Field testing methods for engineering geological investigations

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Abstract. This paper discusses some of the more commonly used field testing methods used to determine various geotechnical parameters needed to classify soils and rocks in engineering terms and to provide engineering design values. The methods of performing, and the application of, various penetration, shear strength, plate loading, pressuremeter and dilatometer and permeability tests are described. The use of geophysical methods to obtain, indirectly, mechanical parameter values is also discussed.

Field testing and, to a lesser extent, geophysical surveying are major sources of both qualitative and quantitative data relating to the ground conditions. In spite of the fact that in most cases field testing is more expensive to carry out than sampling and laboratory testing, it forms an essential part of many site investigations. There are several reasons for this, probably the most important of which is because it provides, for design purposes, parameters which represent a more realistic appraisal of geotechnical ground conditions than is commonly the case with laboratory testing. Samples which are used for laboratory testing, because of their small size, may not be sufficiently representative of the ground from which they were taken. In particular, they may not contain large-scale discontinuities, present in the rock or soil mass, which significantly influence the engineering properties of the materials concerned. Furthermore, sampling inevitably involves some disturbance to the stress conditions and water content of soils and rocks so that parameters obtained in the laboratory may not be fully representative of the in situ conditions. Although not without some problems of its own, field testing overcomes many of these difficulties. However, with in situ tests it can be difficult to test materials at appropriate stresses, or in such a manner that changes in the stress, and other conditions due to engineering works, can properly be taken into account.

Field testing methods are most likely to be used for the determination of the engineering behaviour of soils and rocks in which it is difficult, impossible or very expensive to obtain good quality undisturbed samples for laboratory tests. According to Clayton et al. (1982) such circumstances are most likely to arise with very soft or sensitive soils, clays containing cobbles, un cemented sands and gravels and soft, fissile and highly fractured rocks. Consideration is given in this paper to the principle of operation and engineering geological applicability of various in situ tests which are used frequently for the determination of the values of engineering design parameters. The techniques described include penetration tests, shear strength tests, loading tests, pressuremeter tests and permeability tests as well as geophysical methods. The discussion does not include details of monitoring equipment and field instrumentation. Thus piezometers, and settlement, strain and stress gauges, and inclinometers are excluded, except where, as in the case of the piezcone, the measurement is a major aspect of the technique being considered.

Only a brief discussion is included on the use of geophysical methods for geological surveying to determine layering in the ground and to locate anomalous zones. Instead, the paper concentrates on the determination of physical properties, both directly and indirectly, and their relationships with mechanical properties.

A number of other in situ tests have been developed in recent years including the Marchetti Flat Dilatometer, the Iowa Stepped Blade and various Total Stress Cells. These instruments are used less commonly than those described below and are beyond the scope of this paper. However, brief descriptions of the use of instruments and discussions on the various tests carried out using them, have been given by Jamiolekowski et al. (1985).

Penetration tests

Two types of penetration tests are recognized (Anon 1975 and 1981). The standard penetration test (SPT) is a dynamic method in which a 51 mm external diameter split tube sampler connected to drilling rods, is driven into the ground by a series of hammer blows delivered at the surface. The test is conducted at intervals during the course of boring and it provides a distributed sample for identification purposes. In the static
penetration test a conical point is driven into the ground by means of a steady pressure on the top of the rods. Both tests provide an indirect measure of shear strength since the action of the tests produces a complex failure surface within the soil.

Standard penetration test

Although the standard penetration test provides only a very approximate guide to the properties of sands, clays and weathered rocks it is widely used since it is simple and cheap to carry out. In the test the sample tube is driven into the ground in three successive increments of 150 mm by a 63.5 kg hammer (European Standard; 65 kg, BS 1377 (Anon 1975)) free falling a distance of 760 mm onto an anvil mounted on top of the drill rods. The result quoted is the number of blows (N) required to advance the tube the last 300 mm. For gravels and very coarse grained soils or weak rocks, the sampler may be replaced by a conical point with a 60° apex angle.

Quoted correlations for 300 mm penetration blow counts with both relative density and internal friction angle for sands are given in Table 1. Table 2 provides correlations for the undrained shear strength and consistency of clays. Unfortunately, the results are sensitive to slight changes in technique so it is important that the test is carried out in the correct manner. In addition, these correlations should be treated with caution since the results are dependent on a number of factors, including the mineralogy, grading and water content of the soil as well as the in situ stress conditions, depth and pore water pressure conditions.

In particular, very low values (< 10) may be indicative of the disturbance of sands at the base of the holes caused by high water pressures. For this reason when boring below the water table the hole should be kept topped up with water. Unfortunately, this may not completely overcome the problem, especially with cable percussion boring owing to the suction caused during the withdrawal of the shell (Rodin 1961). In very hard ground there is a danger that the soil at the base of the hole may be disturbed as the casing is driven. Care has to be taken in gravels, and soils containing large clasts, to ensure that the result has not been influenced by the presence of a few large fragments blocking the tube. Usually the lowest N values are quoted. The influence of overburden pressure on relative density has been considered by Douglas (1983) who indicates that the values are over estimated as depth increases.

A crude relationship for granular soils between standard penetration resistance and deformation modulus has been established by Schultz & Melzer (1965) for different effective overburden pressures at the test level. Values of immediate settlement can then be obtained from the deformation modulus, the values of deformation modulus obtained should not be used for net bearing pressures much larger than 100 kPa. Burland & Burbridge (1985) have also determined an empirical relationship for the calculation of settlements on non-cohesive soils based on standard penetration resistance.

The lowest angles of internal friction given in Table 1 are conservative estimates for uniform, clean sand and these values, together with the upper ones for well graded sands, should be increased by about 5° for gravelly sands. Terzaghi & Peck (1967) suggested that due to the generation of pore water pressures, in fine grained and silty sands below the water table, N values greater than 15 should be increased to 15 + \( \frac{1}{2} \) (blow count - 15).

The estimates of strength given in Table 2 apply to clays of medium plasticity. Schmertmann (1975) indicated that for low plasticity clay the value should be reduced to approximately half that quoted, whereas for high plasticity clay the value should be increased by approximately 66%.

Various correlations between standard penetration blow count and the bearing capacities of footings and piles have been suggested. The matter has been considered by Peck et al. (1974) who provided design charts relating the allowable bearing pressure, foundation width and depth to the N value for sands. The same authors also gave correlations between the N value and the bearing capacity factors. Tomlinson (1986) and Sanglerat (1972) provide details of the use of standard penetration test data in pile design.

### Table 1. Standard penetration test below count (N), relative density and internal friction angle of sands

<table>
<thead>
<tr>
<th>N Value (300 mm)</th>
<th>Compactness</th>
<th>Relative Density</th>
<th>Angle of internal friction (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 4</td>
<td>Very loose</td>
<td>&lt; 0.2</td>
<td>&lt; 30</td>
</tr>
<tr>
<td>4–10</td>
<td>Loose</td>
<td>0.2–0.4</td>
<td>30–35</td>
</tr>
<tr>
<td>10–30</td>
<td>Medium dense</td>
<td>0.4–0.6</td>
<td>35–45</td>
</tr>
<tr>
<td>30–50</td>
<td>Dense</td>
<td>0.6–0.8</td>
<td>40–45</td>
</tr>
<tr>
<td>&gt; 50</td>
<td>Very dense</td>
<td>0.8–1.0</td>
<td>&gt; 45</td>
</tr>
</tbody>
</table>

### Table 2. Standard penetration test blow count (N), consistency and undrained shear strength of clays

<table>
<thead>
<tr>
<th>N Value (300 mm)</th>
<th>Consistency</th>
<th>Undrained shear strength (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 2</td>
<td>Very soft</td>
<td>&lt; 20</td>
</tr>
<tr>
<td>2–4</td>
<td>Soft</td>
<td>20–40</td>
</tr>
<tr>
<td>4–8</td>
<td>Firm</td>
<td>40–75</td>
</tr>
<tr>
<td>8–15</td>
<td>Stiff</td>
<td>75–150</td>
</tr>
<tr>
<td>15–30</td>
<td>Very stiff or hard</td>
<td>&gt; 150</td>
</tr>
<tr>
<td>&gt; 30</td>
<td>Hard</td>
<td>&gt; 150</td>
</tr>
</tbody>
</table>
Standard penetration tests have been applied to the classification of weak rocks. For example, Wakeling (1970) correlated N values with the weathering grade of chalk and Douglas (1983) suggested that very weak rocks correspond with a penetration of between 100 and 300 mm for 50 blows compared with a penetration of 30 to 100 mm for weak rocks, whereas, moderately strong rocks give virtual refusal.

**Static cone penetration test**

Sanglerat (1972) gave details of a number of alternative commonly used static cone penetration tests. Probably the most widely employed one is Dutch Cone Penetrometer in which a 60° cone-shaped point, with a base area of 1000 mm², is driven into the ground at a constant rate of 20 mm per second whilst the resistance to penetration is measured. The test is particularly suited to the investigation of soft clays, silts and sands in which undisturbed sampling is very difficult. Since no borehole is required the test is relatively inexpensive to perform.

Measurements of cone penetration resistance may be made either intermittently at depth intervals of 200 or 250 mm using a mechanical penetrometer, or continuously with an electrical cone penetrometer. In both cases it is also possible to measure the friction mobilized on the sides of the holes created by the cone.

The mechanical cone is operated by inserting it to the required depth by means of pressure applied to a hollow sounding tube. The cone resistance is then measured by advancing the cone a distance of 70 or 80 mm by means of pressure applied via rods acting within the sounding tube. The Bergemann (1965) cone incorporates a friction sleeve which is engaged by the cone once it has been advanced by 40 mm. Once engaged both the cone and the friction sleeve are advanced together a further 40 mm. During this stage the combined penetration resistance mobilized on the friction sleeve and the cone are measured; the contribution due to the friction sleeve alone is then derived by subtracting the cone penetration resistance just measured from this value. Finally, by means of pressure on the outer sounding tubes, the cone and friction sleeve are returned to their original locations with respect to each other and the sounding tubes and the instrument are advanced to the next required measurement position.

With the electrical cone penetrometer a continuous measurement of both cone resistance and sleeve resistance are obtained as the probe is advanced on one set of rods. Again, the probe incorporates a cone tip and a friction sleeve; the pressures on these elements are measured independently by means of internal electrical strain gauges. Figure 1 shows a soil profile obtained by an instrument of this type.

With continuous and internal measurement of both cone and sleeve resistance, the electrical cone penetrometer provides a more reliable indication of soil properties, particularly sleeve resistance, than those obtained using the mechanical device. The friction sleeve and the cone penetration results are often quoted in terms of friction ratio which is the ratio of the sleeve to the cone resistance. As a consequence of frictional forces generated during continuous penetration, Heijen (1973) argued that cone resistance may be 20 to 100% higher than those obtained with the discontinuous, mechanical method.

Cone penetration tests are a quick, and convenient method of investigating soils, particularly when used in conjunction with boreholes from which samples or core are obtained. They can be used to log the distribution and thicknesses of the various soil layers over an area while also providing information about their geotechnical properties.

The undrained shear strength \( c_u \) of a soil is normally derived from the expression

\[
q_c = c_u N_k + \gamma z
\]

Fig. 1. Cone penetrometer traces (after Smith 1978).
where $q_c$ is the static cone resistance, $\gamma$ is the total unit weight of the soil, $z$ is the depth and $N_k$ is a factor with a value of between 10 and 15 for normally consolidated clays and 15 and 20 for overconsolidated clays (De Ruiter 1982).

Various attempts at relating static cone resistance to the relative density ($D_r$) of sands have been made. Lunne & Christoffersen (1985) suggested that

$$2.91 D_r = \ln \left( \frac{q_c}{61(\sigma_v')^{0.71}} \right) \times 100\%$$

where $\sigma_v'$ is the effective overburden pressure, and this and $q_c$ are measured in kPa. The widely used relationship given by Schmertmann (1978) gives values which are about 10% higher than this.

The results of static penetration tests are used extensively for pile design. Generally speaking, unless the soil is layered and non-homogeneous, the unit end bearing capacity is usually taken as the average penetration resistance for a depth range of three times the pile diameter below this position. However, following research on dense North Sea sands, De Ruiter (1982) suggested that test results may yield unrealistically high values for the end bearing capacity of piles.

Various relationships for estimating the consolidation characteristics of soils appear in the literature. Meyerhof (1974) and Gielly et al. (1970) derived values for settlement and compression index from cone penetration data. Schmertmann et al. (1978) derived an equation for the calculation of settlement of foundations on non-cohesive soils. The equation is partly based on deformation modulus which can be calculated from cone penetration data. Lunne & Christoffersen (1985) have proposed values of initial tangent modulus for normally and overconsolidated sands which can then be related to deformation modulus.

Van den Berg (1987) described the use of the piezocone which is an electric cone penetration device which incorporates a small piezometer tip. This instrument is particularly suitable for investigating thinly bedded sands and clays. By monitoring pore water pressure dissipation with the probe in one position, the device may also be used in the investigation of the consolidation and permeability of soils.

### Shear strength tests

Various means of obtaining a direct indication of the in situ shear strength are available. Probably the most widely used of these is the shear vane test but brief reference will also be made to direct shear and triaxial tests.

#### Shear vane test

Vane tests, in which a cruciform arrangement of plates mounted on the end of a rod, which may be enclosed within a guide tube, is inserted into the soil at the base of a borehole or trial pit and twisted, provide an indication of both peak and remoulded undrained shear strengths. Undrained shear strength is calculated from the relationship between torque and angular rotation, the vane dimensions and the maximum torque applied. Remoulded shear strength is measured similarly, after remoulding by rapid rotation of the vane and allowing a short period of time for pore pressures to dissipate. The test is used for very soft to firm, saturated, non-fissured, homogeneous clays. La Rochelle et al. (1973) noted that vane test values differ widely from those obtained from laboratory tests, a phenomenon which they attributed to disturbance of the soil during vane insertion. In addition, Bjerrum (1973) indicated that the actual field strength is lower than that measured by vane tests by a factor which depends on the plasticity index of the clay (Fig. 2). However, shear strength anisotropy, pore water pressures and in situ stress conditions also cause variation. Since the results are also sensitive to the rate of rotation of the vane, tests should be performed carefully and in accordance with the relevant specifications (Anon 1975).

For a homogeneous clay the undrained shear strength ($c_u$) is given by

$$3.14 c_u D^3 = 6T$$

where $D$ is the diameter of a vane having a height of $2D$, and $T$ is the torque. The calculation assumes that shear stress is uniformly mobilized on the ends of the cylinder and that the clay is sheared in an undrained condition. In practice, care has to be taken to ensure that rod twist and friction on the sides of the hole do not cause errors. Richardson et al. (1975) described how anisotropic clays may be investigated by the use of vanes with a variety of shapes.

![Fig. 2. Correction factors for undrained shear strengths measured by in situ vane tests (after Bjerrum 1973).](http://egsp.lyellcollection.org/Downloaded from)
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Direct shear and triaxial tests

The shearing characteristics of rock masses, particularly those containing discontinuities, can be determined by the use of in situ shear tests. Such methods are often employed during the investigation of the shear strength mobilized on discontinuities themselves and the tests have also been applied in the case of overconsolidated clays (Marsland 1973a). In situ shear tests are usually performed either in a trial pit or specially excavated gallery (Anon 1981). Usually a block of clay or rock is left projecting above the base of the excavation and both shear and normal forces are applied to it. Both peak and, by the use of large displacements, residual shear strength parameters may be obtained. Unfortunately, in the latter case a very large block may need to be used. This makes it both difficult and expensive to perform the test and furthermore, a non-realistic distribution of stress on the base of the block can occur. Serafim (1964) discussed the significance of volume changes which occur during the test.

It is also possible to carry out in situ triaxial tests, although this technique is rarely used. These tests consist of establishing a block to which lateral and normal pressures can be applied. Voort & Logters (1974) described the use of flat jacks in pre-drilled slots to provide lateral pressures. In situ triaxial tests allow large volumes of rock and soil to be tested under a great variety of stress conditions.

Plate loading tests

Various forms of plate loading tests provide direct information relating to the strength and settlement characteristics of soils and rocks. They are extensively used for the investigation of soils which are difficult to sample, particularly fills.

The test is carried out by monitoring the settlement of an incrementally loaded bearing plate usually between 150 mm and 1 m diameter until either failure takes place or a load equal to three times the design value has been applied. Frequently, the loading is cycled in order to study hysteresis effects. The tests are usually carried out in trial pits although they can be undertaken in large diameter boreholes. In all cases care has to be taken to ensure that the ground surface immediately beneath the plate is not in a disturbed condition (Marsland 1973b).

The undrained shear strength \( c_u \) of a soil may be obtained where a single stage continuous loading test is continued to a pressure \( q_{ult} \) at which failure occurs. The value is given by the equation

\[ c_u N_c = (q_{ult} - \gamma H) \]  

(4)

where \( N_c \) equals 6.2 for a circular plate at the surface and 9.3 at the base of a hole the same diameter as the plate, \( \gamma \) is the unit weight of the soil and \( H \) is the depth.

The modulus of elasticity \( E \) may also be determined from plate loading tests where

\[ 4E \rho = \pi q D(1 - v^2) \]  

(5)

in which \( q \) is the pressure for a settlement of \( \rho \), \( D \) is the diameter of the plate and \( v \) is Poisson’s ratio (\( v = 0.1 \) to 0.3 for granular soils and soft rocks). The determination of \( E \) by this method would provide a measure of the behaviour of the material en masse, particularly if the diameter of the plate exceeded the average discontinuity spacing in the material.

Figures 3 and 4, respectively, compare the results of various determinations of the deformation modulus and undrained shear strength of London Clay. Due principally to the presence of fissures, plate loading tests give lower strength values and higher modulus values than those obtained from laboratory tests on samples. The magnitude of these discrepancies become larger as the size of the plate is increased (Marsland 1973a).
Fig. 4. Results of undrained shear strength tests on London Clay from Hendon (after Marsland 1974, 1980).

Terzaghi & Peck (1967) related this settlement \( (\rho) \) of a 0.305 m square plate to that for a foundation \( (\rho_B) \) of width \( B \) as follows

\[
\rho_B(B + 0.305)^2 = \rho(2B)^2. 
\]

Unfortunately, owing to changes in soil properties with depth and, particularly in plastic soils, consolidation effects, this simple relationship can yield erroneous results.

The screw plate is a variant of a plate bearing test in which a helical screw is rotated into the ground to the requisite depths at which a loading test is performed (Kay & Parry 1982). The test has the advantage that no excavation or drilling is required and the test is easy to perform below the water table. As with the vane test, the insertion of the instrument may cause significant disturbance to the soil in the vicinity of the plate.

Ward et al. (1968) described the use of a very large-scale loading test to determine the mass deformation behaviour of chalk at the proposed site of a proton accelerator at Mundford, Norfolk. They used an 18.3 m diameter water tank to provide an average contact pressure of up to 183 kPa. In addition, a series of smaller plate loading tests were used to give stresses of up to 1.67 MPa.

**Pressuremeter and dilatometer tests**

The Menard pressuremeter is used for the measurement of the in situ strength, deformation behaviour and in situ stresses of soils and weak rocks. A number of authors, including Wroth & Hughes (1973) and Marsland (1976), have described the use of pressuremeters...
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in overconsolidated clays, mudrocks and sands. Examples of testing in a number of Carboniferous and Triassic rocks have been described by Meigh & Greenland (1965). Good comparability was apparently obtained between these latter results and plate loading test data.

The pressuremeter consists of a cylindrical probe which is lowered down a borehole to the test position. The probe is divided into three independent expandable cells arranged one above the other (Fig. 5) where the centre one is the measuring cell. The cells respectively above and below this, serve as guard cells which ensure that end effects are minimized during testing. During a test all three cells are inflated in stages and the resulting volume changes for the measuring cell are monitored by means of a volumeter, or, for greater accuracy, strain gauges situated within the cell. It is preferable to carry out the test in unlined boreholes, although in granular soils and other situations in which collapse is liable to occur, a slotted casing is employed. In practice, allowances must be made both for the expansion of pressure tubes, the resistance of the membrane and any casing to expansion, and the depth of the test position. These correlations are normally determined by calibrating the instrument.

Figure 6 is a typical response curve for a pressuremeter test. Initially, the in situ horizontal stress is re-established, after which the curve enters an approximately linear elastic phase. Eventually, at the creep pressure, this gives way to plastic deformation and ultimate failure at the limit pressure (P_L). Using elastic analysis, Gibson & Anderson (1961) derived expressions from the modulus of elasticity (E) and undrained shear strength (c_u) of clays as follows:

\[ E \Delta V = 2 \Delta P (1 + \nu) V_0 \]  
\[ C_u k = C_u \left[ 1 + \ln \left( \frac{E}{2c_u (1 + \nu)} \right) \right] = P_L - P_0 \]  

where \( \Delta V/\Delta P \) is the gradient of the pressure volume curve, \( V_0 \) is the volume of the measuring cell, \( \nu \) is Poisson's ratio and \( k \) is a constant which equals approximately 5.5 in most cases.

The results of a series of pressuremeter determinations of the undrained shear strength of London Clay are presented in Fig. 4 together with comparable laboratory and plate loading test data. Marsland (1980) noted that since the pressuremeter measures the soil properties in a horizontal direction and the diameter of the probe is small in comparison with the spacings of fissures, the pressuremeter may over-estimate the strength in comparison with the values obtained by other methods. In addition, Marsland & Randolph (1977) indicated that, due to stress relief effects, the in situ horizontal stress (P_0) need not correspond with the initiation of elastic deformation. They suggested that in stiff clays the value of P_0 should be determined from the creep pressure (P_F) as follows:

\[ P_0 = P_F - c_u. \]  

Fig. 5. The Menard pressuremeter (after Bell 1978).

Fig. 6. Pressuremeter test curves for Coal Measures silty mudstone (after Meigh & Greenland 1965).
In sands a value of the drained modulus may be obtained from equation (7) and it can be shown that the slope of a log volumetric strain \((\Delta V/V)\) versus log pressure curve is approximately equal to

\[
\frac{1}{2}\left(1 - \frac{(1 - \sin \phi)}{(1 + \sin \phi)}\right)
\]

from which the angle of internal shearing resistance \((\phi)\) can be obtained.

Most pressuremeters are designed for use in pre-bored holes and, generally, some disturbance to the sides of the hole occurs during boring. A reduction in disturbance can be achieved by the use of a self-boring pressuremeter. This is installed by jacking the instrument steadily into the ground while a rotating cutter removes soil entering the cutting shoe. Cuttings are flushed to the surface within the instrument without the flush coming into contact with soil outside the cutting shoe (Wroth & Hughes 1973). Some instruments also incorporate piezometers so that probe water pressures can be monitored during a test.

The self-boring principle has also been used to install other instruments in the ground such as permeameters and shearometers (Amar et al. 1977).

Dilatometers are similar devices to pressuremeters except that they operate at higher pressures and they are used in the determinaton of the elastic properties of rocks. Rocha et al. (1966) described the use of dilatometers in diamond bit drilled holes. By measuring the diametral strain \((D/\Delta \delta)\) by means of internal strain gauges, the modulus of deformation of the rock may be determined from

\[
E\Delta \delta = (1 + \nu)DP
\]  

where \(\nu\) is Poisson’s ratio and \(P\) is the applied pressure. This relationship is not adequate to describe the behaviour of fissured rocks.

Permeability tests

The simplest form of in situ permeability test can be performed by artificially raising or lowering the water level in a borehole and observing the rate at which this level changes. If casing extends to the base of the hole then the permeability \((k)\) is given by

\[
4Fk(t_2 - t_1) = \pi D^2 \ln(H_1/H_2)
\]

in which \(H_1\) and \(H_2\) are the piezometric levels at times \(t_1\) and \(t_2\) respectively, \(F\) is the intake factor where, in a cased hole in uniform soil, \(F = 2.74D\) and \(D\) is the diameter of the hole (Hvorslev 1951). Other intake factors may apply depending on the situation, two commonly used ones are as follows:

(i) \(F = 2D\), casing extending to the base of an impermeable bed with a permeable one beneath.

(ii) \(F = \frac{2\pi L}{\ln(\ell/D + [1 + (L/D)^2]^{1/2})}\)

partially cased borehole in uniform soil with an uncased length, \(L\).

In soils of higher permeability it is usually more convenient to use a constant head test in which water is either added to, or removed from, the hole so that a constant water level is maintained. Constant head tests are also appropriate in the case of soils which undergo consolidation or swelling in response to changes of stress. The permeability is determined from the steady rate of flow \((Q)\) and the induced change in head \((H)\) as follows:

\[
kFH = Q.
\]

Where impermeability soils are being tested it is usually necessary to obtain the steady state flow rate by extrapolation of the values at various times.

These tests provide only an approximate value of permeability in the general vicinity of the hole. In addition, they are liable to be severely influenced by localized inhomogeneities in the ground, disturbance due to drilling, leakage of supply pipes or casing joints and clogging of soil interstices by water borne sediment. More sophisticated tests may be performed by the use of packers by which means sections of the hole may be isolated, (Dixon & Clarke 1975). By withdrawing water from, or pumping water into, a test section either between two packers or one packer and the bottom of the hole, the permeability of a specific zone of ground may be measured. The values of permeability are determined using either the variable or constant head formulæ given above.

The Lugeon test is a variety of packer test which involves the use of a series of increasing and decreasing pressures and the flow is measured for each one. The permeability is given by

\[
2\pi LHk = Q \ln(2L/D)
\]

where \(H\) is the head of water at the test position. The Lugeon unit of permeability is defined as the water inflow measured in litres per minute per metre of test length for a pressure head of 100 m of water.

A slug test is another method of determining permeability, or transmissivity, that is, the permeability times the thickness of an aquifer, and storage coefficient of confined aquifers. The test is described by Cooper et al. (1967). Water levels are monitored following the injection into, or abstraction from, a hole, of a specific volume of water. The ratio of the head to the value it attained immediately after injection or abstraction is plotted against log time and by fitting the resulting curve to a standard curve, values for both transmissivity and storage coefficient may be obtained. Bouwer & Rice (1976) described a slug test for use in unconfined aquifers.
Single hole tests involving measurements of flow rate and pressure change provide an approximate indication of the permeability of the ground near to the hole. For an accurate assessment of permeability and storage capacity over a broad area it is necessary to observe the effects of water withdrawal or injection in wells or piezometers situated some distance from the test hole. The hydraulic properties of the ground affect the shape of the cones of depression or replenishment which respectively result from abstraction or recharge. In an unconfined aquifer the permeability may be determined from

\[
k\pi(s_1 - s_2) = Q \ln(r_2/r_1)
\]

where \(Q\) is the steady flow rate to cause a drawdown of \(s_1\) and \(s_2\) at distances from the pumping well of \(r_1\) and \(r_2\) respectively. In the case of confined aquifer of thickness \((h_0)\), the equivalent equation is

\[
2\pi k(s_1 - s_2)h_0 = Q \ln(r_2/r_1).
\]

**(Geophysical methods)**

Geophysical investigation methods commonly are used to determine the geological sequence and structure of subsurface rocks by the measurement of certain physical properties. The properties that are made most use of in geophysical exploration are: density, elasticity, electrical conductivity, magnetic susceptibility and gravitational attraction. Whilst the values obtained when measuring these properties are rarely of direct use in geotechnical characterization or design, many of them can be related either theoretically or empirically to more commonly used geotechnical, hydrogeological or geological parameters.

**(Seismic measurements)**

The sudden release of energy from the detonation of an explosive charge in the ground or from impacting or vibrating the ground, generates seismic shock waves which radiate out in hemi-spherical wave fronts from the point of release. The waves generated are compressional \((P)\), dilational shear \((S)\) and surface waves. The velocities of shock waves generally increase with depth below the surface since the elastic moduli increase more rapidly with depth than density. The compressional waves travel faster than the shear waves.

The shock wave velocity depends on many variables, including rock fabric, mineralogy and pore water content. In general, velocities in crystalline rocks are high to very high (Table 3). Velocities in sedimentary rocks increase concomittantly with consolidation, and with increase in the degree of cementation and diagenesis. Unconsolidated sedimentary accumulations have lower velocities which vary as a function of the mineralogy, the volume of voids, either air filled or water filled and grain size. Most rocks and unconsolidated deposits are anisotropic.

As indicated above, the porosity tends to reduce the velocity of shock waves through a material. Indeed, Wyllie et al. (1956) suggested that the compressional wave velocity \((V_p)\) is related to the porosity \((n)\) of a normally consolidated sediment as follows:

\[
\frac{1}{V_p} = \frac{n}{V_{pf}} + \frac{1-n}{V_{pl}}
\]

where \(V_{pf}\) is the velocity in the pore fluid and \(V_{pl}\) is the compressional wave velocity for the intact material determined in the laboratory. The compressional wave velocities may be raised appreciably by the presence of water (Grainger et al. 1973).

D’Andrea et al. (1965) compared the compressional wave velocities measured for a range of American rocks with their uniaxial compressive strengths. Whilst a correlation can be obtained, a similar comparison for a wider range of British rocks showed that a broad range of velocities can be obtained for weak rocks of fairly uniform strength and, conversely, for very strong rocks, velocity remains fairly constant despite large variations in strength values (McCann et al. 1990).

**Table 3. Velocities of compressional waves of some common rocks**

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>(V_p) (km s(^{-1}))</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Igneous rocks</strong></td>
<td></td>
</tr>
<tr>
<td>Basalt</td>
<td>5.1–6.4</td>
</tr>
<tr>
<td>Dolerite</td>
<td>5.8–6.6</td>
</tr>
<tr>
<td>Gabbro</td>
<td>6.5–7.0</td>
</tr>
<tr>
<td>Granite</td>
<td>5.5–6.1</td>
</tr>
<tr>
<td><strong>Metamorphic rocks</strong></td>
<td></td>
</tr>
<tr>
<td>Gneiss</td>
<td>3.5–7.0</td>
</tr>
<tr>
<td>Marble</td>
<td>3.7–6.9</td>
</tr>
<tr>
<td>Quartzite</td>
<td>5.6–6.1</td>
</tr>
<tr>
<td>Schist</td>
<td>3.5–5.7</td>
</tr>
<tr>
<td>Slate</td>
<td>3.5–5.4</td>
</tr>
<tr>
<td><strong>Sedimentary rocks</strong></td>
<td></td>
</tr>
<tr>
<td>Gypsum</td>
<td>2.0–3.5</td>
</tr>
<tr>
<td>Limestone</td>
<td>2.8–7.0</td>
</tr>
<tr>
<td>Sandstone</td>
<td>1.4–4.4</td>
</tr>
<tr>
<td>Shale</td>
<td>2.1–4.4</td>
</tr>
<tr>
<td><strong>Unconsolidated deposits</strong></td>
<td></td>
</tr>
<tr>
<td>Alluvium</td>
<td>0.3–0.6</td>
</tr>
<tr>
<td>Sands and gravels</td>
<td>0.3–1.8</td>
</tr>
<tr>
<td>Clay (wet)</td>
<td>1.5–2.0</td>
</tr>
<tr>
<td>Clay (sandy)</td>
<td>2.0–2.4</td>
</tr>
</tbody>
</table>
Rather than attempt to relate empirically some velocities, measured either in situ or in the laboratory, with individual geotechnical parameters, a better approach is to use velocity measurements as a measure of rock quality taking into account mass rock properties, including fracturing. A relationship between sonic velocity and Bieniawski's (1976) rock mass rating has been obtained for Dalradian rocks (McCann et al. 1990) but more work is needed to confirm this approach for other rock types.

Shear wave velocity measurements have been used by Seed et al. (1983) as one of the parameters for the determination of liquefaction potential. The use of shear wave velocities, in this way, implies a correlation with SPT N value and with cone penetration resistance as these two parameters can also be used for the same purpose.

When both the compressional and shear wave velocities are known, it is possible to calculate the dynamic elastic properties of a material, such as Young's modulus or Poisson's ratio. However, dynamically derived values tend to be higher than statically derived ones because of the different strain levels at which the two sets of measurements are made.

**Resistivity measurements**

The resistivity of rocks and soils varies over a wide range. Since most of the principal rock forming minerals are practically insulators, the resistivity of rocks and soils is determined by the amount of conducting mineral constituents and the water content in the pores. The latter condition is by far the dominant factor and, in fact, most rocks and soils conduct an electric current only because they contain water (Kollar 1969). As the amount of water present is influenced by the porosity of a rock, the resistivity provides a measure of its porosity. For example, in granular materials in which there are no clay minerals, the relationship between the resistivity ($\rho$), on the one hand, and the density of the pore water ($p_w$), the porosity ($n$) and the degree of saturation ($S_r$), on the other, is as follows (Keller 1966)

$$\rho = a p_w n^{-x} S_r^{-y}$$  \hspace{1cm} (17)

where $a$, $x$ and $y$ are variables ($x$ ranges from 1.0 for sand to 2.5 for sandstone; $y$ is approximately 2.0 when the degree of saturation is greater than 30%). If clay minerals do occur in sands and sandstones, then the resistivity of the pore water is significantly reduced by ion exchange between the latter and the clay minerals so that the above relationship becomes invalid. For those formations which occur below the water table and therefore are saturated, the above expression becomes

$$\rho = a p_w n^{-x} S_r^{-y}$$  \hspace{1cm} (18)

since $S_r = 1$ (that is, 100%). In fact, if two rocks have the same water content and one has a porosity of 1% and the other of 30%, then the former is ten times as resistive as the latter.

The widely differing resistivity of the various types of impregnating water can cause variations in the resistivity of soil and rock formations ranging from a few tenths of an ohm-metre to hundreds of ohm-metres, as can be seen from Table 4. Resistivity values for some common rock types are given in Table 5 but, as indicated above, values can vary widely depending upon the degree of saturation or the moisture content.

Resistivity measurements have been successfully used to locate intrusions of saline groundwater or the alteration in groundwater salinity with time (Bugg & Lloyd 1976; Oteri 1983) and, under favourable conditions, may be applicable to the determination of the boundaries of pollutant plumes in groundwater. In landslip investigations, the observation of variations in soil moisture content may be important and this can be successfully monitored using resistivity measurements. Denness et al. (1975) described seasonal fluctuations in moisture content for a landslip at Charmouth in Dorset. Forster & McCann (in press) have described the use of resistivity and other geophysical methods in reconnaissance investigations for landslides.

Resistivity methods can be applied to a variety of problems by varying the electrode configuration in an appropriate manner. Resistivity depth soundings are used to establish a correlation between resistivity and lithological layers. Horizontal profiling is used to determine the variations in the apparent resistivity in a horizontal direction at a pre-selected depth. The application of the different electrode configurations in

<table>
<thead>
<tr>
<th>Table 4. Resistivity of some types of natural water</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of water</td>
</tr>
<tr>
<td>----------------------------------------------------</td>
</tr>
<tr>
<td>Meteoric water, derived from precipitation</td>
</tr>
<tr>
<td>Surface waters, in districts of igneous rocks</td>
</tr>
<tr>
<td>Surface waters, in districts of sedimentary rocks</td>
</tr>
<tr>
<td>Ground water, in areas of igneous rocks</td>
</tr>
<tr>
<td>Ground water, in areas of sedimentary rocks</td>
</tr>
<tr>
<td>Sea water</td>
</tr>
</tbody>
</table>
Table 5. Resistivity values of some common rock types

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Resistivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Topsoil</td>
<td>5–50</td>
</tr>
<tr>
<td>Peat and clay</td>
<td>8–50</td>
</tr>
<tr>
<td>Clay, sand and gravel mixtures</td>
<td>40–250</td>
</tr>
<tr>
<td>Saturated sand and gravel</td>
<td>40–100</td>
</tr>
<tr>
<td>Moist to dry sand and gravel</td>
<td>100–3000</td>
</tr>
<tr>
<td>Mudstones, marls and shales</td>
<td>8–100</td>
</tr>
<tr>
<td>Sandstones and limestones</td>
<td>100–1000</td>
</tr>
<tr>
<td>Crystalline rocks</td>
<td>200–10,000</td>
</tr>
</tbody>
</table>

depth soundings and horizontal traversing is described in most geophysical text books (e.g. Kearey & Brooks 1984).

For depth soundings the data obtained are plotted as a graph of apparent resistivity against a function of electrode spacing, the exact function being dependent upon the particular electrode configuration used. However, the apparent resistivity is a measure of the effects of all the layers between the maximum depth of penetration and the surface. For this reason, interpretation becomes more difficult as the number of beds increases. Three or four distinct layers are about the maximum number for accurate interpretation unless other subsurface information is available.

Ground resistivities are never uniform so that a single depth determination cannot be relied upon. Readings may be repeated at the same location but with the electrode spread orientated in a different direction and additional sets of measurements are often taken nearby.

For all depth determinations from resistivity soundings it is assumed that there is no change in resistivity laterally. This is not the case in practice. Indeed, sometimes the lateral change is greater than that occurring with increasing depth and so corrections have to be applied for these lateral effects when depth determinations are made. In some cases, the lateral effect is the major feature of the curve and depth interpretation can be very inaccurate.

The data from a constant separation survey may be used to construct a contour map of lines of equal resistivity. From borehole and depth probe interpretations the relationship between overburden thickness and apparent resistivity is derived and if it seems reasonably constant over the area the resistivity contours may be regarded as rough depth contours.

In a fully saturated rock a fundamental empirical relationship exists between the electrical and hydrogeological properties which involves the concept of the formation resistivity factor, $F_a$, defined as

$$F_a = \frac{\rho_0}{\rho_w}$$  (19)

where $\rho_0$ is the resistivity of the saturated rock and $\rho_w$ is the resistivity of the saturated solution. In a clean rock, that is, one in which the electrical current passes through the interstitial electrolyte during testing with the rock mass acting as an insulator, the formation resistivity factor is closely related to the porosity. Worthington (1973) showed that the formation resistivity factor was related to the true formation factor, $F$, by the expression

$$F = \frac{\rho_A F_a}{\rho_A - F_a \rho_w}$$  (20)

in which $\rho_A$ is a measure of the effective resistivity of the rock matrix. In clean rock $\rho_A$ is infinitely large, consequently $F = F_a$. Generally $F$ is related to the porosity ($n$) by the equation

$$F = \frac{a}{n^m}$$  (21)

where $a$ and $m$ are constants for a given formation (for instance, according to Barker & Worthington (1973), in the Bunter Sandstone of the Fylde region, Lancashire, $a = 1.05$ and $m = 1.47$). It is possible, also, to determine the formation factor using a seismic survey. For example, Barker & Worthington (1973) showed that the relationship between the compressional wave velocity, $V_p$ and true formation factor ($F$) in the Bunter Sandstone took the form

$$V_p = 2.07 \log_{10} F + 0.35.$$  (22)

The true formation factor in certain formations has also been shown to be broadly related to intergranular permeability ($k_g$) by the expression

$$F = \frac{b}{k_g^n}$$  (23)

where $b$ and $n$ are constants for a given formation. The quoted values for the same Bunter Sandstone were 3.3 and 0.17 respectively. In sandstones in which intergranular flow is important, the above expressions can be used to estimate hydraulic conductivity and thence, if the thickness of the aquifer is known, its transmissivity.

According to Worthington & Griffiths (1975) it appears that quantitative geophysical investigations of variations in transmissivity and hydraulic conductivity are economical only where intensive development of a sandstone aquifer is planned. The techniques are not useful in highly indurated sandstones, where the intergranular permeabilities are less than $1.0 \times 10^{-7}$ m s$^{-1}$, and the flow is controlled by fissures. Similarly, these methods are of little value in multilayered aquifers, except, perhaps, where the thickness of the different layers are fortuitously distributed so as to allow a complete and definite geophysical interpretation.

Electromagnetic measurements

Electromagnetic methods are widely used in mineral exploration and are being increasingly applied in site investigation. Electromagnetic energy is introduced
into the ground by inductive coupling and sensed by the receiver in the same way. The method is used in a similar way to constant spacing resistivity traversing and provides an apparent conductivity which depends only on the separation of the transmitting and receiving coils and the conductivity of the ground. For horizontally-layered ground, the current in each layer is fixed and its contribution to the total reading can be calculated. Hence, layer thicknesses and conductivities can be determined using simple formulae.

A fixed coil instrument such as the Geonics EM 31 has a coil separation of 4 m and penetrations of up to 6 m can be achieved. Penetrations of up to about 50 m can be obtained with instruments where the coils are separated (e.g. the Geonics EM 34) and can be moved apart to a distance of 40 m.

The method has been used for the detection of cavities and air-filled fissures (Culshaw et al. 1987).

Magnetic measurements

All rocks are magnetized to a lesser or greater extent by the Earth’s magnetic field. As a consequence, in magnetic prospecting accurate measurements are made of the anomalies produced in the local geomagnetic field by this magnetization. The intensity of magnetization and hence the amount by which the Earth’s magnetic field is changed locally, depends on the magnetic susceptibility of the material concerned. In addition to the magnetism induced by the Earth’s field, rocks possess a permanent magnetism that depends upon their history.

Some minerals, for example, quartz and calcite are demagnetized reversely to the field direction and therefore have negative susceptibility. They are described as diamagnetic. Paramagnetic minerals, which are the majority, are magnetized along the direction of magnetic field so that their susceptibility is positive. The susceptibility of the ferromagnetic minerals, such as magnetite, ilmenite, pyrrhotite and hematite, is a very complicated function of the field intensity. The magnitude of susceptibility of the ferromagnetic minerals is 10 to 10² times the order of susceptibility of the other minerals.

If the magnetic field ceases to act on a rock, then the magnetization of paramagnetic and diamagnetic minerals dissappears. However, in ferromagnetic minerals the induced magnetization is diminished only to a certain value. This residuum is called remanent magnetization and is of great importance in rocks. All igneous rocks have a very high remanent magnetization acquired as they cooled down in the Earth’s magnetic field. In the geological past, during sedimentation in water, grains of magnetic materials were orientated by ancient geomagnetic fields so that some sedimentary rocks show stable remanent magnetization.

The strength of a magnetic field is measured in oersteds, but as the average strength of the Earth’s magnetic field is about 0.5 oersted the variations associated with magnetized rock formations are very much smaller than this. Consequently the practical unit on magnetic surveying is the gamma which is 10⁻⁵ oersted.

Aeromagnetic surveying has almost completely supplanted ground surveys for regional reconnaissance purposes. Accurate identification of the plan position of the aircraft for the whole duration of the magnetometer record is essential. The base map with transcribed magnetic values is contoured at 5 or 10 gamma intervals to produce an aeromagnetic map.

The aim of most ground surveys is to produce accurate magnetic profiles and maps of anomalies to enable the form and depth of the causative magnetized body to be estimated.

Magnetic susceptibility also can be determined with a proton magnetometer in the laboratory using small rock or soil plugs. The equipment is very simple to use and the directly read results can then be used to assist interpretation of magnetic maps.

Magnetic methods have limited applicability in site investigation, but have been used successfully to detect a clay-filled swallow hole (McDowell 1975) and disused mineshafts (Dearman et al. 1977).

Gravity measurements

The Earth’s gravity field varies according to the density of the subsurface rocks but at any particular locality its magnitude is also influenced by latitude, elevation, neighbouring topographical features and the tidal deformation of the Earth’s crust. The effects of these latter factors have to be eliminated in any gravity survey where the aim is to measure the precise variations in acceleration due to gravity. These can then be used to construct isogals on a gravity map. The acceleration at the Earth’s surface due to gravity is about 980 gals (one gal is an acceleration of 10 mm s⁻²), but the gravity variations which are measured are extremely small, of the order of 10⁻⁴ gal. Hence, the unit of measurement is the milligal. Modern gravity meters used for exploration, measure not the absolute value of the acceleration due to gravity but the small differences in this value between one location and the next. Gravity methods are mainly used in regional reconnaissance surveys to reveal anomalies which may be subsequently investigated by other methods. Since the gravitational effects of geological bodies are proportional to the contrast in density between them and their surroundings, gravity methods are particularly suitable for the location of structures in stratified formations.

Eaton et al. (1965) reviewed the application of gravity measurements to reconnaissance mapping of sub-alluvial bedrock topography, the estimation of depth to bedrock and the in situ measurement of aquifer porosity. They concluded that gravimetric
mapping of an alluviated terrain underlain by relatively dense bedrock provided a rapid means of obtaining a qualitative picture of buried bedrock topography and that the information would be of use to the hydrogeologist. Precise gravity surveys in areas where unconsolidated sediments overlie uniformly dense, crystalline bedrock can yield figures on depth to bedrock with an average error of only ±10%, provided that data exist for establishing depth control in some part of the area. Where depth to bedrock is known independently, detailed gravity measurements, coupled with laboratory determinations of average grain density, allow calculation of an average in situ porosity for an aquifer overlying bedrock. The basis for such a calculation depends on the expression:

\[
\text{porosity} = \frac{\text{grain density} - \text{bulk density}}{\text{grain density} - \text{percent saturation}}
\]

where the bulk density is determined from an analysis of the gravity anomaly.

The greatest weakness in the gravity method lies in the fact that small geological changes are difficult to detect. It is therefore, like the magnetic method, of little value in most detailed problems of geological or hydrogeological exploration. In site investigation, the method has been used to detect cave systems (Neumann 1977), thickening of superficial deposits (Cornwell 1985), and may be of use for the detection of mineshafts (McCann et al. 1987)

### Downhole measurements

As summarized in Table 6, many of the surface geophysical methods can be adapted to produce geophysical logs of boreholes. This can assist with correlation between boreholes and also may provide information on the lithological and geotechnical character of the ground, particularly its intactness.

#### Table 6. Borehole geophysical logging methods (after Johnson 1968)

<table>
<thead>
<tr>
<th>Method</th>
<th>Uses</th>
<th>Recommended conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Electronic logging:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Single-electrode resistance.</td>
<td>Determining depth and thickness of thin beds. Identification of rocks, provided general lithologic information is available, and correlation of formations. Determining casing depths.</td>
<td>Fluid-filled hole. Fresh mud required. Hole diameter less than 200 to 250 mm. Log only in uncased holes.</td>
</tr>
<tr>
<td>Short normal (electrode spacing of 400 mm).</td>
<td>Picking resistivity of the tops of resistive beds. Determining resistive beds. The invaded zone. Estimating porosity of formations (deeply invaded and thick interval). Correlation and identification, provided general lithologic information is available.</td>
<td>Fluid-filled hole. Fresh mud. Ratio of mud resistivity to formation-water resistivity should be 0.2 to 4. Log only in uncased part of hole.</td>
</tr>
<tr>
<td>Long normal (electrode spacing of 1.6 m).</td>
<td>Determining true resistivity in thick beds where mud invasion is not too deep. Obtaining data for calculation of formation-water resistivity.</td>
<td>Fluid-filled hole. Ratio of mud resistivity to formation-water resistivity should be 0.2 to 4. Log only in uncased part of hole.</td>
</tr>
<tr>
<td>Deep lateral (electrode spacing approximately 5.8 m).</td>
<td>Determining true resistivity where mud invasion is relatively deep. Locating thin beds.</td>
<td>Fluid-filled uncased hole. Fresh mud. Formation should be of thickness different from electrode spacing and should be free of thin limestone beds.</td>
</tr>
<tr>
<td>Limestone sonde (electrode spacing of 0.8 m).</td>
<td>Detecting permeable zones and determining porosity in hard rock. Determining formation factor in situ.</td>
<td>Fluid-filled uncased hole. May be salty mud. Uniform hole size. Beds thicker than 1.5 m.</td>
</tr>
<tr>
<td>Microlog</td>
<td>Determining permeable beds in hard or well-consolidated formations. Detailing beds in moderately consolidated formations. Correlation in hard-rock country. Determining formation factor in situ in soft or moderately consolidated formations. Detailing very thin beds.</td>
<td>Fluid required in hole. Log only in uncased part of hole. Bit-size hole (caved sections may be logged, provided hole enlargements are not too great).</td>
</tr>
</tbody>
</table>
### Table 6. (cont.)

<table>
<thead>
<tr>
<th>Method</th>
<th>Uses</th>
<th>Recommended conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Microlaterolog</strong></td>
<td>Determining detailed resistivity of flushed formation at wall of hole when mud-cake thickness is less than three-eighths inch in all formations. Determining formation factor and porosity. Correlation of very thin beds.</td>
<td>Fluid-filled uncased hole. Thin mud cake. Salty mud permitted.</td>
</tr>
<tr>
<td><strong>Induction logging</strong></td>
<td>Determining true resistivity, particularly for thin beds (down to about 0.6 m thick) in wells drilled with comparatively fresh mud. Determining resistivity of formations in dry holes. Logging in oil-base muds. Defining lithology and bed boundaries in hard formations. Detection of water-bearing beds.</td>
<td>Fluid-filled or dry uncased hole. Fluid should not be too salty.</td>
</tr>
<tr>
<td><strong>Spontaneous potential.</strong></td>
<td>Helps delineate boundaries of many formations and the nature of these formations. Indicating approximate chemical quality of water. Indicate zones of water entry in borehole. Locating cased interval. Detecting and correlating permeable beds.</td>
<td>Fluid-filled uncased hole. Fresh mud.</td>
</tr>
<tr>
<td><strong>Sonic logging</strong></td>
<td>Logging acoustic velocity for seismic interpretation. Correlation and identification of lithology. Reliable indication of porosity in moderate to hard formations; in soft formations of high porosity it is more responsive to the nature rather than quantity of fluids contained in pores.</td>
<td>Not affected materially by type of fluid, hole size, or mud invasion.</td>
</tr>
<tr>
<td><strong>Radioactive logging:</strong></td>
<td>Differentiating shale, clay and marl from other formations. Correlations of formations. Measurements of inherent radioactivity in formations. Checking formation depths and thicknesses with reference to casing collars before perforating casing. For shale differentiation when holes contain very salty mud. Radioactive tracer studies. Logging dry or cased holes. Locating cemented and cased intervals. Logging in oil-base muds. Locating radioactive ores. In combination with electric logs for locating coal or lignite beds.</td>
<td>Fluid-filled or dry cased or uncased hole. Should have appreciable contrast in radioactivity between adjacent formations.</td>
</tr>
<tr>
<td><strong>Gamma ray</strong></td>
<td>Delineating formations and correlation in dry or cased holes. Qualitative determination of shales, tight formations, and porous sections in cased wells. Determining porosity and water content of formations, especially those of low porosity. Distinguishing between water- or oil-filled and gas-filled reservoirs. Combining gamma-ray log for better identification of lithology and correlation of formations. Indicating cased intervals. Logging in oil-base muds.</td>
<td>Fluid-filled or dry cased or uncased hole. Formations relatively free from shaly material. Diameter less than 150 mm for dry holes. Hole diameter similar throughout.</td>
</tr>
<tr>
<td><strong>Neutron</strong></td>
<td>Locating approximate position of cement behind casing. Determining thermal gradient. Locating depth of lost circulation. Locating active gas flow. Used in checking depth and thickness of aquifers. Locating fissures and solution openings in open holes and leaks or perforated sections in cased holes. Reciprical-gradient temperature log may be more useful in correlation work.</td>
<td>Cased or uncased hole. Can be used in empty hole if logged at very slow speed, but fluid preferred in hole. Fluid should be undisturbed (no circulation) for 6 to 12 hours minimum before logging; possibly several days may be required to reach thermal equilibrium.</td>
</tr>
<tr>
<td><strong>Temperature logging</strong></td>
<td>Locating approximate position of cement behind casing. Determining thermal gradient. Locating depth of lost circulation. Locating active gas flow. Used in checking depth and thickness of aquifers. Locating fissures and solution openings in open holes and leaks or perforated sections in cased holes. Reciprical-gradient temperature log may be more useful in correlation work.</td>
<td>Cased or uncased hole. Can be used in empty hole if logged at very slow speed, but fluid preferred in hole. Fluid should be undisturbed (no circulation) for 6 to 12 hours minimum before logging; possibly several days may be required to reach thermal equilibrium.</td>
</tr>
</tbody>
</table>
**Table 6. (cont.)**

<table>
<thead>
<tr>
<th>Method</th>
<th>Uses</th>
<th>Recommended conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fluid-conductivity logging</td>
<td>Location point of entry of different quality water through leaks or perforations in casing or opening in rock hole. (Usually fluid resistivity is determined and must be converted to conductivity.) Determining quality of fluid in hole for improved interpretation of electric logs. Determining fresh-water–salt-water interface.</td>
<td>Fluid required in cased or uncased hole. Temperature log required for quantitative information.</td>
</tr>
<tr>
<td>Fluid-velocity logging</td>
<td>Locating zones of water entry into hole. Determining relative quantities of water flow into or out of these zones. Determine direction of flow up or down in sections of hole. Locating leaks in casing. Determine approximate permeability of lithologic sections penetrated by hole, or perforated section of casing.</td>
<td>Fluid-filled cased or uncased hole. Injection, pumping, flowing, or static (at surface) conditions. Flange or packer units required in large diameter holes. Capiler (section gage) logs required for quantitative interpretation.</td>
</tr>
<tr>
<td>Casing collar locator</td>
<td>Locating position of casing collars and shoes for depth control during perforating. Determining accurate depth references for use with other types of logs.</td>
<td>Cased hole.</td>
</tr>
<tr>
<td>Caliper survey</td>
<td>Determining hole or casing diameter. Indicates lithologic character of formations and coherency of rocks penetrated. Locating fractures, solution openings, and other cavities. Correlation of formations. Selection of zone to set a packer. Useful in quantitative interpretation of electric, temperature, and radiation logs. Used with fluid-velocity logs to determine quantities of flow. Determining diameter of underreamed section before placement of gravel pack. Determining diameter of hole use in computing volume of cement seal to annular space. Evaluating the efficiency of explosive development of rock wells. Determining construction information on abandoned wells.</td>
<td>Fluid-filled or dry cased or uncased hole. (In cased holes does not give information on beds behind casing.)</td>
</tr>
<tr>
<td>Dipmeter survey</td>
<td>Determining dip angle and dip direction (from magnetic north) in relation to well axis in the study of geological structure. Correlation of formations.</td>
<td>Fluid-filled uncased hole. Carefully picked zones needing survey, because of expense and time required. Directional survey required for determination of true dip and strike (generally obtained simultaneously with dipmeter curves).</td>
</tr>
<tr>
<td>Directional (inclinometer)</td>
<td>Locating points in a hole to determine deviation from the vertical. Determining true depth. Determining possible mechanical difficulty for casing installation or pump operation. Determining true dip and strike from dipmeter survey.</td>
<td>Fluid-filled or dry uncased hole.</td>
</tr>
<tr>
<td>Magnetic logging.</td>
<td>Determining magnetic field intensity in borehole and magnetic susceptibility of rocks surrounding hole. Studying lithology and correlation, especially in igneous rocks.</td>
<td>Fluid-filled or dry uncased hole.</td>
</tr>
</tbody>
</table>

Resistivity values from borehole logging have been used to determine permeability and transmissivity (Croft 1971, Kelly 1977), while other logs such as induction logging and spontaneous potential logging can be used for determining various aspects of geology and lithology. For example, induction logging provides an accurate and detailed record of strata over a wide range of conductivity values. Its focusing ability has...
excellent resolving power, and shows almost no distortion opposite thin beds. This is the principle advantage of induction logging over conventional electrical logging, that is, its ability to locate beds as thin as only 50 mm. Also, the depths to contacts between beds can be determined accurately.

Spontaneous potential logging measures the very small differences in electrical potential which exist at the boundaries of permeable rock units especially between such strata and less permeable beds. Permeable sandstones and limestones show large spontaneous potential. If sandstones and shales are interbedded, then the spontaneous potential log has numerous troughs separated by sharp or rounded peaks, the widths of which vary in proportion to the thickness of the sandstones. Where there are no sharp contrasts in permeability between adjacent beds of rock, spontaneous potential curves lack relief and are of little value.

Spontaneous potential logs are frequently recorded at the same time as resistivity logs. Interpretation of both sets of curves yields precise data on the depth, thickness and position in the sequence of the beds penetrated by the drillhole. They also enable a semi-quantitative assessment of lithological and hydrogeological characteristics to be made.

Sonic logging produces a downhole sonic velocity log of the borehole wall. Some logs can be interpreted to give assessments of rock properties such as porosity, strength, degree of weathering or amount of fracturing. However, one of the most appropriate uses in site investigation may be in the determination of rock mass quality, along the lines of the mechanically-based systems of Bieniawski (1976) and Barton et al. (1974).

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