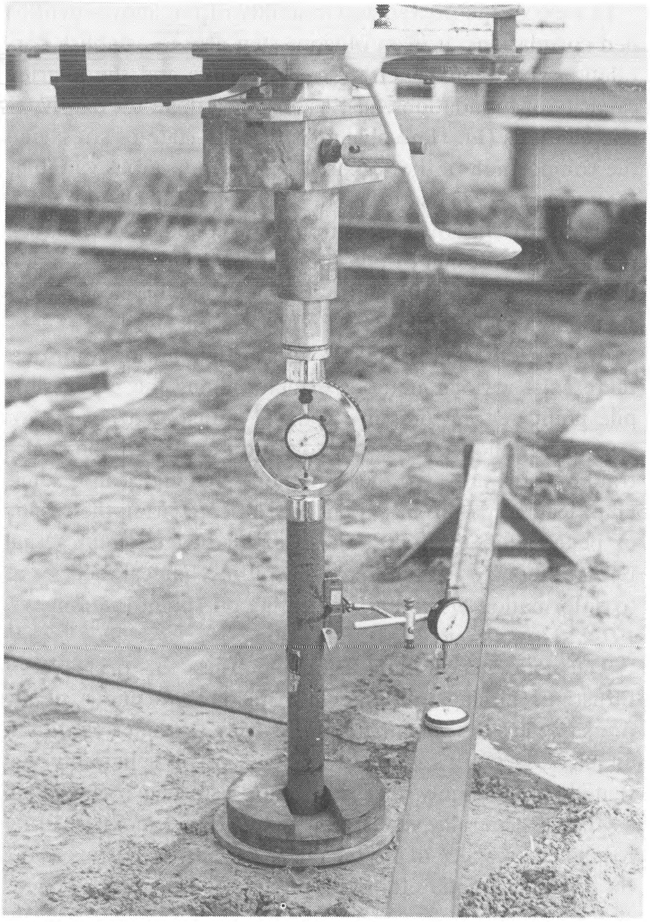

Fig. 9
CBR test.

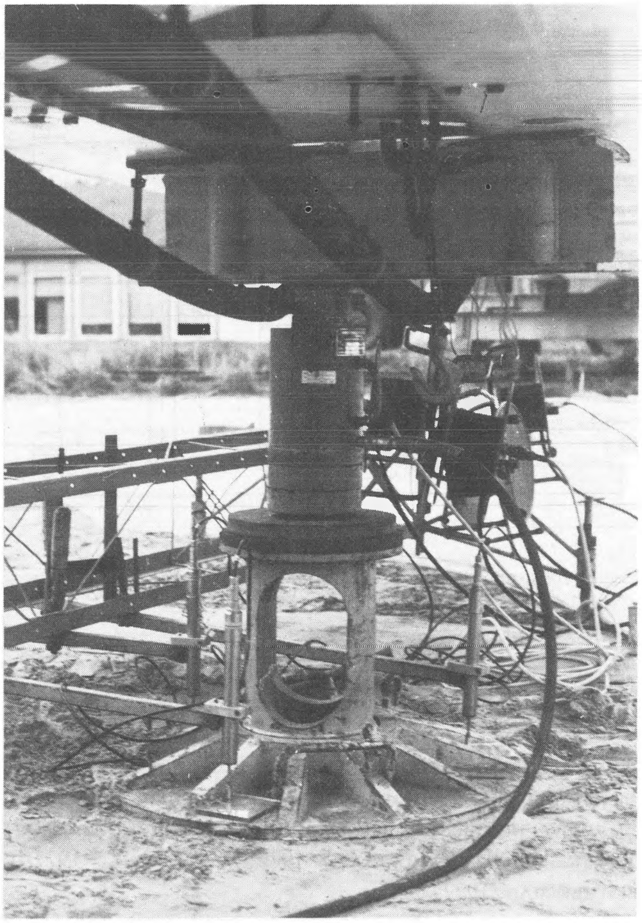

Fig. 10
Plate bearing test.

in which first a string of larger-diameter penetration tubes is thrust into the ground and which contains a string of normal-diameter tubes, of which the bottom tube with the penetration cone protrudes from the first-mentioned string; when the larger-diameter tubes have been thrust into the ground to a certain depth, penetration testing continues with the normal tubes.

The two last-mentioned methods offer the additional advantage that the risk of buckling in the upper strata, where the compressive stress in the penetration tubes is greatest, is reduced. The execution of cone penetration tests from the surface of water with the aid of a pontoon is in principle also confronted with buckling problems. In such circumstances the string of penetration tubes is therefore also enclosed within a casing which serves as a guiding tube. Besides, by deriving the reaction force from a kentledge block which has been lowered to the bed of the waterway beforehand, the casing will be subjected to tensile stresses which reduce the buckling hazard. In order to avoid complications with respect to the technical execution of investigation from the surface of water, the thrusting equipment can be lowered to the seafloor. In that position penetration tests can be performed by divers or remote-controlled (ANONYMOUS, 1975; VERMEIDEN, 1961; ZUIDBERG, 1972; VAN REE, 1977).

Related to the cone penetration test is the CBR test, which is extensively used in highway- and airfield-construction engineering (ANONYMOUS, 1977). It is a practical loadbearing test in which a plunger of 5 cm diameter is thrust into the top layer of soil to a depth of 2.5 cm at a specified very low speed. The test is usually performed at depths of 10 to 20 cm below ground level and serves as a basis for the design of flexible pavements (Fig. 9). The result of the test also provides an indication of the quality of the ground as regards passability for the contractor's equipment. On account of its small range in depth, the method is hardly ever used for purposes of soil mechanics and foundation engineering, and even in highway engineering its application is limited for the same reason.

The plate bearing test is also used in highway engineering, but is moreover employed for investigation purposes in connection with foundation engineering (ANONYMOUS, 1977). In contrast with the CBR test, which is displacement-controlled, this test is stress-controlled. A circular steel plate a few decimetres in diameter is pushed into the top layer of soil with the aid of a reaction force provided by kentledge (Fig. 10). The test gives information on the stress-strain behaviour of the soil and more particularly on the elastic behaviour of the upper layers.

Another test belonging to category (2) is the vane test (ANONYMOUS, 1975). In principle, a four-bladed vane is thrust carefully into undisturbed water-saturated cohesive soil and then rotated about its vertical axis, causing the cylindrical mass of soil within the circumference of the blades to shear off (Fig. 11). The torque required to do this is measured and provides a criterion for the undrained shearing resistance (cohesion) of the soil. The test can be performed from the bottom of a borehole formed in advance. In another method of execution of the test, the vane with the extension tubes, enclosed within an outer set of tubes, is thrust into the ground until it arrives just above the desired testing depth, after which the vane is thrust farther down below the lower end of the outer tubes. According to most investigators the results of the vane test are in reasonably good agreement with those of the local friction measurement. The composition of the soil is probably an important variable governing the results.

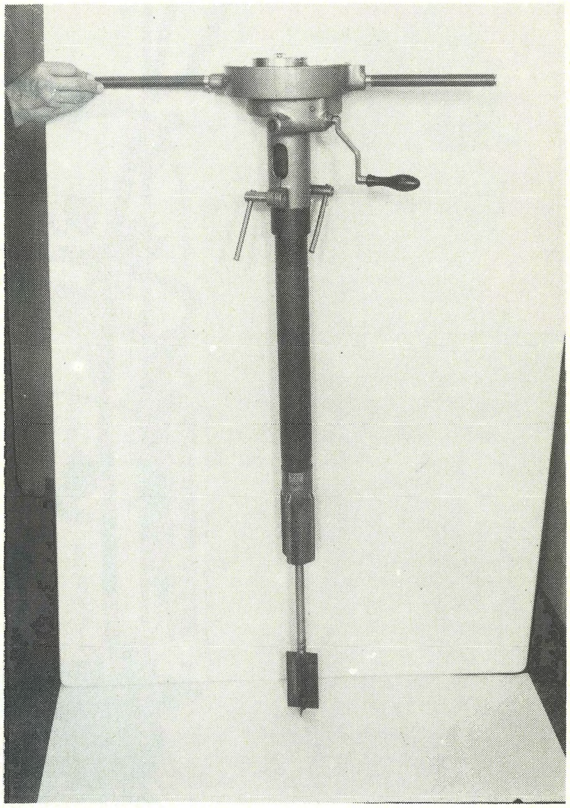

Fig. 11
Four-bladed vane.

SUMMARY AND CONCLUSIONS

The present-day trend is towards the construction of larger and larger structures both on land and on the sea bed, and this trend is reflected in the demands made upon soil mechanics and foundation engineering. Besides, in the present economic climate with its aim to cut expenditure, foundation structures have to be built as inexpensively as possible. To this end it is necessary, first and foremost, to have adequate soil-mechanical knowledge; in addition, sufficient and reliable information on the quality of the subsoil is essential. It will therefore be necessary, on the one hand, to strive for qualitative improvement in soil sampling (also at great depths) and for rapid and direct measurement of soil mechanical parameters. On the other hand, with regard to cone penetration testing (or similar measuring methods), further research into extending fundamental knowledge of soil mechanics will be needed, while at the same time the relatively simple technique of testing will have to be geared even more to the rapid acquisition of information on the subsoil at considerable depths below the surface of the ground.

With regard to sampling, various approaches have already been initiated which can lead to optimum results; continuous boring has proved to be a technique with which samples of very good quality can be obtained relatively quickly in soil investigations carried out both on land and from the surface of water. A drawback of this method is that it is still not yet possible to penetrate very compact or 'cemented' sand strata. Rotary boring equipment can, however, provide assistance in passing through such strata.

Of the methods envisaged in type II procedures the undisturbed borehole system (2) will ensure that soil-mechanical data can be determined better and better *in situ*, more particularly for stress-strain behaviour, state of stress, density and permeability. In general, however, the technique of performing the tests will be so complicated that the methods will in principle be confined to fundamental research in soil mechanics and to very special foundation-engineering projects, as they can be used only by specially trained personnel.

The displacement methods of forming a borehole (3) will be more suitable for routine application.

The two last-mentioned categories (2) and (3) do, however, still require a good deal of research with regard to execution technique and to interpretation in soil mechanical terms.

The cone penetration test is best suited for yielding information, quickly and cheaply, on the quality of the foundation soil strata and is preferable for routine investigation purposes. Although the method has been available and widely used for a good many years, it hardly had any substantial basis in soil mechanics until fairly recently. Research conducted in the 1960s and 1970s has changed this and can be expected to elucidate more fully the difficult set of problems associated with the penetration of a conical body.

As regards the technique of executing the tests, developments are moving in two directions:

(1) Combination of cone penetration testing with the

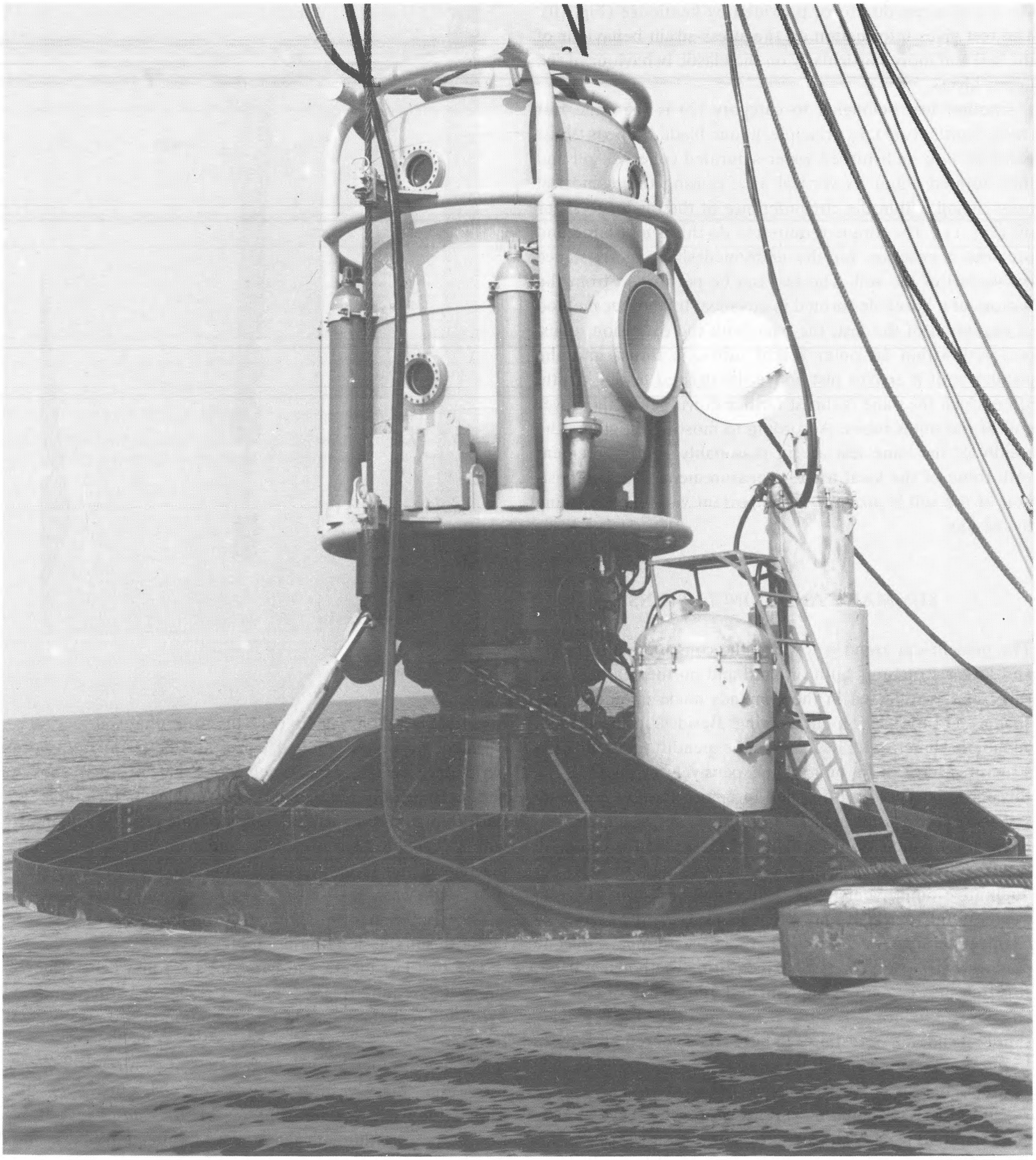

Fig. 12
Diving bell.

measurements of friction, density, water pressure, permeability and horizontal soil stress. At present some of these combinations are already applied in routine testing (ANONYMOUS, 1977; RIETSEMA & VIERGEVER, 1977; TORSTENSSON, 1978) (Figs. 5, 6, 8).

(2) Facilitating and speeding-up the execution of tests where deep penetration is required or tests performed from the surface of water, by using:

- more specialized equipment;
- more kentledge and more friction-reducing measures;
- measures for piercing very compact or 'cemented' sand strata.

The most striking representatives in this area can be found in offshore soil investigation: During the last two decades Fugro developed several apparatuses for *in situ* investigation of the North Sea floor, most of them working remote-controlled (ZUIDBERG, 1972; VAN REE, 1977). Delft Soil Mechanics Laboratory together with Vriens Diving Company constructed a diving bell which can be lowered to the sea floor; cone penetration tests as well as 66 mm Begemann borings are performed by divers inside the bell (ANONYMOUS, 1977; VERMEIDEN & LUBKING, 1978) (Fig. 12).

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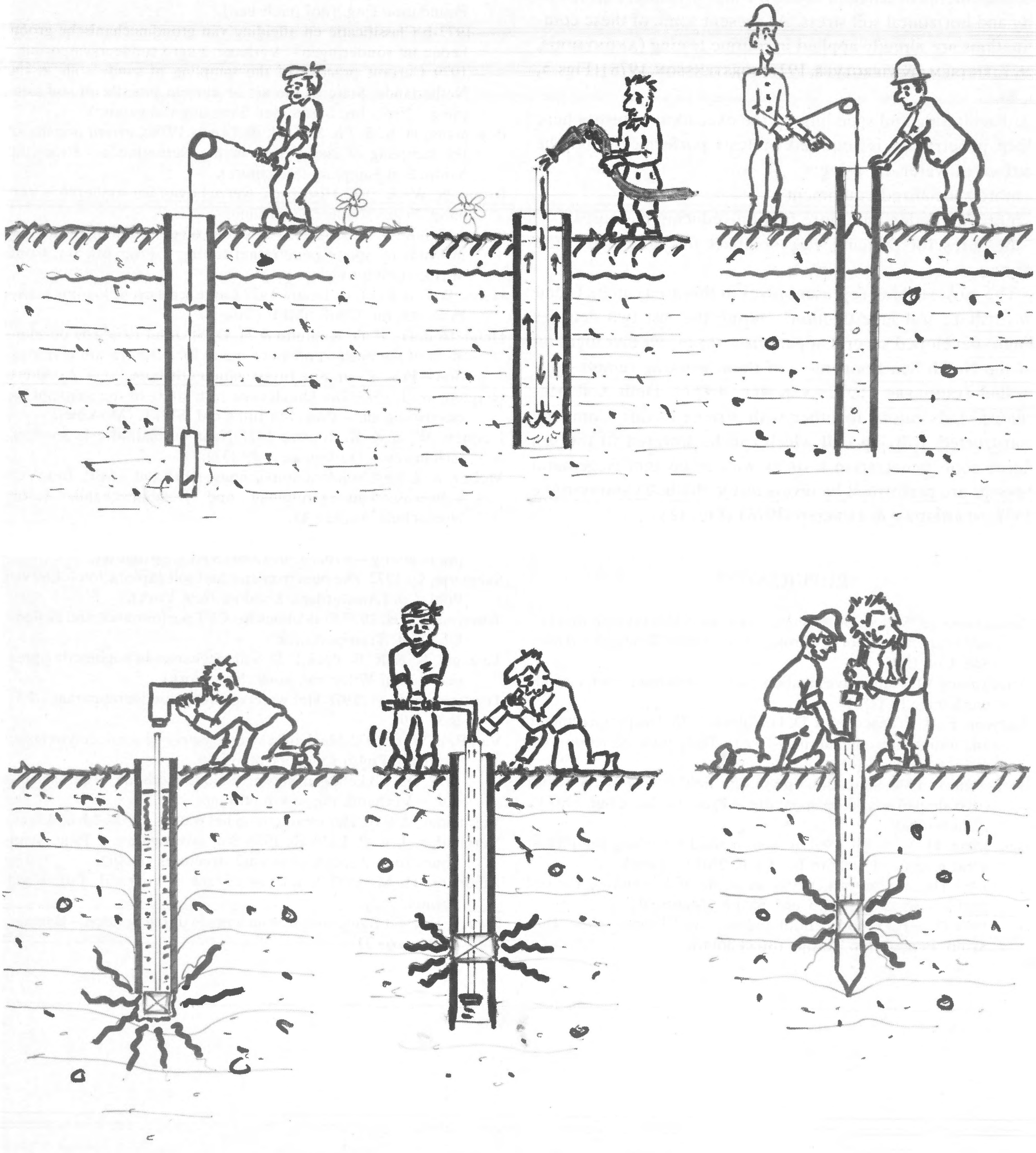

These cartoons illustrate the various principles of soil-investigation methods, mentioned in this paper.

Upper part (from left to right): boring techniques (1), (2) and (3).
Lower part (from left to right): borehole techniques (1), (2) and (3).