DRAINAGE AND IRRIGATION DEPARTMENT
MINISTRY OF AGRICULTURE
GOVERNMENT OF MALAYSIA

JABATAN PARIT DAN TALIAIR
KEMENTERIAN PERTANIAN
KERAJAAN MALAYSIA

GROUNDWATER RESOURCES FOR AGRICULTURAL USE IN MALAYSIA

EXPLORATION & DEVELOPMENT MANUAL

Sir M. MacDonald & Partners Limited
Consulting Engineers
Demeter House
Station Road
Cambridge CB1 2RS
England

ERRATA SHEET

Chapter 2

p.14, para.2, sentence 4 should read: "in order to avoid electrode polarisation".
p.14, para.3, 3rd line should read: "As an illustration; the resistance of metallic electronic conductors tends to zero near absolute zero (0°K) temperature, and ..."
p.20, para.5, 2nd line should read: "$K = 2\pi a$".
p.22, Figure 2.4(b) and 2.4(d) are from: R.D. Barker, 1979. Signal Contribution Sections and their use in Resistivity Studies: Geophysical Journal of the Royal Astronomical Society. Vol. 59, No. 1, p.123-129.

Spelling/Typing Mistakes

p.13, para. 2, line 1 - "relevant"
p.14, para. 4, line 4 - "pore"
p.43, 2.6.4., para.1, line 3 - "layer"
p.43, 2.6.4., para.2, line 4 - "function"
p.46, 2.7.4., para 1 line 10 - "buried"

Chapter 4

p.105, Figure 4.7. The lithologs should be titled, from left to right, PT9, PM3, PM2.
p.107, para 2. Equation (2) is divided through by Q to give Equation (3).
p.122, Equation (17) should read:

$$SW = \frac{Q}{2\pi T} \quad \ln \quad \frac{r_e}{r_w}$$

Equation (4)* should read:

$$SW = \frac{2.303Q}{2\pi T} \,.\, \log \frac{r_e}{r_w}$$

p.125. In the boundary conditions applicable to derivation of Theis formula, r tends to ∞ (infinity).
p.126. Equation (23) should read:

$$u = \frac{r^2 S}{4Tt}$$

Equation (25) should read:

$$\log \frac{r^2}{t} = \log \frac{4T}{S} + \log u$$

In both cases, storage coefficient is upper case S. Lower case s in Equation (24) is drawdown.

p.130. Equation preceding Equation (30) should show Δs (delta S).
p.136, last para, Figure 4.8 should read 4.9.

Spelling/Typing Mistakes

p.134, para 3, line 4 - "infinitesimally"
p.132, para 3, line 4 - "phenomena"
p.116, para 4, line 7 - "subtractive"
p.110, para 3, line 5 - "equation"
p.109, para 1, line 7 - "approximation", "situation"
line 9 - "entails"
line 17 - "necessary"
p.104, para 1, line 1 - "fissured".

EXPLORATION & DEVELOPMENT MANUAL

CONTENTS

Page No.

3 FORMATION IDENTIFICATION AND TESTING

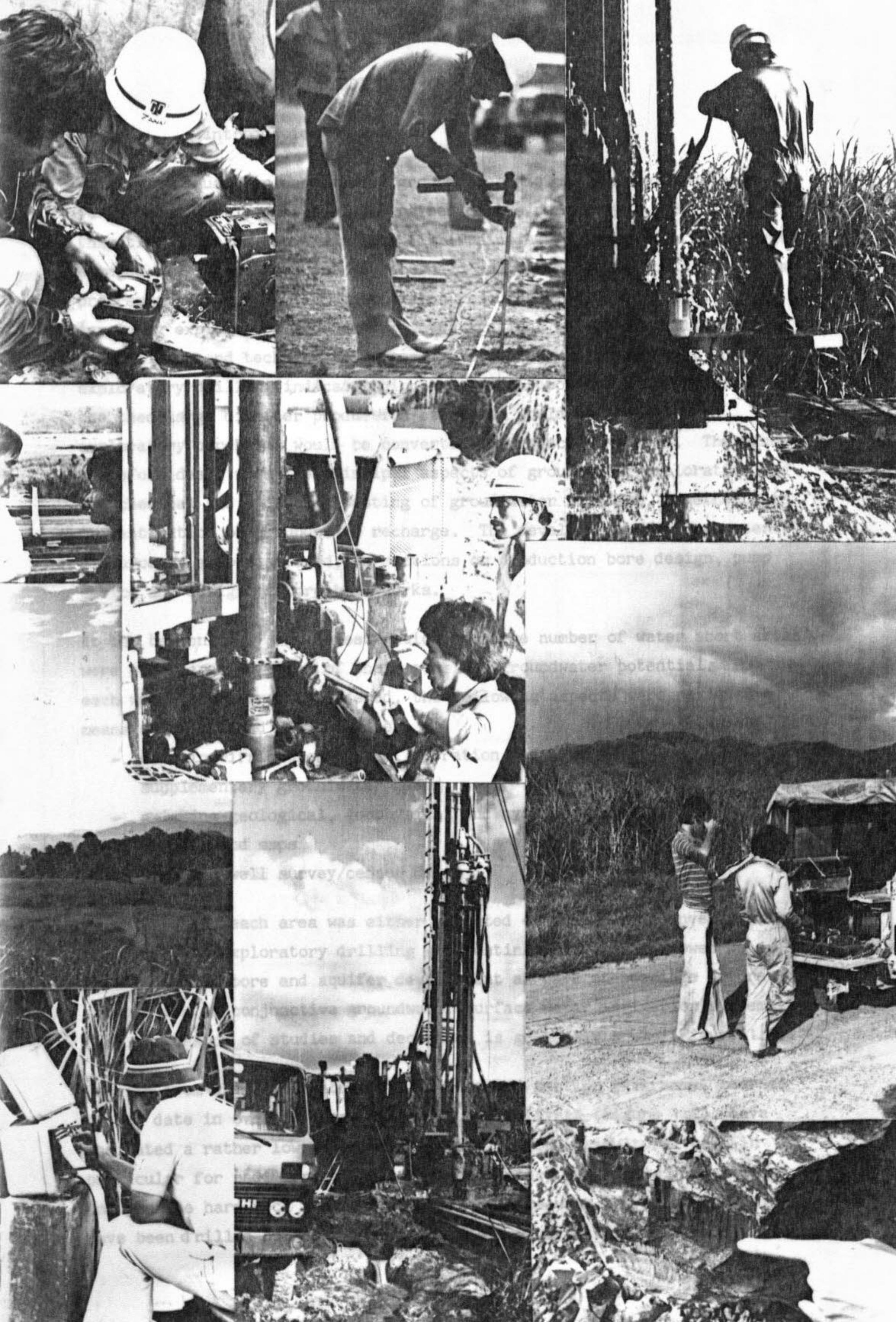
4 AQUIFER CHARACTERISTICS AND WELL TESTING

5 PRODUCTION BORE DESIGN

FIGURES

Page No.

TABLES



1. PRELIMINARY STUDIES

1.1 Introduction

This is a manual for undertaking groundwater exploration and development. It is based partly on experience gained in the period 1981-1983; during this time, Jabatan Parit dan Tali Air (Drainage and Irrigation Department) engaged the consultants Sir M. MacDonald and Partners Ltd. to assist in a programme of assessment of groundwater resources for agricultural use, involving exploratory groundwater investigations using JPT drilling rigs and drilling and technical personnel. It was the intention that where exploratory drilling indicated high groundwater potential for agricultural use then large diameter production boreholes would be installed or exploratory boreholes would be converted to production status. The manual therefore deals with the principal aspects of groundwater exploration and the chemical and hydraulic testing of groundwater aquifers, together with the estimation of groundwater recharge. The development of groundwater resource is also discussed, in sections on production bore design, pump choice and design of bore head works.

At the beginning of the investigation, a large number of water short areas were considered in terms of their possible groundwater potential. For each water short area considered, the following aspects were reviewed by means of office and field studies:

- existing irrigation system, operation and likely supplementary groundwater needs;
- existing geological, geophysical and hydrogeological reports and maps
- bore and well survey/census data.

On this basis, each area was either rejected or assigned an investigative priority. An exploratory drilling and testing phase was followed as appropriate by bore and aquifer development as part of possible future groundwater or conjunctive groundwater-surface water irrigation systems. This progession of studies and decisions is generally shown (Figure 1.1).

Results from the JPT exploratory drilling and testing programme, carried out to date in over 13 years in Peninsular Malaysia (Figure 1.2), have indicated a rather low groundwater potential for agricultural use, in particular for padi irrigation. Metasedimentary sand-shale, limestone and granite hard rock aquifers, eluvial granite-wash and alluvial aquifers have been drilled but only in the thick alluvium of north-northeast Kelantan

FIGURE 1.1 GROUNDWATER EXPLORATION AND PRODUCTION

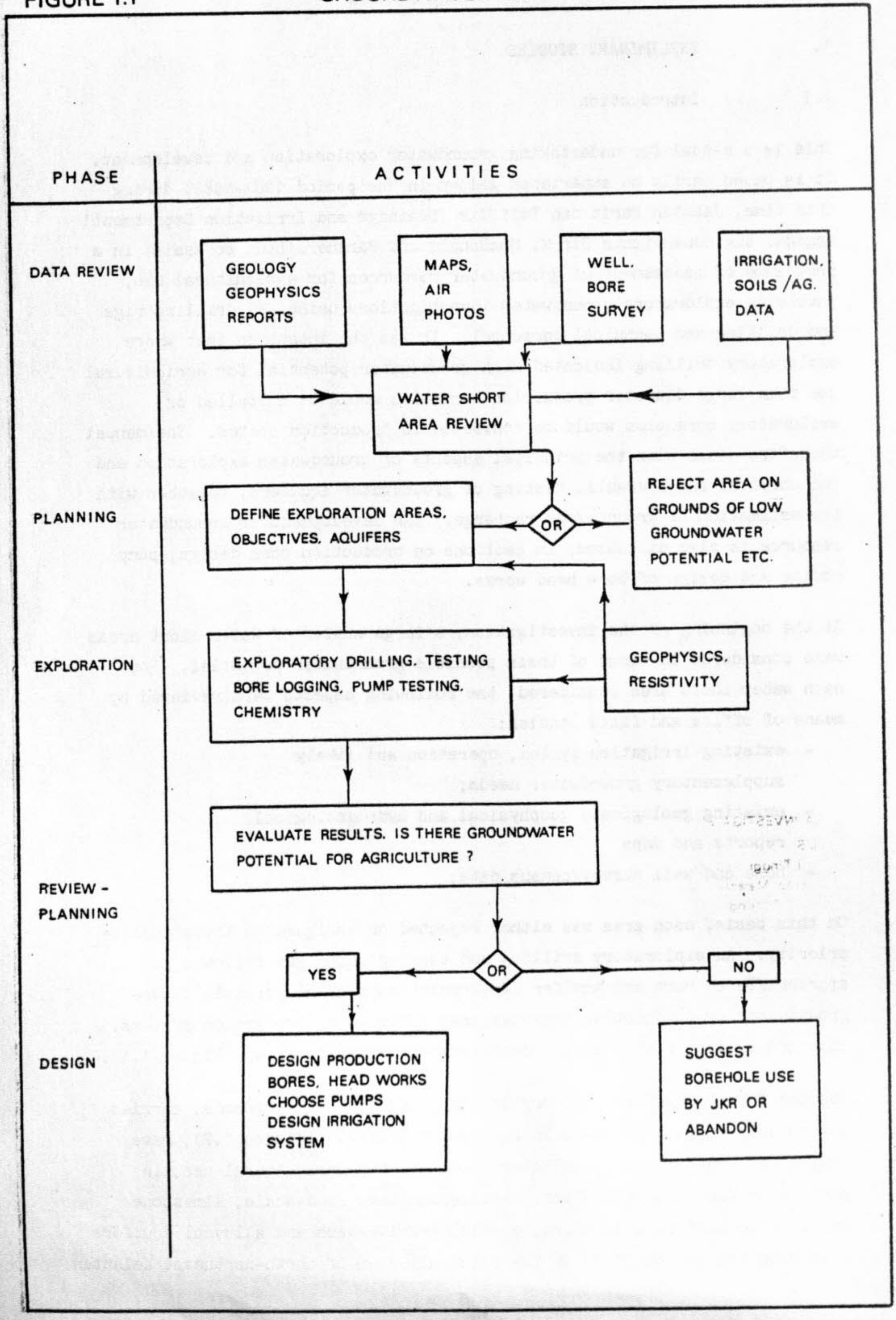
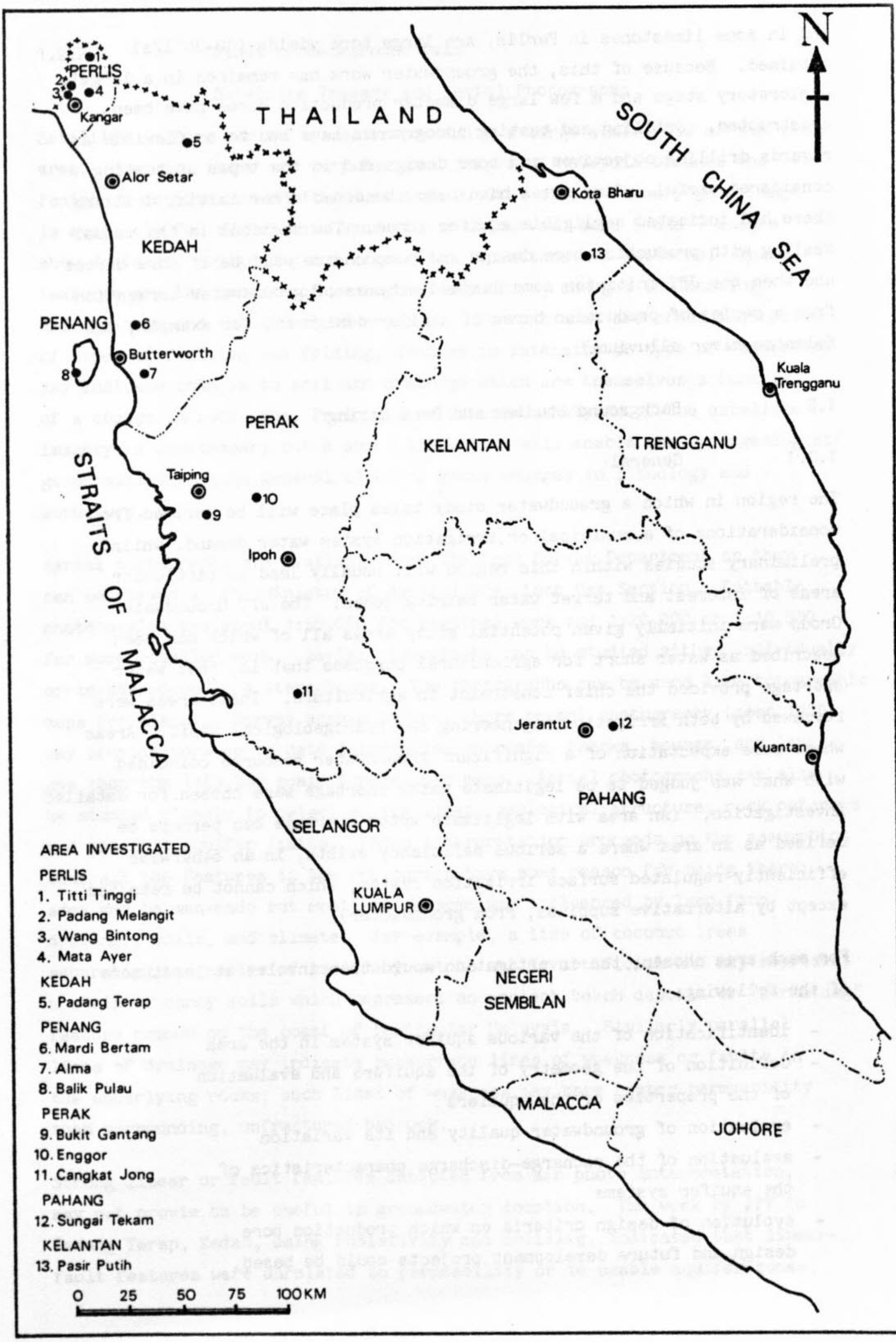


FIGURE 1.2 LOCATION OF DRILLING INVESTIGATIONS



and in some limestones in Perlis, are large bore yields (10-30 l/s) obtained. Because of this, the groundwater work has remained in a largely exploratory stage and a few large diameter production bores have been constructed. Drilling and testing pprogrammes have had to be flexible as regards drilling objectives and bore design and in the types of testing considered useful. Some areas have been abandoned after initial drilling there had indicated negligible aquifer zones. The sections in the manual dealing with production bore design and pump choice will be of more direct use when the JPT initiates some limited schemes of groundwater irrigation from a series of production bores of similar design in, for example, the Kelantan river alluvium.

1.2 Background Studies and Bore Siting

1.2.1 General

The region in which a groundwater study takes place will be derived from considerations of a municipal or irrigation system water demand, whilst preliminary studies within this region will usually lead to particular areas of interest and target water bearing zones. The JPT Groundwater Group were initially given potential study areas all of which had been described as water short for agricultural purposes that is, that water shortage provided the chief constraint to agriculture. These areas were reviewed by both irrigation engineering and hydrogeological staff. Areas where some expectation of a significant groundwater resource coincided with what was judged to be legitimate water shortage were chosen for detailed investigation. (An area with legitimate water shortage can perhaps be defined as an area where a serious deficiency exists, in an otherwise efficiently regulated surface irrigation system, which cannot be remedied except by alternative supplies, from groundwater).

For each area chosen, the investigation would then involve at least some of the following:

- identification of the various aquifer system in the area
- definition of the geometry of the aquifers and evaluation of the properties of the aquifers
- evaluation of groundwater quality and its variation
- evaluation of the recharge-discharge characteristics of the aquifer systems
- evolution of design criteria on which production bore design and future development projects could be based.

1.2.2 Study of Background Data

Satellite Imagery and Aerial Photographs

Satellite imagery can be of use in providing a large scale view of the study area. However, there are very few good images of Malaysia because the Peninsula is often covered by more than 3/10ths cloud. A very good image is available for NW Malaysia whose false colour and band 5 and 7 images show drainage, topography and vegetation lineations and patterns. These features are the product of geological structure, changes in lithology and drainage and the effect of erosion. The patterns are often indicative of lines of faulting and folding; changes in intensity or type of vegetation may indicate changes in soil and drainage which are themselves a function of a change in rock type. Prolonged or detailed analysis of the satellite imagery is unnecessary but a short inspection will enable the hydrogeologist/ geophysicist to gain general clues on gross changes in lithology and structure.

Aerial photographs are available from the Land Survey Department or they can be viewed at the Ministry of Agriculture, Land Use Section. Suitable photo scales are about 1:50,000 for regional work and 1:20,000 or 1:10,000 for more detailed work. Aerial photographs can be studied either individually or in stereo under a stereoscope. The photographs can be used like topographic maps for planning survey access routes. More recent photography (post 1970) may provide more up to date information on roads, tracks, houses, and land use than the 1:63,360 scale topographic maps. Aerial photographs can also be studied closely to detect spring lines, geological structure, rock outcrops and possible aquifer limits. Photo interpretation proceeds on the assumption that all the features in the photograph have some reason for being there; some may be man-made but most are in some way influenced by land form, drainage, soils, and climate. For example, a line of coconut trees separating two strips of padi many not be wholly artificial and may represent a strip of sandy soils which represent an ancient beach deposit or "permatang" feature common on the coast of Peninsular Malaysia. Similarly parallel lines of drainage may indicate subsurface lines of weakness or faults in the underlying rocks; such lines of weakness may have higher permeability than surrounding, unfractured bedrock.

Strong linear or fault features detected from air photo interpretation, may not provie to be useful in groundwater location. The work by JPT in Padang Terap, Kedah, using resistivity and drilling, indicated that linear-fault features were unrelated to permeability or to usable aquifer zones.

Air photo interpretation should therefore not be used excessively or as a substitute for drilling activity.

Topographic and Geological Maps

Topographic maps are often drawn by cartographers using aerial photographs and may therefore contain less detail than the photographs. However they can be used in a similar fasion, to assist in resistivity and drill rig access or for recognizing drainage, geomorphology or land use patterns. The 1:63,360 scale maps (1 inch to 1 mile) are very useful for large scale analysis of the drainage pattern alignment in study areas.

The 1:63,360 scale geological mapping of Malaysia is still in progress; many of the final maps and accompanying memoirs are not printed until many years after the field work has been completed. However unpublished maps and draft manuscripts of the memoirs can be seen at the Geological Survey Department, Ipoh. These documents constitute useful background material, but are somewhat biased to hard rock geology and economic geology. It is only Memoir 17 (Geology and Mineral Resources of Perlis, North Kedah and the Langkawi Islands by C.R. Jones) which contains substantial information on groundwater occurrence.

The geological maps largely show solid geology and are therefore of limited use in alluvial areas except to define the alluvial basin margins. Geologic maps are usually based upon the interpretation of the surface rock outcrops and, as new roadworks or mines are made, they may become inaccurate. Geologic maps are likewise open to reinterpretation on the basis of results, for instance, from water bore exploration and geophysical work.

Previous Reports

Relevant reports by Government departments, consultants and drilling companies should be read before planning field surveys and investigations. A large number of reports either specifically on geology and hydrogeology or containing references to these topics, exist. The principal author is the Geological Survey of Malaysia, Hydrogeological Section which has carried out exntesive drilling and testing for groundwater particularly in coastal and alluvial areas of Kelantan, Trengganu and Kuantan. Other reports (unpublished) are available from private drilling contractors or from consultants reports to government agencies. A thorough search of this literature is essential; already repeat drilling exploration has occurred in areas earlier adequately investigated.

Below is a list of the information that the hydrogeologist and geophysicist should try and abstract:

- description of geology, structure and potential aquifers
- details of the positions of previous geophysical surveys, and details of the results and problems
- the position, and construction details of existing boreholes; also copies of the geological and geophysical logs and details of aquifer properties.
- information on the chemistry of the groundwater in particular spatial variations in electirical conductivity.

Irrigation Studies

Although the main emphasis of the preliminary studies will be on geological or hydrogeological data, a complementary study of available literature on irrigation and agricultural practice and water shortage will be made, usually by a JPT irrigation engineer. Topics which ought to be reviewed for each water short area include:

- present water use and distribution
- legitimacy, location and duration of water shortage, alternative sources to groundwater
- constraints to crop production, including water shortage
- cropping pattern and water demand and distribution
- crop water requirements, soil permeability or moisture holding capacity.

This review should give some idea of required bore yields (stated in units such as l/s/ha) for particular crops and hence of the likely bore design and distribution.

1.2.3 Reconnaissance Field Studies

Assessment and review of existing data will be followed by field visits and liaison at state level, by both hydrogeologist geophysicist and irrigation engineers. Particular areas of interest will usually be as follows:

Hydrogeologist (+ Geophysicist)

- to check geologic maps, define aquifer boundary and examine geomorphology/topography
- to examine existing bores and dug wells; where possible to check electrical conductivity, water level and discharge of

these structures. Where possible a full well/bore census should be made, in order to allow plotting of water table and salinity maps. The hydrogeologist should initiate this census work at an appropriate sampling level; a census form was used by JPT during well census in the Bertam area of Seberang Prai.

- to check access in the areas of interest. The geophysicist will wish to check on the alignment and straightness of tracks as suitable profiling sites and also to check the position of overhead power cables and buried pipelines.
- to check on drilling rig access to proposed exploratory drilling sites and to evaluate whether such sites are where possible appropriate to an irrigation end use i.e. the sites have a suitable command area. Drilling sites should be positioned such that several criteria are met. The site should have a position consistent with the aims of hydrogeological study, and should assist in geophysical calibration. Its position should be in suitable command of the water short area or else near a supply canal. It should also be in a position where massive site preparation or tree felling is not needed to allow rig access, but where hard rig standing and a rig water supply is available.
- to liaise with state personnel on site positions, and required site preparation and access
- to evolve practical exploration programmes using drilling and appropriate geophysical methods.

Irrigation Engineer

The irrigation engineer,possibly in association with agricultural staff, would need to carry out the following:

- make field checks on soil type, existing cropping patterns and the application of irrigation water
- examine existing surface water system operation and identify any system constraints to irrigation supply
- identify pilot or exploratory bore sites which are in the best position for later irrigation use, if successful. Bore need to be sited so as to avoid long conveyance canals

and the need for canal lining. Final choice of bore site must be on the basis of both irrigation and hydrogeological opinions.

Where exploratory drilling work leads to groundwater development, then for each bore or groups of boreholes, a large number of irrigation and agro-economic factors (including labour, markets, land holding, bore and pump operation and maintenance, water distribution, cropping patter, drainage) will need close examination. At this exploratory phase, the main consideration is for the irrigation engineer to establish water shortage and, in liaison with his colleagues, to site bores on sound irrigation grounds.

2. RESISTIVITY METHODS

2.1 General

This chapter principally discusses the geophysical, resistivity methods used by the JPT Groundwater Section. Emphasis is placed on resistivity work as a component of groundwater studies, carried out in close collaboration with hydrogeologists and drilling staff. The use of the present resistivity equipment and the interpretation of results is discussed as this assists in groundwater exploration.

Geophysics is the applied science of trying to measure, by remote means, the physical properties of the earth; the following are the main branches of the science:

- seismic
- gravity
- magnetism
- resistivity + spontaneous potential
- induced potential
- radioactivity
- temperature

As an academic discipline, geophysics can be pursued for its own ends but usually it is an applied tool used to understand or solve a problem in another practical discipline. Sometimes the geophysical property measured has importance in its own right; airborne or surface scintillation surveys, for example, measure radioactivity in the rocks and can therefore be used to isolate areas with radioactive mineral potential. However, in most cases, the geophysical property of rocks has only an indirect relationship to the practical discipline. Seismic surveys attempt to measure the ability of the underlying rock to transmit shock waves but this property alone has little practical relevance; rather the response of the rock to shock waves is used to interpret rock lithology which can be further interpreted in terms of possible groundwater content or oil bearing properties. Geophysics is often expected to provide all the answers and obviate the necessity of drilling or direct sampling. Later, it will be seen that such expectations are illfounded.

A geophysicist, asked to provide information about the subsurface, first has to define the problem and must then choose the most appropriate geophysical tool for the job. No one geophysical method will suit all subsurface problems and the decision on which method and equipment to choose is not always straightforward. Often several methods are appropriate and in some cases both or all are used. The final decision depends on other factors such as time, money, access and availability of equipment. The decision by the groundwater section to use the resistivity method for hydrogeological investigations is illustrative. In many geological settings, seismic methods would give better answers yet the seismic method would be of no assistance in the study of groundwater salinity in the coastal region, an important consideration in Malaysia. The seismic method requires explosives to make deep investigations and the detonation of explosive charges in and around villages, livestock, and on the surface of roads and bunds, is a very anti-social activity requiring the permission of many people; deep seismic work although indicated, may therefore not be chosen because of its limited acceptability.

Within the groundwater section, the geophysicist is widely involved in groundwater exploration. Figure 1.1 indicates the place of resistivity studies as a part of the hydrogeological exploration programme. The upper part of the diagram shows the background data that the geophysicist and the hydrogeologist will review before the start of the resistivity work whilst the central and lower parts of the diagram show, ideally, how the geophysics team will carry out an exploratory or reconnaissance survey and then formulate plans for exploration borehole sites. On the basis of drilling results, the geophysics team will often return to the area to check the consistency of geophysical results or to assist in defining the limits of the best area for production bore drilling. The main point behind this method of approach is that the level of interpretation by the geophysicist depends upon the amount and quality of information he has on the geology and hydrogeology. The resistivity equipment can only measure the resistivity of the subsurface and if the geophysicist has no background data on the subsurface, he can only report the resistivity findings and guess what these mean in terms of rock layers and types. If good background information on the surface and subsurface geology is available, then this guess will be much more informed or reliable. If the geophysicist has information from boreholes and pumping tests then his guess can include an assessment of the hydrogeological meaning of his resistivity readings.

It can be seen that the hydrogeologists and geophysicists must collaborate; each discipline should give feed-back to the other, so that analysis and decisions can continually be upgraded.

2.2 Simple Resistivity Theory

2.2.1 Resistivity and Conductivity

Resistivity is not synonymous with the commonly understood term "electrical resistance". Electrical resistance (R) is defined by Ohm's law as:

$$R = \frac{\Delta V}{I} \qquad (1)$$

where ΔV is the difference in voltage between the two ends of a resistance, and I is the current flowing through the resistance. The resistance of a wire is proportional to the length of the wire (L) and inversely proportional to the cross-sectional area (A).

$$R \propto \frac{L}{A}$$

or

$$R = \rho \times \frac{L}{A} \qquad (2)$$

The symbol ρ is the constant of proportionality, known as the electrical resistivity or the electrical specific conductance of the material. If equations (1) and (2) are combined to provide a definition of resistivity, we get:

$$\rho = \frac{A}{L} \times \frac{\Delta V}{I} \qquad (3)$$

Resistivity is defined for practical purposes as being numerically equal to the electrical resistance of a block of the material of unit dimensions. The most commonly used dimensions are metres, and hence resistivity is usually expressed in ohms times metres or ohm-metres.

In order to understand resistance and resistivity it is necessary to understand conductance, the inverse of resistance and conductivity, the inverse of resistivity. Conductance is the property of the material to transmit either direct or alternating electrical current. Electric current is not transmitted in the same way by all materials. Metal wires, such as are used in the resistivity instrument cables, contain free electrons and are good conductors; a good conductor is usually regarded as having a resistance less than 1 x 10^{-5} ohms. The migration or oscillation of these free electrons under a potential imposed by a battery or an alternating current source, is called electronic (or ohmic) conduction. Most rocks do not have free electrons and they are not usually regarded as good conductors; exceptions to this are certain metallic ore deposits and graphite. Most rocks are either termed semi-conductors or insulators.

Two other types of conduction are levant to rocks, electrolytic and dielectric conduction. Electrolytic conduction takes place in fluids containing ions, which are molecules which have an excess or deficiency of electrons. It takes place slowly by comparison with electronic conduction. The matrix, or groundmass of a rock is not, of course, a solution of ions, and rocks that are completely dry have a very high resistance. Rocks that contain water in their pores, cracks and voids have a much lower resistance, because electric current can be carried by the free ions of the minerals dissolved in the water. The more ions in the water (i.e. the higher the salinity), the better the conductance and the lower the resistance. The resistivity of a rock therefore depends upon the salinity of the groundwater. It also depends upon the volume of the pores in relation to the volume of the rock, termed porosity and the arrangement of the pores. If all the pores are interconnected, the current flow will be un-interrupted but if the pores are isolated and separated by dry solid rock, current flow is impeded. An empirical formula was derived by Archie in 1942 to describe the relationship between resistivity, porosity, degree of saturation, and groundwater salinity. This is given below:

$$\rho e = a \phi^{-m} X S^{-n} \rho w \qquad (3)$$

Where ρe = resistivity of the rock
ρw = " of the water
ϕ = fractional pore volume (porosity)
s = fraction of pores containing water
n = 2
a = constant with a value less than 2.5 and greater than 0.5
m = constant with a value less than 2.5 and greater than 1.3

This equation is useful when estimating either the resistivity characteristics or the porosity of a rock. For example, if a borehole intersects a particular formation, the porosity can be estimated for the rock samples and the groundwater resistivity (the inverse of the electrical conductivity) can be measured from water samples during pumping tests. By making reasonable assumptions for s, a, m and n in equation (3), it is possible to calculate the resistivity of the rock. This calculated value can be used to check the depth-layer calculations from resistivity soundings (Section 2.8). Conversely, the rock porosity can be calculated if resistivity data is available. However, a word of warning to those who attempt to use resistivity data and Archie's formula to gauge the hydraulic properties of a rock and hence bore yield and drawdown; porosity should not be confused with permeability. A clay has a high porosity, but usually a very low permeability.

Dielectric conduction is the displacement of atomic (rather than molecular) electrons in relation to their atomic nucleii under the influence of a varying electric field, for example, alternating current. Dielectric conduction takes place in poor conductors and insulators, such as absolutely pure water containing no ions or dry rock. The slight relative separation of the negative electrons and the positive nucleii is called dielectric polarization. Dielectric conduction only takes place under relatively high frequency alternating current and resistivity equipment which uses an alternating current source employs low frequency current in order to avoid measuring or being influenced by dielectric conduction. Field resistivity methods aim to determine the inverse of the electronic and electrolytic conductivity under a direct current. The reason for this is that these properties are more directly related to such subsurface characteristics as porosity, saturation, salinity and mineral content. Low frequency alternating current is in effect the same as direct current.

One other characteristic of conduction is that it is influenced by temperature. Electronic conduction increases with decreasing temperature, and the opposite is true for both electrolytic and dielectric conduction. As an illustration; metallic electronic conductors have zero resistance at absolute zero ($0^{o}K$) temperature, and hot groundwater has a lower resistivity than cool groundwater of the same salinity.

Resistivity of subsurface materials therefore depends upon the following factors:

- the porosity of the material
- the degree of saturation of the bore spaces
- the salinity of the water in the pore spaces
- the degree of interconnection between the pore spaces
- the mineral composition of the material
- temperature

2.2.2 Resistivity Characteristics of Different Rock Types

Each rock type does not have an unique and easily definable resistance or resistivity; sandstones for example are of many types with varying resistivities. A further misconception is that if the resistance of a buried layer can be measured then that rock type can be precisely defined which is certainly not the case. The problem partly stems from the imprecise terminology used by geologists; the term sandstone is the title for a group of clastic granular rocks of grain size between 0.06 and 2 mm. There are however several different types of sandstones and considerable variation in the precision of description of sandstone types. A quartz clast sandstone can be described by the grain size of the clasts, by silt-clay fraction in the matrix, by its hardness, porosity or weathering characteristic or by the entry of non-quartz clastic fragments. Other sandstones are composed of calcium carbonate grains and may be termed calcarenytes. Each different type of sandstone will have a different resistance and also water bearing properties and therefore the major rock type termed sandstone will have a range of resistances or resistivities. This range increases if one considers the values for dry sandstones and sandstones saturated with fresh or saline water.

Table 2.1 is data compiled from various standard references; it shows the expected range of resistivites for different rocks types.

Table 2.1 Electrical Resistivities of Rocks

Rock Type	Resistivity (Ohm-m)
Clay	1-100
Marl	3- 70
Sand (unconsolidated)	10-800
Sandstone	35-4,000
Siltstone	10-800
Limestone	120-400
Marble	$100-1 \times 10^6$
Granite	$300-1 \times 10^6$
Basalt lava	$10-1.3 \times 10^7$

2.2.3 Theory of Resistivity Method

Resistivity measurements are made in a lab by putting current electrodes at either end of a core sample. Current is passed through the sample and the voltage difference is measured between two potential electrodes encircling the sample a fixed distance apart (Figure 2.1). Laboratory measurements are relatively straightforward because the current flows through a defined cross section for a fixed distance.

Resistivity measurements in the field use an electrode arrangement of similar form but laid on the ground surface; the cross sectional area cannot be defined however and the subsurface is always heterogenous and anisotropic. The problem is best understood by studying the current flow and potentials in an infinite homogenous isotropic material. Figure 2.2a shows the current flow and equipotential lines around a single electrode or point source on the surface of a homogenous medium. As can be seen, the equipotential lines are hemispherical and the current always flows at right angles to them. Figure 2.2(b) shows how equipotential and current flow lines are distorted when two current electrodes are placed on the surface. When the distance between the two current electrodes is finite or relatively small, the potential at a nearby surface point will be influenced by both current electrodes.

If a potential electrode is put into the ground surface at a distance from either current electrode, the potential will be:

$$\text{potential } V\ (r) \quad = \quad \frac{I\rho}{2\pi\, r} \qquad (4)$$

where I = current, and $= \rho$ resistivity

If two potential electrodes (called P_1 and P_2) are placed between or near the two current electrodes (called C_1 and C_2) the potentials at P_1 and P_2 will be different; and can be simply expressed as

$$\Delta V = (VP_1\ C_1 - VP_1\ C_2) - (VP_2\ C_1 - VP_2\ C_1) \qquad (5)$$

where, for example $VP_1\ C_1$ is the potential at P_1 due to the current entering at C_1. The potential at each potential electrode relates directly to the distance between the current and potential electrodes.

FIGURE 2.1 SCHEMATIC DIAGRAM OF MEASUREMENT OF RESISTIVITY OF CORE SAMPLE.

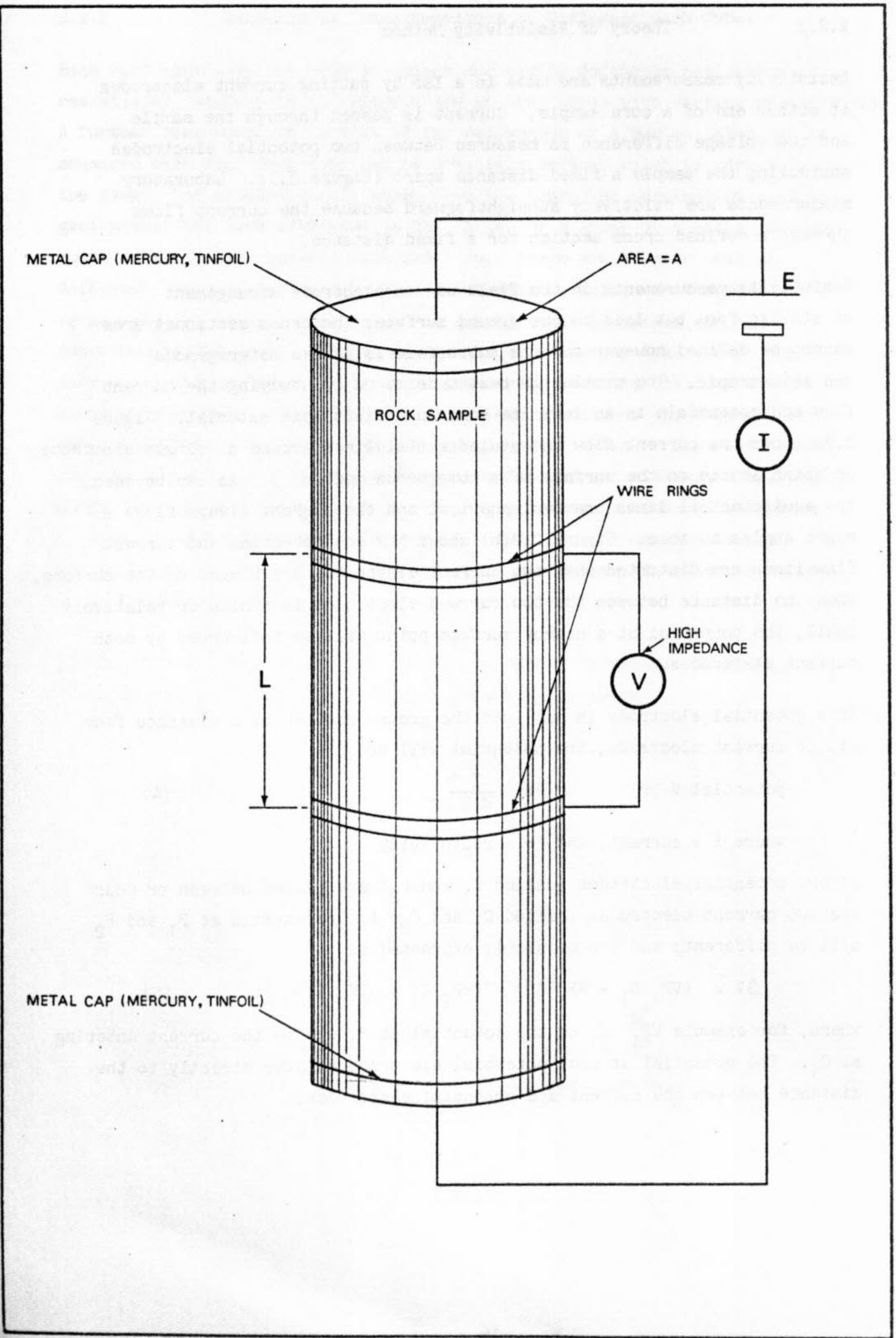
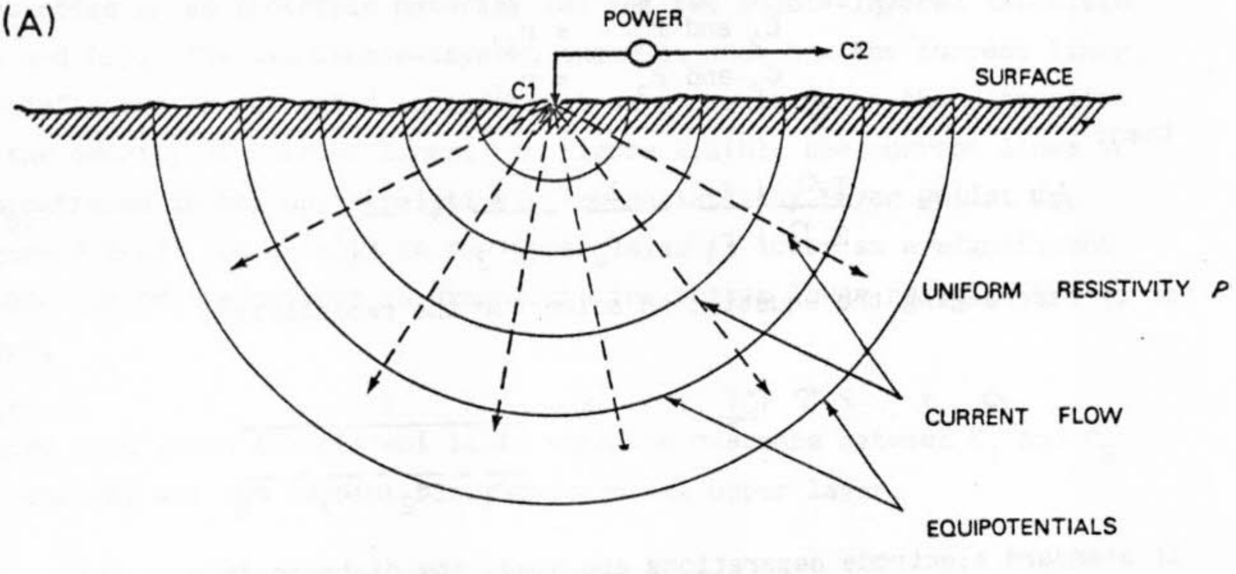
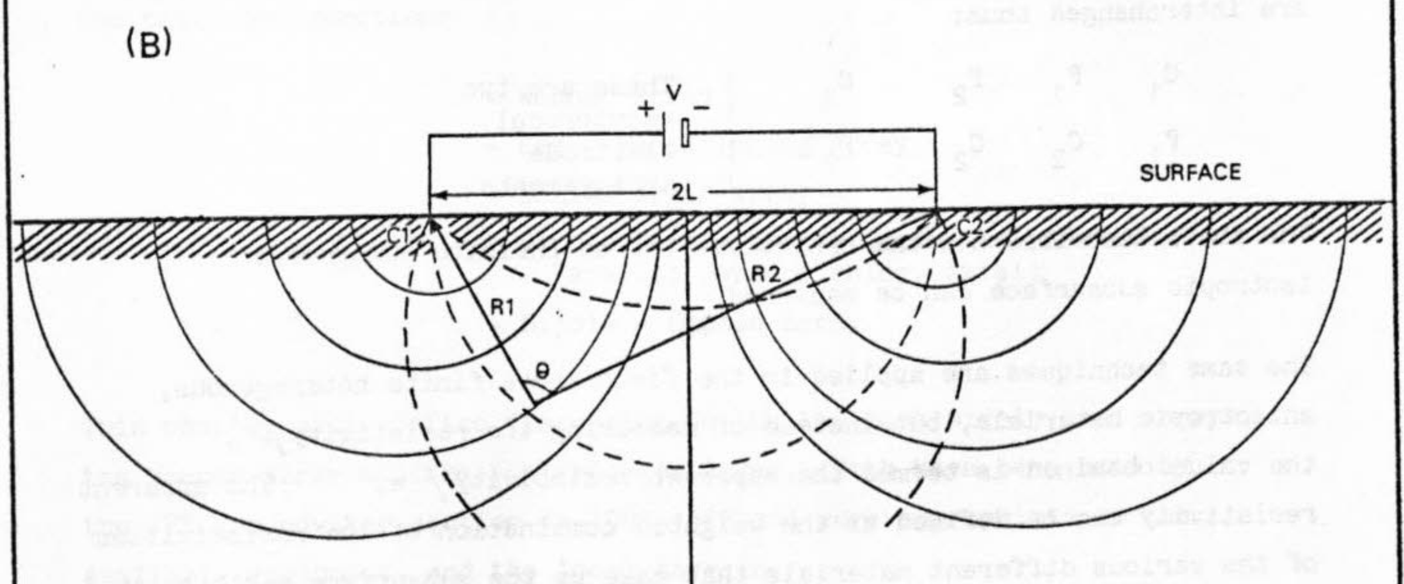


FIGURE 2.2 CURRENT FLOW IN A HOMOGENEOUS MEDIUM



POINT SOURCE OF CURRENT AT THE SURFACE OF A HOMOGENEOUS MEDIUM



DISTORTION OF EQUIPOTENTIALS AND CURRENT FLOWLINES FOR TWO POINT SOURCES OF CURRENT IN A HOMOGENEOUS MEDIUM (AFTER DOBRIN 1960)

If the distance between C_1 and $P_1 = r_1$,
C_1 and $P_2 = r_2$,
C_2 and $P_1 = r_3$,
C_2 and $P_2 = r_4$,

then:

$$\Delta V = \frac{I\rho}{2\pi}\left(\frac{1}{r_1} - \frac{1}{r_2} - \frac{1}{r_3} + \frac{1}{r_4}\right) \qquad (6)$$

or by rearranging the equation to solve for the resistivity:

$$\rho = 2\pi\left(\frac{\Delta V}{I}\right) \times \frac{1}{\frac{1}{r_1} - \frac{1}{r_2} - \frac{1}{r_3} - \frac{1}{r_4}} \qquad (7)$$

If standard electrode separations are used, the distance between the electrodes can be simplified to a constant K for each set of separations; therefore:

$$\rho = K \frac{\Delta V}{I} \qquad (8)$$

It is relevant to note that according to Helmholtz's reciprocity theorem, the value of ρ will be unaltered if the current and potential electrodes are interchanged thus:

C_1	P_1	P_2	C_2	These are two recriprocal electrode arrangements.
P_1	C_2	C_2	P_2	

The above describes how the resistivity of an infinite homogenous isotropic subsurface can be measured.

The same techniques are applied in the field above finite heterogenous, anisotropic materials, but instead of measuring the resistivity ρ, the value obtained is termed the apparent resistivity ρa. The apparent resistivity can be defined as the weighted combination of the resistivities of the various different materials that make up the subsurface earth below the point of measurement. The apparent resistivity is not the average resistivity of the rocks between and below the electrodes, because the current flow lines and the equipotentials are distorted by the heterogenous characteristics of the subsurface materials.

Figure 2.3 schematically shows the current flow lines between two current electrodes in an isotropic material (a) and two double-layered materials (b) and (c). The two double-layered examples show how the current lines are deflected or refracted in different ways resulting in the alteration of the density of current lines. In Figure 2.3(b), the current lines are concentrated in the upper relatively low resistivity layer whilst in Figure 2.3(c), the density in the upper layer is lower as a significant proportion of the current is drawn into the bottom lower resistivity layer.

Figure 2.3d shows the current lines when the distance between C_1 and C_2 is reduced; all the current flows through the upper layer.

2.2.4 Electrode Configurations

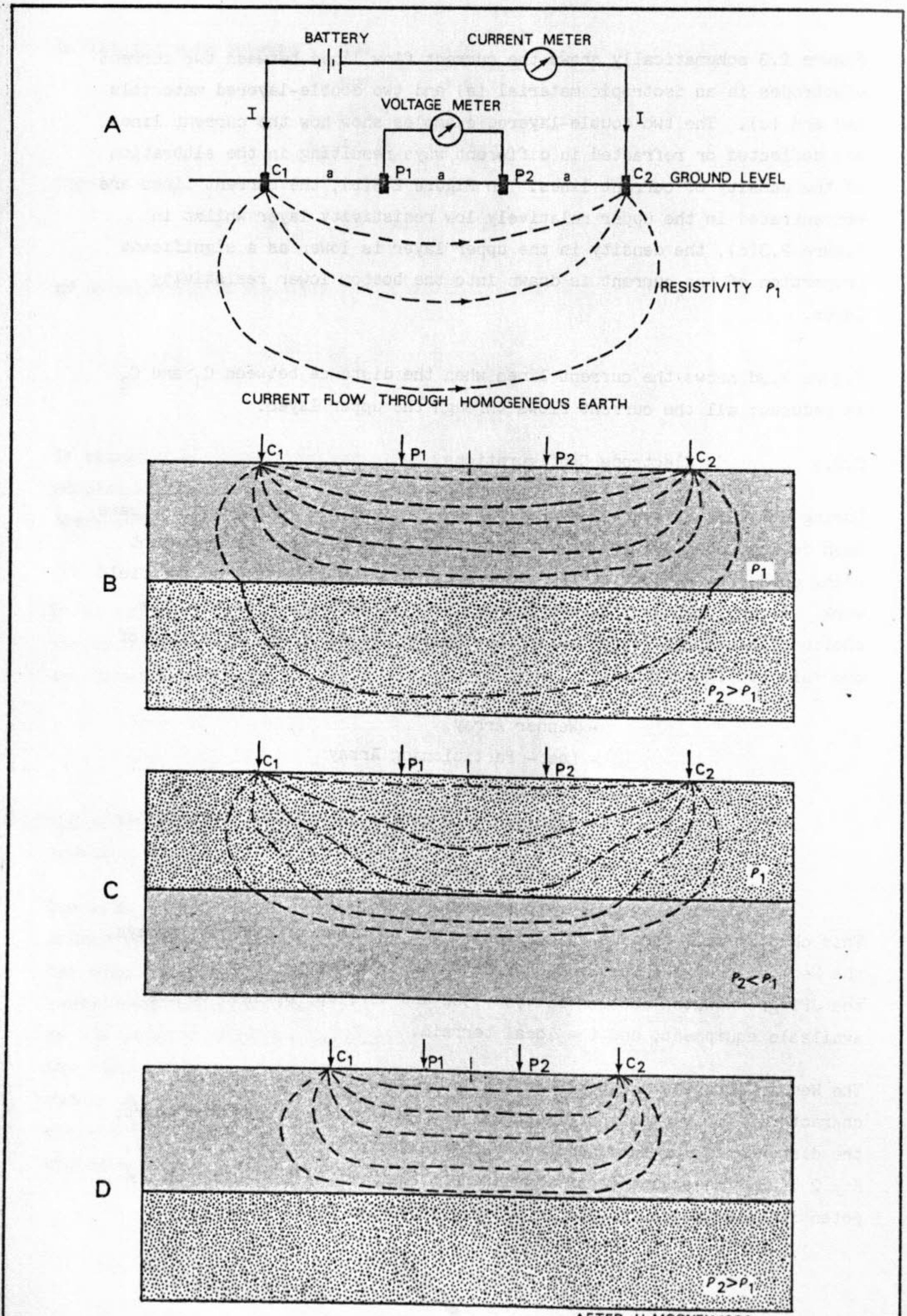
During the last 70 years, several different electrode configurations have been devised, the suitability of which has depended upon the equipment being used, the geology of the study area, and the logistics of the field work. As equipment has become increasingly refined and flexible, the choice of electrode configuration has become wider. Below is a list of the main configurations:

- Wenner Array
- Lee - Partitioning Array
- Schlumberger Array
- Dipole-Dipole arrays (Azimuthal, Radial, Parallel Perpendicular, Axial)
- Bipole - Dipole array

This chapter will confine discussion to the two most widely used arrays, the Wenner Array and the Schlumberger Array. Both have been used by the JPT groundwater section in 1981-1982 and were appropriate to the available equipment, and the local terrain.

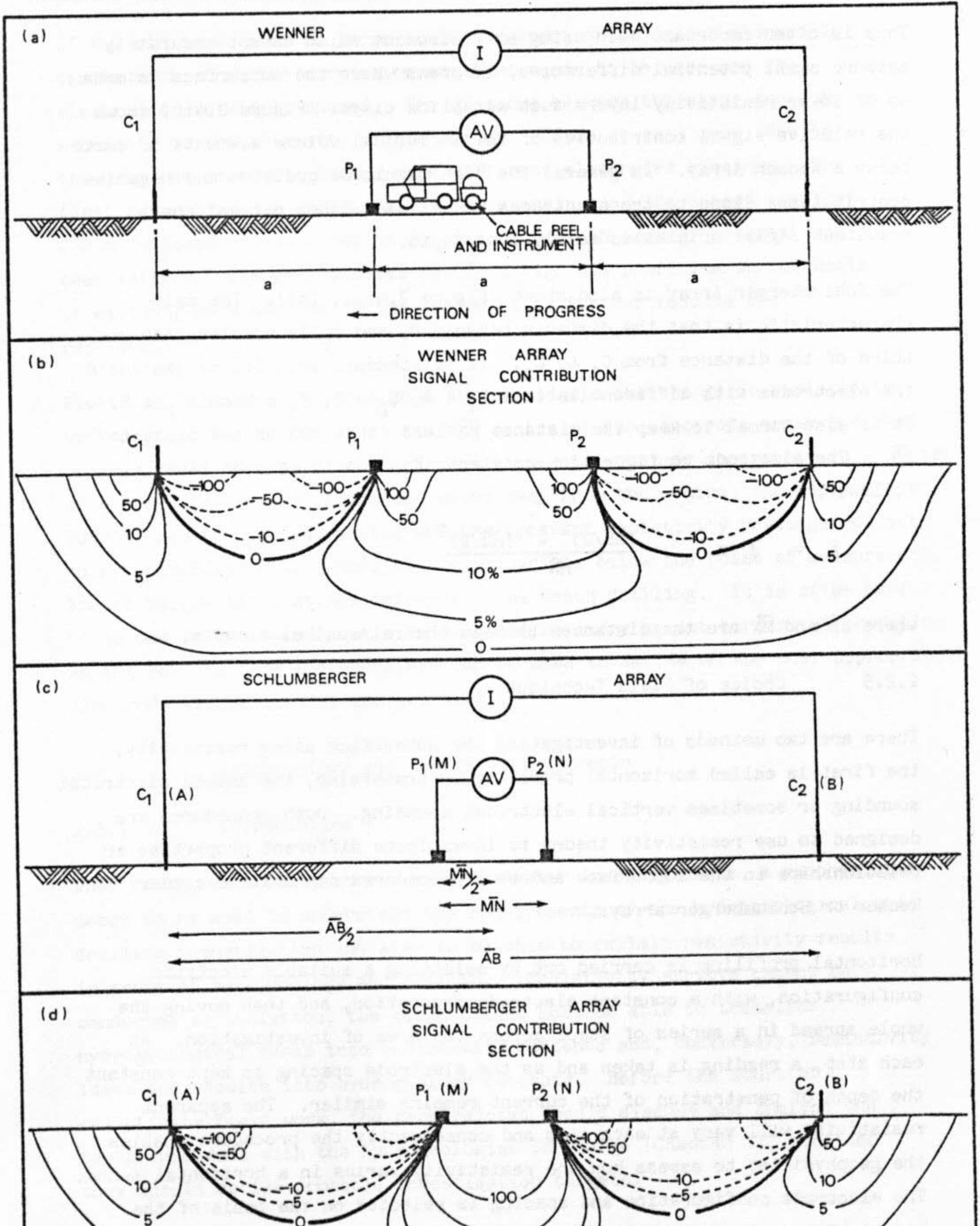
The Wenner Array is shown schematically (Figure 2.4(a)). The main characteristic is that the electrodes are kept an equal distance apart; the distance is commonly denoted by the letter a and hence in eqn (8), K = 2 IIa. The advantage of the array is that the wide spacing of the potential electrodes results in a large potential difference.

FIGURE 2.3 CURRENT FLOW IN ISOTROPIC AND ANISOTROPIC MEDIUM



AFTER H. MOONEY 1980

FIGURE 2.4 ELECTRODE ARRAYS AND SIGNAL CONTRIBUTION CONTOURS



NOTE: FIGURES 2.4b, 2.4d, THE CONTOURS SHOW THE RELATIVE CONTRIBUTION MADE BY THE INDIVIDUAL VOLUME ELEMENTS OF THE HOMOGENEOUS EARTH TO THE TOTAL POTENTIAL DIFFERENCE MEASURED BETWEEN P_1 TO P_2. BROKEN LINES ARE NEGATIVE CONTOURS.

This is often important when using an instrument which cannot accurately measure small potential differences, in areas where the subsurface is made up of lower resistivity layers such as saline clays. Figure 2.4(b) shows the relative signal contribution of the individual volume elements of earth below a Wenner Array. In general the high magnitude positive and negative contributions close to the electrodes cancel each other out and the resultant signal originates mainly from depth.

The Schlumberger Array is also shown (Figure 2.4(c), (d)). The main characteristic is that the distance between P_1 and P_2 is smaller than one third of the distance from C_1 to C_2. It is standard practice to designate the electrodes with different letters $C_1 = A$, $C_2 = B$, $P_1 = M$ and $P_2 = N$. It is also normal to keep the distance $\overline{MN}$ less than 20% of the distance $\overline{AB}$. The electrode configuration constant, K, used in eqn (8) is as follows:

$$K = \pi \times \frac{(\overline{AB}/2)^2 - (\overline{MN}/2)^2}{\overline{MN}}$$

where $\overline{AB}$ and $\overline{MN}$ are the distances between the relevant electrodes.

2.2.5 Choice of Field Technique

There are two methods of investigating the subsurface using resistivity; the first is called horizontal profiling or traversing, the second electrical sounding or sometimes vertical electrical sounding. Both procedures are designed to use resistivity theory to investigate different properties or relationships in the subsurface and both procedures can utilize either Wenner or Schlumberger arrays.

Horizontal profiling is carried out by selecting a suitable electrode configuration, with a constant electrode separation, and then moving the whole spread in a series of steps across the area of investigation. At each step, a reading is taken and as the electrode spacing is kept constant the depth of penetration of the current remains similar. The apparent resistivity will vary at each step and consequently the procedure enables the geophysicist to assess how the resistivity varies in a horizontal sense. The electrode configuration and spacing is selected on the basis of the target of investigation. If the aim is to monitor variations in the thickness of shallow overburden or alluvium, the current penetration needs to be shallow and hence the electrodes spacing is short. Conversely, if the aim is to pick up large scale structure or lithology changes in the bedrock, then the current penetration needs to be deep and the electrodes are widely spaced. The distance between the steps can vary according to the intensity of the survey. Often it is a good idea to traverse the area with two or more different electrodes spacings. The first reconnaissance profile might for example use wide spaced steps of 200 m followed by re-survey in 20 m steps once an area of interest had been defined. The geophysicist has to design his profiling on the basis of existing data and modify the procedure as further results are received.

Electrical soundings are designed to investigate the change in resistivity with depth. The method is to fix the centre of the electrode array and then progressively move out the electrodes either together or separately in a series of steps. As the electrodes are moved away from the centre, the depth of current penetration increases, and the apparent resistivity readings reflect an ever increasing thickness of the subsurface below the point of measurement. The technique is sometimes referred to as depth drilling. It is often useful to do one or two soundings in an area before carrying out horizontal profiling as the results from the soundings can be used to decide on the most appropriate electrode separation for the profiling.

2.3 Hydrogeology and Resistivity Concepts

2.3.1 Introduction

When using resistivity to assist groundwater investigations, a geophysicist needs to be able to understand the requirements of the hydrogeological and drilling investigation and also to be able to explain resistivity results in terms of hydrogeology and drilling. Resistivity surveys cannot be conducted in isolation; the geophysicist must be able to translate hydrogeological ideas into resistivity concepts and, conversely, resistivity ideas and results into hydrogeology concepts. Before the start of resistivity field work, the geophysicist should discuss and analyse the background data with the hydrogeologist in charge (Chapter 1); together they should decide probable investigation targets.

A series of hydrogeological settings common in Malaysia are discussed below. Each description and diagram is accompanied by a discussion of of the implications in resistivity terms.

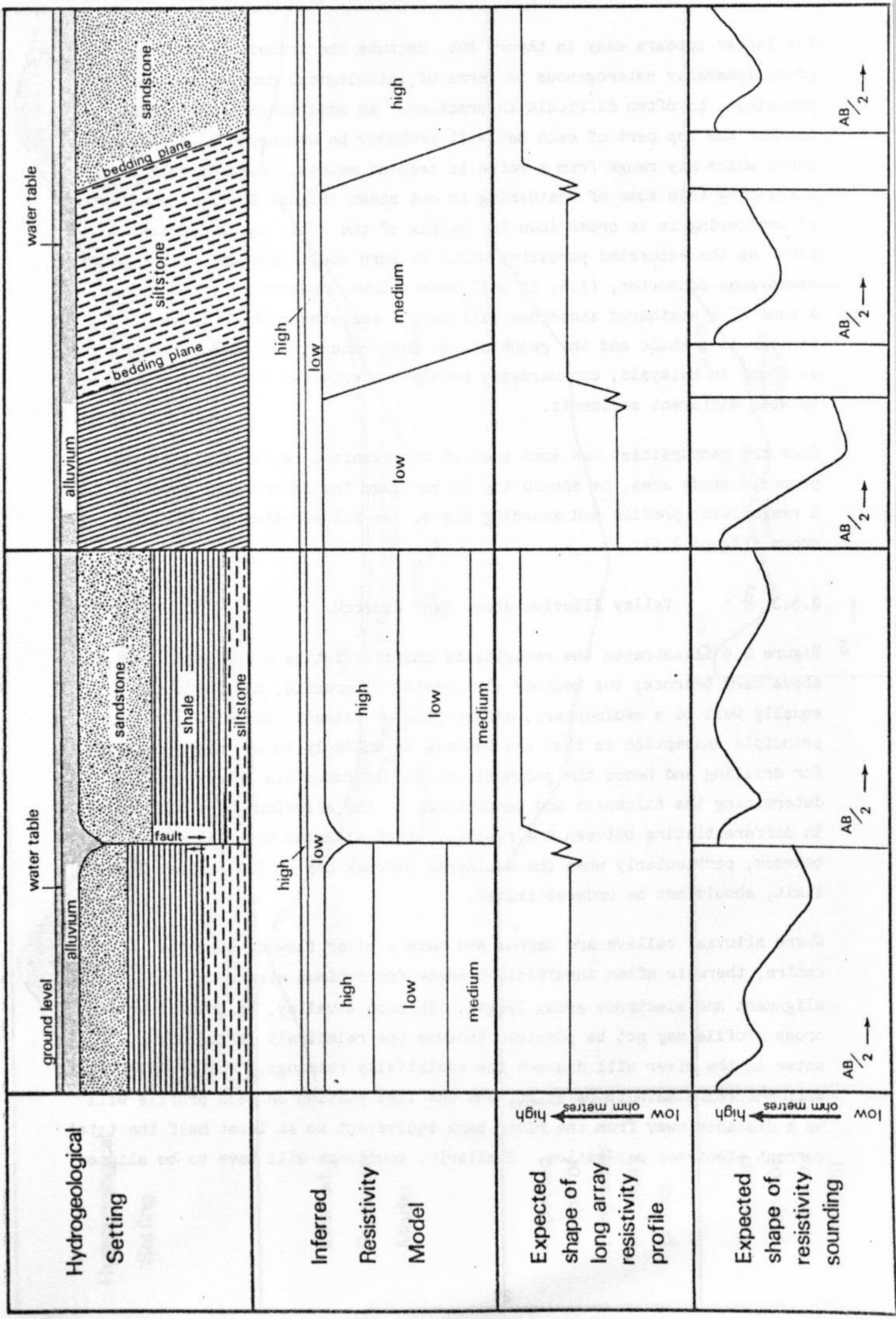
2.3.2 Sedimentary Bedrock below thin overburden

In this setting, the hydrogeologist will probably recommend that the shallow overburden or alluvium is dismissed as a potential aquifer, and that the aquifer potential of the bedrock should be assessed. The hydrogeologist will probably recommend that the resistivity team tries to identify variations in bedrock resistivity that can be correlated with variations in both rock type and aquifer properties.

Figure 2.5 illustrates two geological settings where hardrock sediments underly a thin cover of alluvium/colluvium/weathered rock. The underlying rocks are shown as relatively thick layers with either a horizontal or steep dip. If the layers or beds are very thin (as found in the Semanggol Formation below Padang Terap, Kedah) it is unlikely that resistivity soundings or profiles can distinguish between them.Instead the resistivity survey could only aim to pick out changes in average or bulk resistivity between one area and another. Figure 2.5 illustrates how the geophysicist starts to translate a purely geological and hydrogeological idea into its implications for a resistivity survey. He would expect soundings, in areas where the beds are near horizontal, to reveal changes in lithology, thickness of lithology, the depth to the water table, and the depth of overburden. He would expect his profiles to perhaps pick up sudden discontinuities such as faults in the bedrock. The identification of these zones of discontinuity may be important because the shattering around the fault may enhance rock permeability, Conversely, the shattering may have opened up the fault zone to deep weathering which, depending upon the nature of the bedrock, may result in the fault being clogged with residual clayey minerals of low permeability. The geophysicist should aim at identifying the fault zones; exploratory boreholes will define whether or not the faults are zones of higher permeability.

If the sedimentary rocks are thought to be steeply dipping, the geophysicist should seek to distinguish between different lithologies and also try to identify the different lithologies by means of soundings, carried out with the electrodes lined parallel to the geological strike.

FIGURE 2.5 HARD ROCK SEDIMENTS BELOW THIN ALLUVIUM



The latter appears easy in theory but, because the individual beds are often laterally heterogenous in terms of lithological composition and porosity , is often difficult in practice. An additional complication is that the top part of each bed will probably be weathered to a variable depth which may range from a metre to tens of metres. For the sake of simplicity this zone of weathering is not shown (Figure 2.5). The effect of weathering is to break down the matrix of the rock, which process enhances the saturated porosity, which in turn would make the rock a better electrical conductor, (i.e. it will have a lower resistance). Therefore, a zone of a weathered sandstone will have a resistance which is very similar to a shale and the geophysicist must recognize that deep weathering, as found in Malaysia, can markedly reduce the expected resistivity contrast between different sediments.

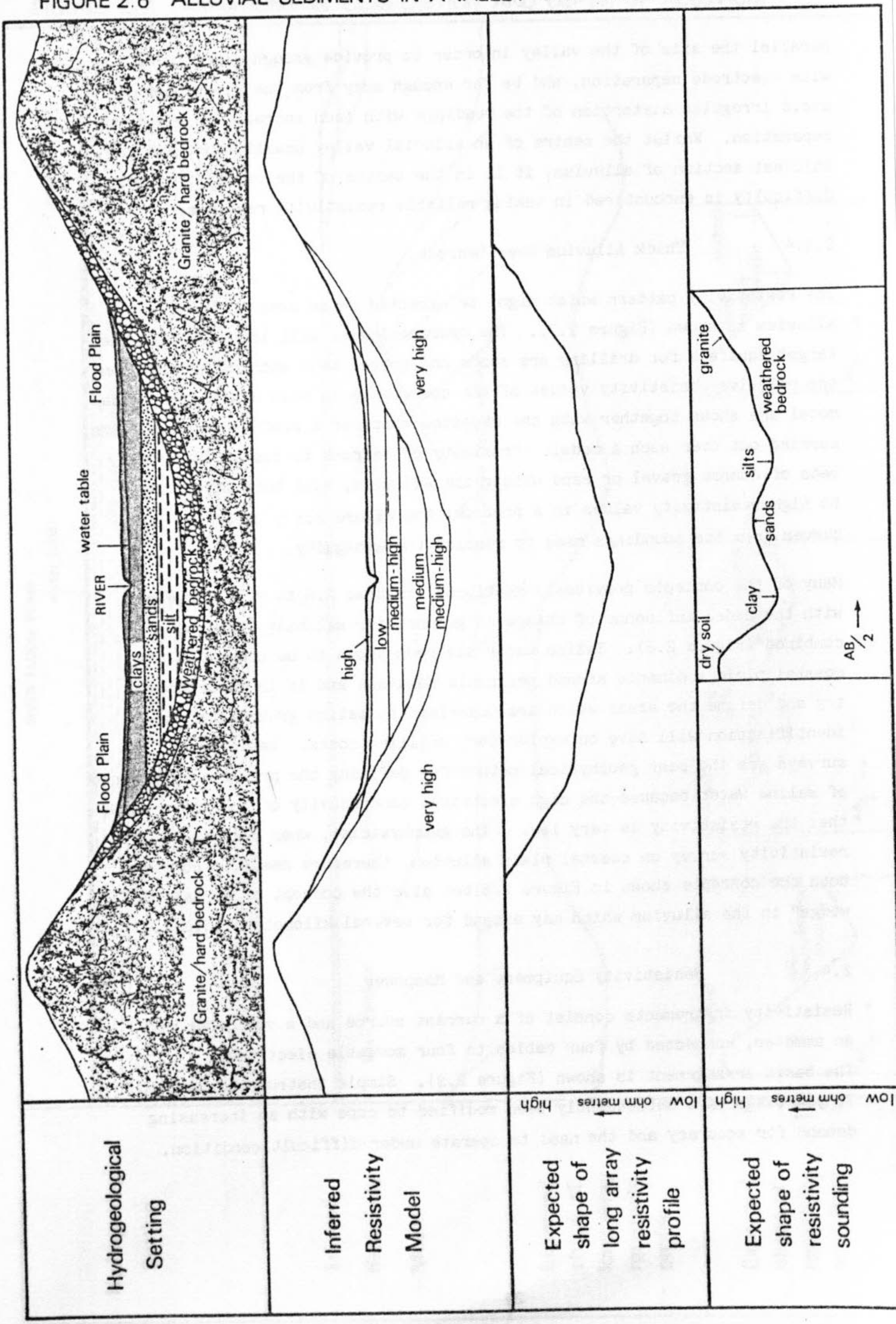
Once the geophysicist has some idea of the expected resistivities in the proposed study area, he should try to envisage the theoretical shape of a resistivity profile and sounding there. An illustration of this is shown (Figure 2.5).

2.3.3 Valley Alluvium Above Hard Bedrock

Figure 2.6 illustrates the resistivity characteristics of valley alluvium above hard bedrock; the bedrock illustrated is granite, but it could equally well be a sedimentary, metamorphic or volcanic bedrock. The principle assumption is that the bedrock is unlikely to be the main target for drilling and hence the geophysicist should focus his intentions on determining the thickness and composition of the alluvium. The difficulty in differentiating between the resistivity of alluvium and weathered bedrock, particularly when the weathered bedrock has an ill defined lower limit, should not be underestimated.

Where alluvial valleys are narrow and have a river flowing down the centre, there is often insufficient space for optimum electrode alignment and electrode array length. In such a valley, an uninterrupted cross profile may not be possible because the relatively large body of water in the river will distort the resistivity readings. The profile will therefore have to be split, and the last station on each profile will be a distance away from the river bank equivalent to at least half the total current electrode separation. Similarly, soundings will have to be aligned

FIGURE 2.6 ALLUVIAL SEDIMENTS IN A VALLEY CUT INTO HARD BEDROCK.



parallel the axis of the valley in order to provide enough space for a wide electrode separation, and be far enough away from the river bank to avoid irregular distortion of the readings with each increase in electrode separation. Whilst the centre of an alluvial valley usually contains the thickest section of alluvium, it is in the centre of the valley that most difficulty is encountered in taking reliable resistivity readings.

2.3.4 Thick Alluvium Over Bedrock

The resistivity pattern which might be expected in an area of thick alluvium is shown (Figure 2.7). The hydrogeologist will indicate that the target aquifers for drilling are sands and gravel beds within the alluvium; the relative resistivity values of the components in such a hydrogeological model are shown together with the expected shape of a profile and soundings carried out over such a model. Proximity of bedrock to the surface, and beds of coarse gravel or sand within the alluvium, will both give rise to high resistivity values in a profile; the Figure 2.8 illustrates the curves from the soundings made to resolve the ambiguity.

Many of the concepts previously mentioned (Figures 2.6 to 2.7), together with the added influence of change in groundwater salinity, have been combined (Figure 2.8). Saline water has been shown to be common in the coastal plain sediments around peninsula Malaysia and it is important to try and define the areas which are underlain by saline groundwater as identification will save on exploratory drilling costs. Resistivity surveys are the best geophysical method for defining the presence or absence of saline water because the high electrical conductivity of the water means that the resistivity is very low. The geophysicist, when planning a resistivity survey on coastal plain alluvium, therefore needs to consider both the concepts shown in Figure 2.8 but also the concept of a "saline wedge" in the alluvium which may extend for several kilometres inland.

2.4 Resistivity Equipment and Manpower

Resistivity instruments consist of a current source and a voltmeter and an ammeter, connected by four cables to four moveable electrodes. The basic arrangement is shown (Figure 2.3). Simple instruments used 70 years ago have subsequently been modified to cope with an increasing demand for accuracy and the need to operate under difficult condition.

FIGURE 2.7 VARIATIONS IN COMPOSITION AND DEPTH OF ALLUVIUM.

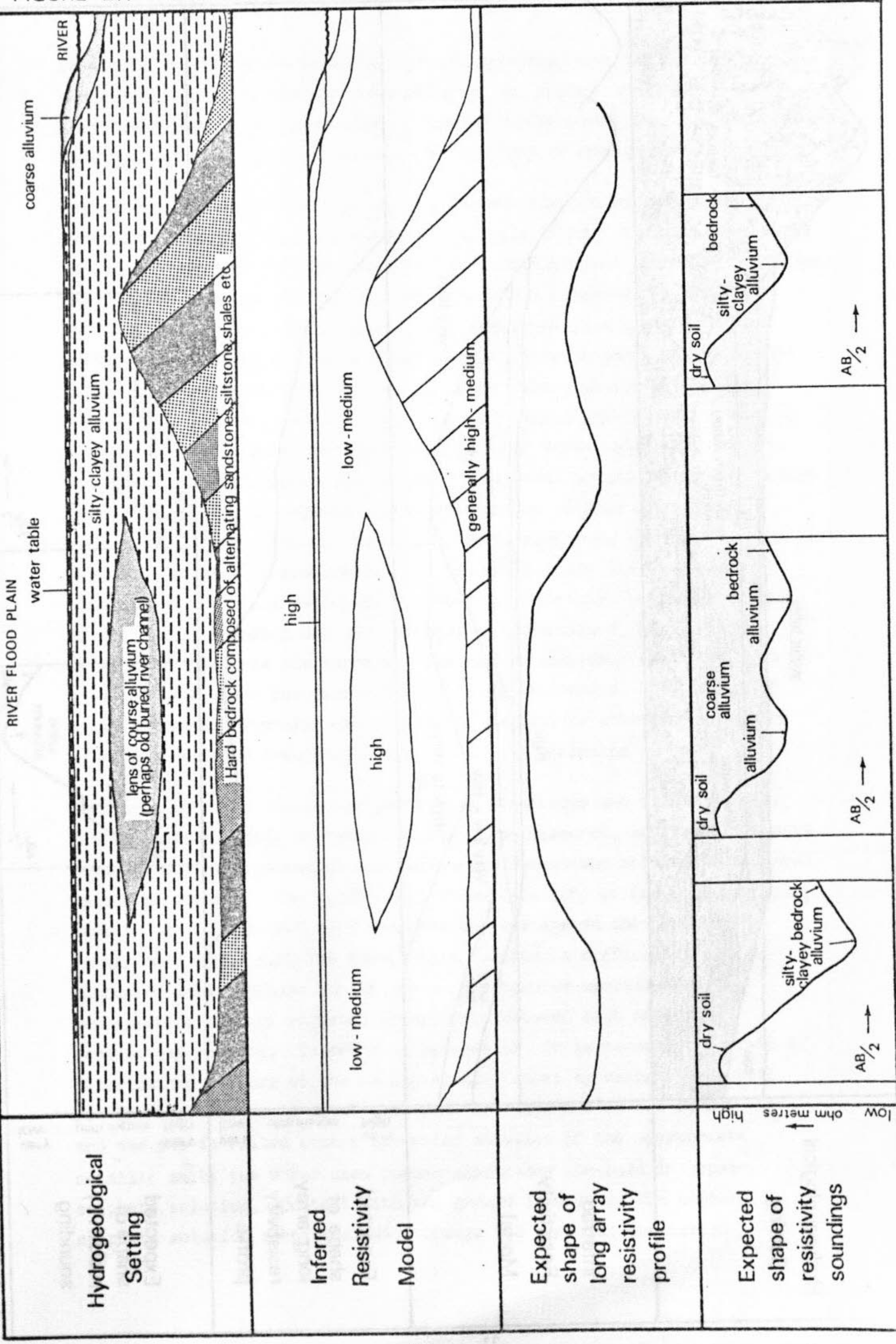
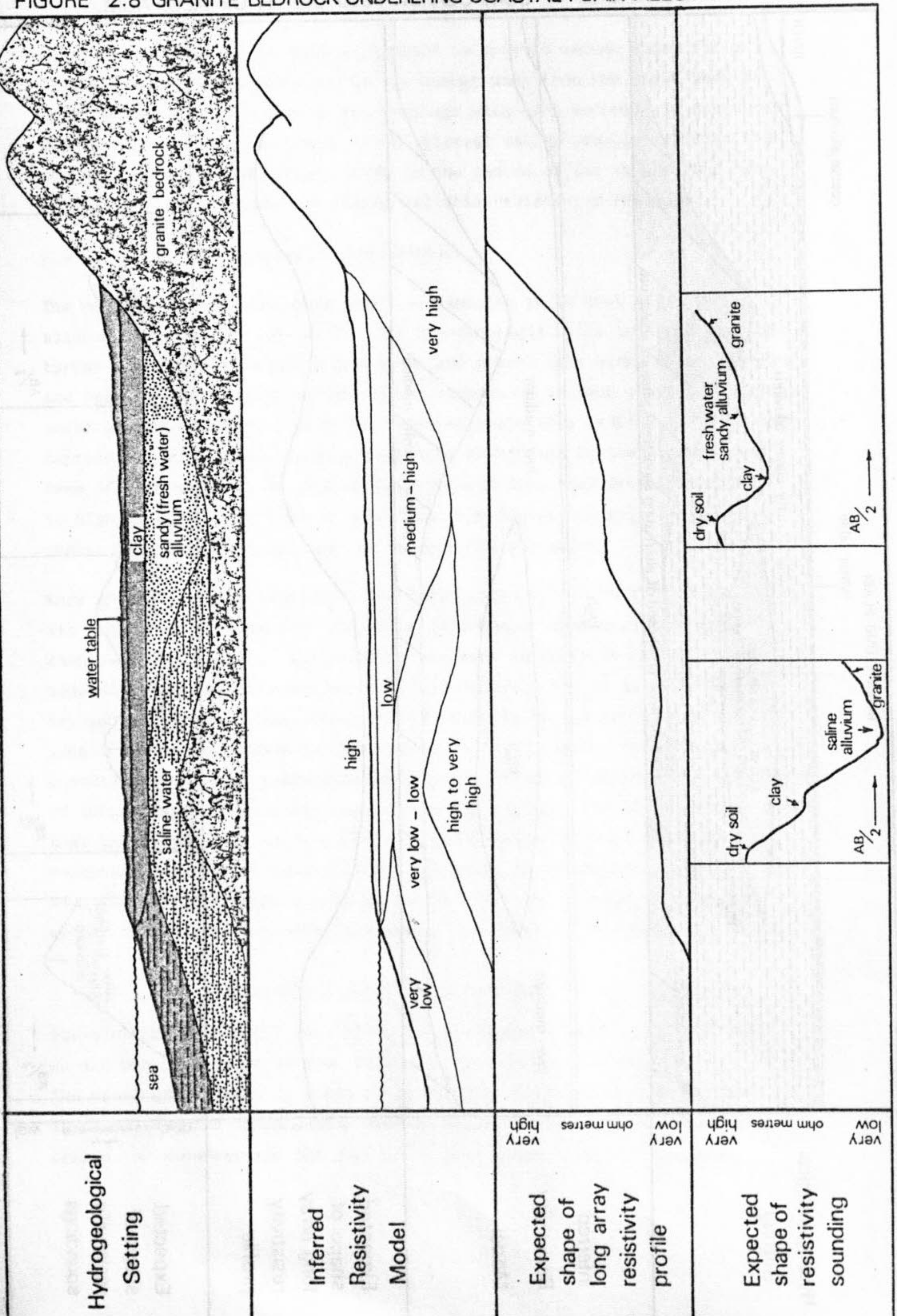


FIGURE 2.8 GRANITE BEDROCK UNDERLYING COASTAL PLAIN ALLUVIUM



No single instrument is suitable for all purposes, even though in their literature, instrument manufacturers may suggest otherwise. An instrument should be chosen depending upon its proposed use, the required level of portability, and the required accuracy of measurements.

The power source may be either direct current (dc) or low frequency alternating current (ac) preferably less than 60 Hz. The dc power source is usually composed of a set of 45 or 90 volt batteries connected in series; this is the power source for the Scintrex RSP-6 provided for the JPT groundwater section. Batteries possess the great advantage of being portable but they also have limited current capacity and a short working life. Ac current sources are usually motor driven generators or low-frequency sine-wave transistor oscillators, with a transformer output of a few watts. The generators are necessary for large scale work but are not portable; transistor oscillators can be used for smaller scale studies where portability is needed. Unidirectional dc current can cause electrolytic polarization. Therefore, on dc machines, the current direction should be reversed periodically. On the RSP-6, this has to be done by hand with a reversing switch but on more sophisticated instruments such as the ABEM Terrameter SAS 300, mechanical commutators, relay systems, or vibrators reverse the current. The rate of reversal can range from three or four times per minute to 100 times per second. The effect of these automatic reversals is that the dc current is effectively chopped up into a very low frequency, square wave, alternating current.

Each type of power source has particular advantages and disadvantages. The dc sources enable dc resistivities to be measured, and these are more accurate but the instrument also measures spontaneous potentials and this is a disadvantage. The spontaneous potentials (SP) at the movement site, have to be counteracted by a compensating voltage in the instrument. As has been found with the RSP-6, this is often a difficult and time consuming task, because the SP has to be measured manually and the compensating voltage adjusted accordingly between each attempted resistivity reading. In order to measure SP, it is necessary to reduce polarization effects at the potential electrodes by using porous pots for the electrodes. A metal electrode is suspended in the porous pot and the pot is filled with a saturated solution of the appropriate metallic salt; the RSP-6 uses copper electrodes immersed in copper sulphate solution. Contact with the ground is made by the copper sulphate solution seeping slowly through the pores at the bottom of the pot. Porous pots are a disadvantage because they have to be handled and positioned with greater care than steel electrodes.

The use of ac or rapidly interrupted square wave dc current means that SP does not have to be measured. In addition, narrow band amplifers, tuned to the power source frequency, can be used to increase the signal to noise ratio and more reliable readings can be obtained. However these current sources can cause problems because inductive coupling can occur between the potential and current cables where these lie close to each other. Also current leakage from poor insulation on the cables is likely to present a more serious problem. Both inductive coupling and current leakage cause erratic readings. Ac and interrupted dc sources can use steel electrodes. With all current sources good quality plastic covered cables should be used as poor quality, thin insulation and thin gauge wire can cause many problems in the field. It is impossible to completely avoid snagging or abrading the cables, and if breaks in the insulation or the wire occur, many hours can be spent by a geophysics crew hunting for a leak or break in continuity.

Modern resistivity metres are utilizing micro-processors which can handle many of the switchings and adjustments needed on the older manual machines; the micro-circuitry also means that the instruments can be very sophisticated but still light-weight and rugged. The reduction in the number of wires inside the instrument also means that problems caused by poor insulation or poor contacts are reduced. The RSP-6 instrument currently in use is continually beset by such problems. The ABEM Terrameter SAS 300 which may in future be used by JPT will make it possible for resistivity measurments to be made more rapidly and reliably.

2.5 Reading the Scintrex RSP-6

2.5.1 Introduction

This section is written with reference to the Scintrex RSP-6 instrument; two of these instruments have been used by the JPT groundwater geophysics section for the last year. These instruments are slow to use, are prone to malfunctions, and their measurement range is limited and an alternative type of instrument, an ABEM Terrameter, may soon be acquired. If so, the RSP-6 will perhaps be maintained for training purposes.

The basic principles of using the RSP-6 will also apply to the use of the ABEM Terrameter; subsequent sections on fault finding and maintenance are also relevant to both instruments. An annotated drawing of the face of the RSP-6 console is given (Figure 2.9); each knob, switch or connection referred to in the subsequent description has been given a reference number. It is assumed in the next section that an electrode array has been laid out and connected to the instrument; layout and other aspects of soundings and profiles are discussed in a later section.

2.5.2 Measurement Procedure

Preliminaries

i. connect the short cables between the cable reel terminals and the terminals on the instrument console; standard practice is that C_1 denotes the leading current electrode, C_2 the trailing current electrode, and similarly for the potential electrodes P_1 and P_2.

ii. check that none of the current cables cross, touch, or come close to the potential cables.

iii. take off the battery compartment lid, and attach the batteries to the leads. Experience has shown that it is wise to put all the batteries in a plastic bag and place them outside the instrument case; this reduces the risk of the instrument becoming "live".

iv. check the 22½ volt battery by switching knob 2 to "Battery Check". The other batteries have to be checked individually using a multi-tester (see section 2.5.3 on fault finding and maintenance.

v. turn knob 2 to "Set Red Mark" and calibrate the potentiometer by adjusting knob 3 (Red Mark Setting) so that the needle on meter 4 exactly overlies the red mark on the right hand side of the microampere scale.

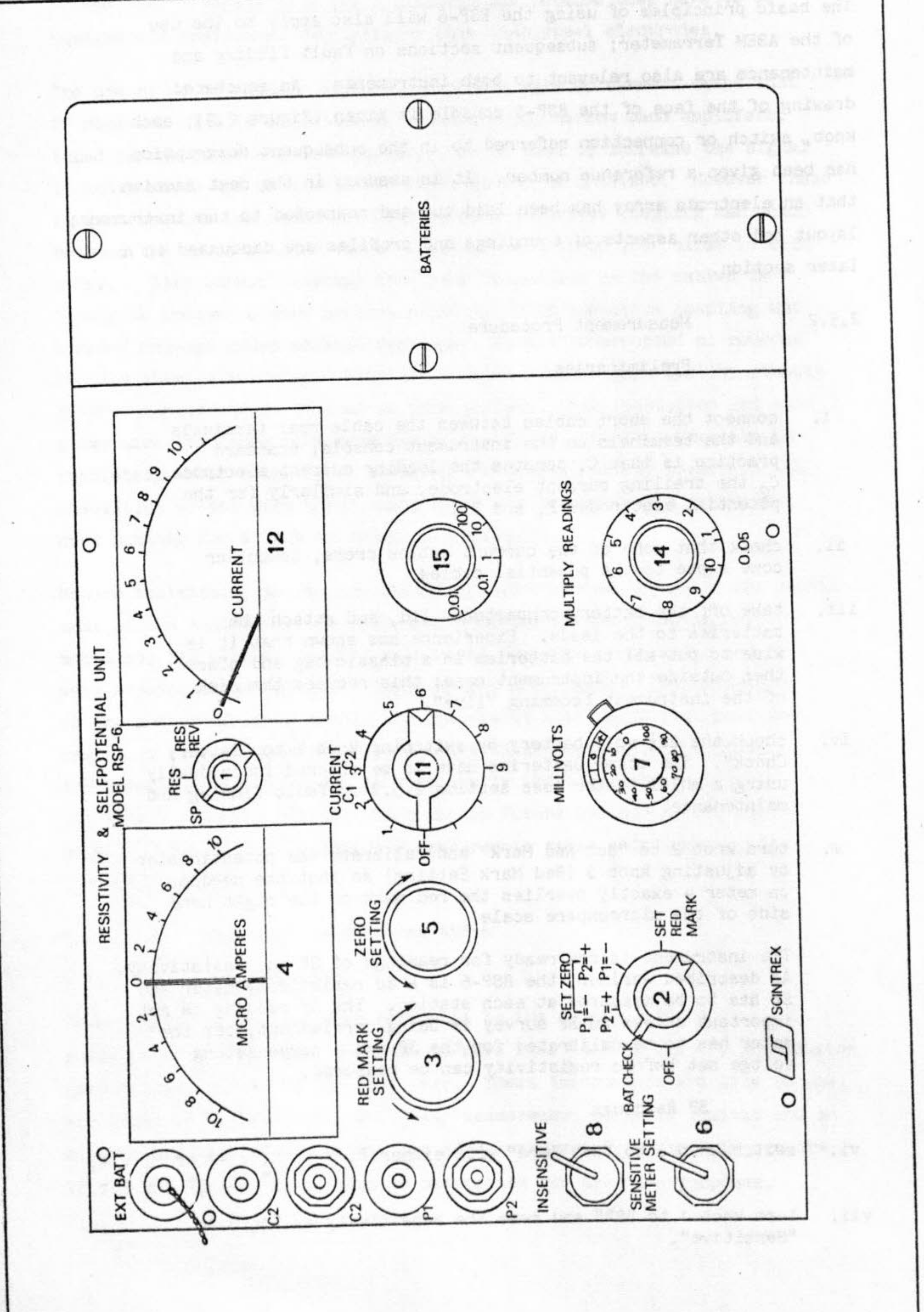
The instrument is now ready for readings of SP and Resistivity. As described earlier, the RSP-6 is a dc resistivity meter and SP has to be measured at each station. The SP reading is not important unless an SP survey is being carried out, but the meter has to be calibrated for the SP and a compensating voltge set before resistivity can be measured.

SP Readings

vi. switch knob 2 to "Set Zero" with either $P_1 = -$, $P_2 = +$, or $P_1 = +$, $P_2 = -$

vii. turn knob 1 to "SP" and move the sensitivity switch 8 to "Sensitive".

FIGURE 2.9 ANNOTATED CONSOLE OF RSP—6



viii. adjust the needle on meter 4 to 0 using the zero setting knob 5

ix. depress the "meter setting" switch 6 to "measure" and twirl the potentiometer knob 7 until the needle on meter 4 returns to zero. If the needle cannot be zeroed, go back to step vi and try the alternative setting on swtich 2; this reverses the polarity. Repeat steps vii to ix.

x. read off the SP in millivolts from the dial around potentiometer knob 7 and record the reading. If the set zero is on $P_1 = -$, $P_2 = +$ then local convention suggests that the SP reading is prefaced by the symbol +. If the knob is on the alternative setting, the reading is prefaced by the symbol -

At each new electrode position, it is quite normal for the SP readings to fluctuate during the first 1 to 5 minutes. It is worthwhile monitoring these fluctuations and waiting for the SP to reach "equilibrium" before proceeding to take resistivity readings. In extreme circumstances, such as when thunderstorms are imminent, the SP may change very rapidly and work should be abandoned. If work continues, SP and resistivity readings will be erratic and meaningless. Thunderstorms can create massive static charges which can be picked up by long electrode arrays and hence present a risk to both equipment and personnel.

Resistivity Readings

xi. once an SP reading has been obtained, quickly switch knob 1 to "Res" (Direct current direction resistivity).

xii. turn "Current" switch 11 to an appropriate setting and record both the setting and the current reading on meter 12, on the data sheet.

The current setting should be the highest possible which can still allow the needle meter 4 to be zeroed (see step xiii). With a very short electrode separation, it may be possible to only use current settings 1, 2 and 3. With longer separations, say greater than AB/2 of 10 metres, it is normal to use current setting 4, 5, 6 or, with the booster attached, settings 7, 8 and 9 and obtain a current reading of 4 to 8 on the scale on meter 12.

xiii. zero the needle on meter 4 using the zero setting knob 5.

xiv. for expected low resistance (<1 ohm), depress the sensitivity switch.

xv. simultaneously depress the measure switch 6 and adjust the galvanometer knob 14 and multiplier scale 15 to rezero the needle on meter 4. Record the resistance reading ($\Delta V/I$) on knob 14 multiplied by the value on the knob 15.

If the needle swings off centre to the right, turn knob 14 clockwise. If you still can't zero then move up a multiplier scale, e.g. from 0.1 to 1.0, and repeat. The procedure is reversed if the needle swings to the left.

xvi. turn 'Switch 1' back to SP and repeat the SP reading (steps vi to x).

xvii. turn switch 1 to "Res. Rev." and repeat steps xi-xv to get the reverse resistance reading

xviii. when the direct and reverse resistivity readings are either consistent, or consistently different, select the best or most appropriate value and multiply it by K (the electrode separation constant) in order to obtain the apparent resistivity (ρ_a) value.

The zeroing of the needle for measuring the resistance is often difficult as the needle may drift or appear to vibrate. If more than 10 seconds are required to complete the operation, it is usually more efficient to abort the resistivity reading and return to recalibrate the SP.

It is very difficult to get consistent readings on the 0.01 to 0.099 ohms scale.

Do not place much reliance on readings taken between 0.05 and 1.0 on knob 14. If you can, switch down to a lower multiplier, on knob 15.

The booster pack can be connected via a lead to socket 10. With the booster connected, the instrument frequently becomes "live" and the SP reading usually changes by 10 to 15 mV. No remedy has been found for this problem.

The "best or most appropriate" reading referred to in step xviii is not the average reading. It is the reading that looks the most credible with respect to the preceding values.

2.5.3 Fault Finding and Maintenance

Most faults relate to bad electrical contacts, weak batteries or contaminated electrodes. There are several points in the full electrical circuit which are vulnerable. Below are a series of comments on various sections of the system which may be of use to future resistivity operators.

Batteries

- the battery check switch only measures the charge on the 22½ volt battery; the others need to be tested with a multi-tester.
- weak 45 volt batteries may give a reading of 45 volts on the multi-tester if they have not been used for a few hours. A way of assessing the battery strength is to turn to resistivity and then turn the current setting knob through settings 3, 4, 5 and 6. At each turn of the knob, pause and watch the current needle; if it rapidly declines, it means the batteries can't maintain the voltage and they need replacing.
- check the battery terminals. The thin battery leads often break.
- the manufacturers literature states that the batteries are expected to last up to 6 months, but it does not qualify the statement by quantifying usage. Experience in Malaysia indicates that when working in areas with low resistivity, the batteries can be expended during the course of a 10 day field trip.

Connections, Cables and Cable Reels

- check that all terminals are clean and free from corrosion.
- check that the securing screws on top of the spring loaded carbon/graphite bushes on the cable reels are tight.
- check that all jack plugs and sockets are free from dirt and all securing screws are tight. Antistatic head cleaner aerosol spray is very useful for getting dirt and grease out of the sockets.
- check the wire bindings on the current electrodes. Periodically they need to be stripped, the stainless steel electrodes rubbing down with emery paper, and the bare wire rewinding. It is advisable to bind the wire with 3 M self-vulcanizing tape and then to cover with a final layer of ordinary electrical insulating tape.
- check the cables for breaks in the plastic insulation or breaks in the wire core. Tracing unexposed breaks in the wire core is very tedious. Try as a matter of course to avoid snagging the wire during profiling. During the first year of operation, there have been no breaks in the main cables.

Electrodes

- the current electrodes are robust and maintenance free.
- the porous pots by contrast require maintenance and careful handling. They need to be kept clean, but the outside surface should not be scraped with metal. The interaction of the particles of metal and the copper sulphate will create a spurious potential.
- try to ensure good contact between the earth and both the current and potential electrodes. Seat the pots on damp earth. If the ground is very dry, make a small hole for the pot and fill it with water. The current electrodes often need wetting and driving deep into the ground when either the surface layers are very dry or the current electrode separation is very large. Always try to get the highest current reading possible at each station.

Tool Kit and Ancillary Equipment

Below is a list of the basic requirements for the tool kit to accompnay the resistivity crew. Not all the items can be stored in a tool box and a large wooden box has been manufactured to hold large ancillary items.

Tool Box

Wire strippers
Snub nose pliers
Fine " "
Multi-tester
Tin snips
Flat and rounded files
Flat, and posi-drive headed screw drivers
Jewellers screw drivers
Emery Paper
Silicone rubber glue
Insulating tape
Self vulcanizing tape

Ancillary Equipment

100 m glass fibre tapes
30 m " " "
Hammers
Pick
Stools
Surveyors' Umbrellas

Clip-boards

2.6 Resistivity Soundings

2.6.1 General

Resistivity soundings are carried out in order to try and measure the change in resistivity with depth. The sounding is achieved by starting off with the electrodes assembled in a straight line about a fixed centre. Step by step, the electrodes are moved outwards along the same alignment and readings are taken at every step and the results are plotted on logarithmic graph paper. In order to achieve an even spread of data points on the logarithmic scale, the incremental increase in distance of the electrodes from the centre point is also logarithmic. A specimen data sheet is given (Figure 2.10). The final data plot of ρ_a against electrode separation is in effect a crude depth profile of the resistivities of the subsurface layers. One of the main problems of interpreting sounding curves is that the theory assumes that the subsurface is composed of distinct isotropic horizontal layers and that change in resistivity only takes place in the vertical plane. This is never correct, and lateral variations in thickness, or composition always occur at each sounding site.

2.6.2 Electrode Configurations

Wenner or Schlumberger arrays can be used for soundings. During the past year the JPT team have used Schlumberger arrays, for three reasons, as follows:

- the data derived using a Schlumberger array is less sensitive to horizontal changes in resistivity; this is because the potential electrodes remain static for four or five moves of the current electrodes.
- the results provide a means for recognising the magnitude of the lateral variations, and a method for attempting to correct them.
- the Schlumberger array is faster because it involves less electrode moves.

2.6.3 Field Procedure

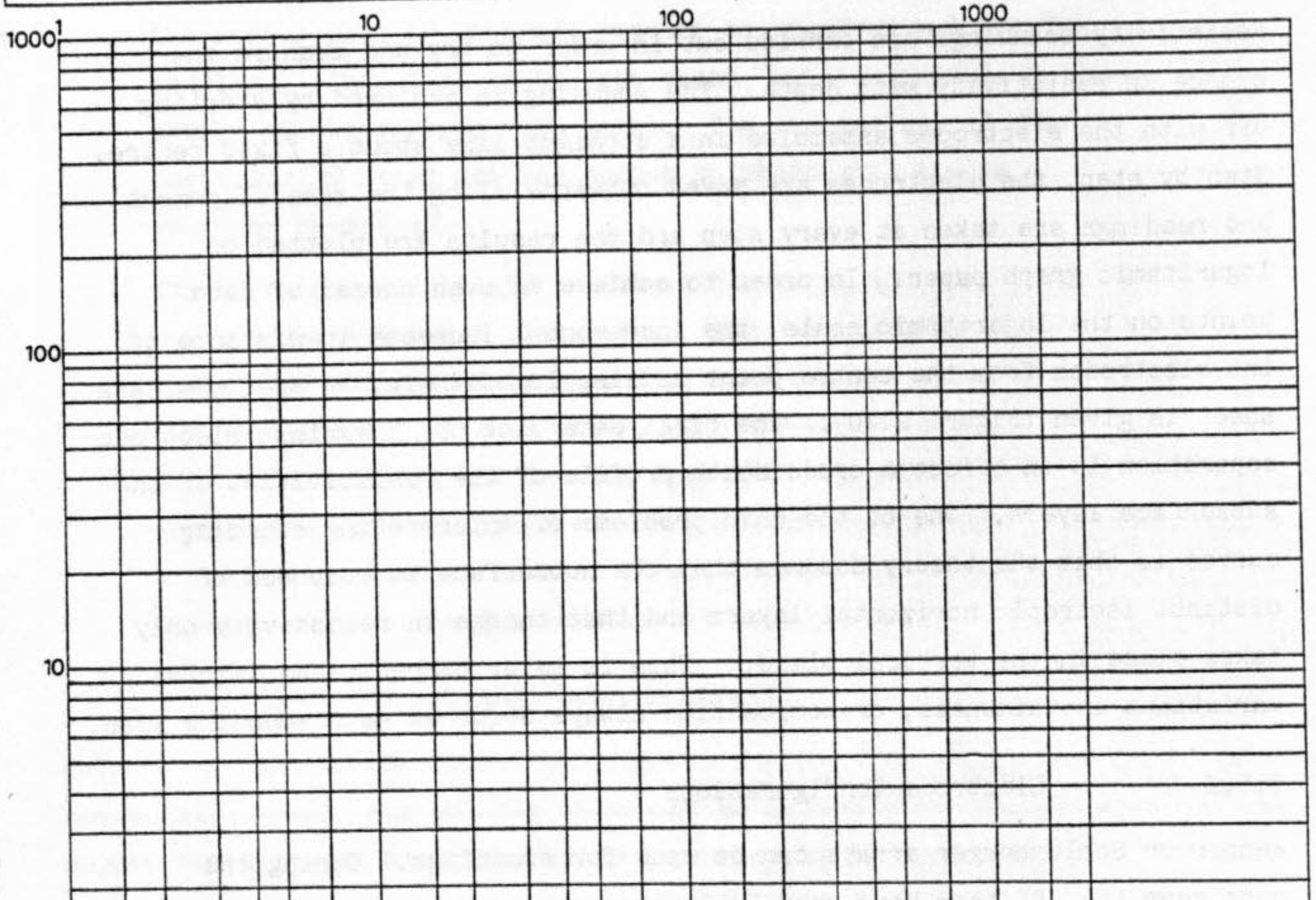
The first step in starting a sounding is to select the site. A site should have the following characteristics:

- there should be relatively straight and flat access for at least 100 metres and preferably 300 to 500 metres on both sides of the centre point.

FIGURE 2.10 ELECTRICAL SOUNDING DATA SHEET

JOB No
ELECTRICAL SOUNDING LOCATION No
DATE
OPERATOR

HUNTING SURVEYS LIMITED
ELSTREE WAY, BOREHAMWOOD
HERTFORDSHIRE WD6 1SB ENGLAND.



Sta-tion. no.	Mn/2	Ab/2	K	SP.	Current	Direct Readings	Reverse Readings	Best Value	ρ_a KR
1	0.25	10	5.9						
2		1.6	13.7						
3		2.0	24.7						
4	0.25	3	56.15						
5	0.75		17.7						
6		4	32.3						
7		5	51.2						
8		7	101.4						
9	0.75	10	208.3						
10	2.5		58.9						
11		15	137.4						
12		20	247.4						
13	2.5	30	561.6						
14	7.5		176.7						
15		40	323.3						
16		50	511.8						
17		70	1014						
18	7.5	100	2083						
19	2.5		589						
20		150	1374						
21		200	2474						
22		300	5616						

- the ground below the chosen alignment should, at least superficially, appear to be uniform both in terms of soil, drainage and topography for 30 metres on either side of the centre point
- the alignment should avoid passing across large bridges or culverts, rivers or canals, roads or railways, high voltage power lines, buried metal water pipes and large features such as swamps or roads
- where such features cannot be avoided, the alignment should be made parallel to such features.

To find such an ideal site is very difficult. Often, in flooded padi areas, the only access is along a raised laterite track with a canal on one side and a deep drain on the other. It is still worthwhile trying to do a sounding under these circumstances but it is important to try and keep the sounding alignment equidistant from the low resistivity features (i.e. the water) on both sides. Data collected from sites such as these should only be interpreted semi-quantitatively.

One a site has been selected, a metal stake is driven into the ground to denote the centre point and 100 metre tapes should then be run out in opposite directions; these tapes are used to precisely locate the positions for the electrodes. Using the Schlumberger array, the first electrode positions are for the pots (potential electrodes P_1 and P_2) to be placed 0.25 metres on either side of the centre point stake. The current electrodes first position is at 1 metre from the stake.

Whilst the crew are setting out the tapes, reels and electrodes, the technician or the geophysicist should be filling in the following reference information on the data form:

- JPT Electrical Sounding (E.S.) Number
- Geographic Location and Date and Operator
- A detailed location map showing the centre of the array in relation to permanent landmarks, hazards that are likely to cause data anomalies, and the alignment of the sounding annotated with the magnetic bearing.

- comments on ground conditions, water levels and the weather.

It is also useful if one member of the team carries out a search for nearby dug wells so that a measurement of the depth to the static water level and groundwater salinity can be obtained. If no well is available in the vicinity and yet the water table appears to be close to the surface it is a good idea to dig a shallow well near to the sounding alignment. The section seen in the sides of the well plus the depth to water can be useful for calibrating the early part of the sounding curve.

2.6.4 Taking Readings

The first readings usually require low current settings and the measurements are easy and rapid; care should be exercised because these first few readings define the resistivity of the top lyaer, to which all others are related. When the current electrodes have reached an AB/2 = 3 m, two readings are taken. The first is with the potential electrodes in their initial position, or MN/2 = 0.25 m, the second is with the potential electrodes at 0.75 m. If there is no change in the near surface layers below the two potential electrode positions, the calculated resistivity values should be exactly the same. If there is a considerable difference between the values, it may be worth moving the entire array and starting again.

A sounding can be carried out by one man but it is faster and more efficient if the instrument operator confines his attention to obtaining the readings, and a second member of the team records the data. This second person also fulfills another fucntion; he can calculate the apparent resistivity values as the readings are made, and he can tell the operator whether the readings fit with the pattern of the preceding data thereby alerting the operator to any gross anomalies. As the electrode spread length increases, measurements usually become more difficult, and the values for direct and reverse readings often differ. The current setting should always be as high as posssible and the current electrodes will frequently need to be driven deep into the ground to ensure good contacts. Around an AB/2 = 30 m, the computed apparent resistivity value may be 100 ohm metres or less whilst the instrument reading may be less than 1 or even 0.1 ohms. These resistance values are well within the measurement range of the RSP-6, yet often a spacing of AB/2 = 30 m marks the beginning of measurement difficulties, and even greater care and concentration is required to obtain reliable and consistent readings. Frequently, between 5 and 20 measurements are needed for both direct and reverse current directions. When measuring low resistances, efforts should be made to prevent contact between the operators fingers and metal parts of the instrument console. It has been found that operators develop a static electricity charge after one or more hours at one instrument position and accidental contact between operator and instrument often causes a needle deflection greater than that created by the voltage potential between the two pots. There is no easy solution to this problem with the RSP-6.

The length of a sounding array depends upon the site, the objective and the validity of the measurements. In areas where the deep subsurface alluvium contains saline groundwater, it may not be possible to obtain readings at several stations because the resistance is below the measurement capacity of the instrument. However experience has shown that it is worthwhile to extend the array out either to the limits of the site for the maximum length of the cables. Often higher resistance readings can be obtained at, for example AB/2 = 300 or 400 m, and these give a general idea of the proximity of the bedrock.

2.7 Resistivity Profiles

2.7.1 Objectives

The primary objective of resistivity profiling is to try and measure the spatial variation in apparent resistivity and profiling is a useful qualitative method of determining changes in bulk resistivity over an area. It can be aimed at defining the position, orientation and sometimes inclination of vertical or near vertical structural discontinuities. Type curves have been published in order to encourage a more quantitative analysis but in most cases this level of precision is not required. As electrode spacing is kept constant and the whole array is moved in steps, the profile gives no information on depth variations.

2.7.2 Array and Site Selection

Any array can be used for profiling but usually the choice is restricted by site access, the type of equipment and the resolution required. Wenner and Schlumberger arrays have been used by the JPT geophysics team. Initially Schlumberger arrays were favoured but were found unsuitable for use in certain saline areas with the Scintrex RSP-6; the potential differences in saline areas often gave resistance values that fell below the measurement range of the equipment. Wenner arrays were used in order to get overall higher potential differences to make measurements easier. Use of a Wenner array, where the distance between each electrode is the same, also means that field procedure is simplified and faster.

Site selection and features to be avoided are as discussed in an earlier section on electrical soundings. As the interpretation of a profile is more qualitative than a sounding, avoidance of features such as rivers and roads is less critical. It is important to choose a route for the profile that is either straight or a series of long straight sections broken up by simple bends; this is because if the electrode array is to be on a tortuous path, this effectively shortens the distance between the electrodes and invalidates the electrode separation constant. Roads and laterite tracks have an obvious appeal as profile routes but road margins or the centre of the track should be avoided. This is because even with a long electrode spacing (for example, where a = 100 m) the resistivity variation may be more a measure of the variation in the hard core/laterite composition than that of the alluvium and bedrock below. If possible, electrodes should be placed in the fields below the embankment at the side of the road whilst the landrover can still carry the instrument cables and reels.

2.7.3 Electrode Configurations

The layout of equipment for a profile differs from that used for a sounding. Manpower is also more important because the electrodes, cables and sometimes the instrument have to be moved in unison. Layout for a Wenner array was shown earlier (Figure 2.4). To save manpower, the field vehicle is sited next to the leading potential electrode so that the instrument operator can pick up and carry the pot and binding post during each move; three men are required to pick up, carry and place the other electrodes. At the start of the profile, tapes are used to fix the electrode positions. Once the positions have been fixed, the cables are tied to a metal stake or binding post next to P_1 electrode and the tapes can then be rolled up. Whilst the instrument operator is taking the reading, the leadman at C_1 digs a small hole by the side of his electrode; this hole is the next site for the pot (P_1). The advantage of using a Wenner configuration is that all the electrodes are equidistant. If the geophysicist choses to use the same distance (a) between each reading or station, then all electrodes (bar C_1) always move into the old position of the electrode in front. The new position for C_1 is controlled by the C_1 cable tied to the binding post.. The man holding the C_1 electrodes simply feels a tug on his cable when P_1 and the field vehicle or instrument has reached his previous position.

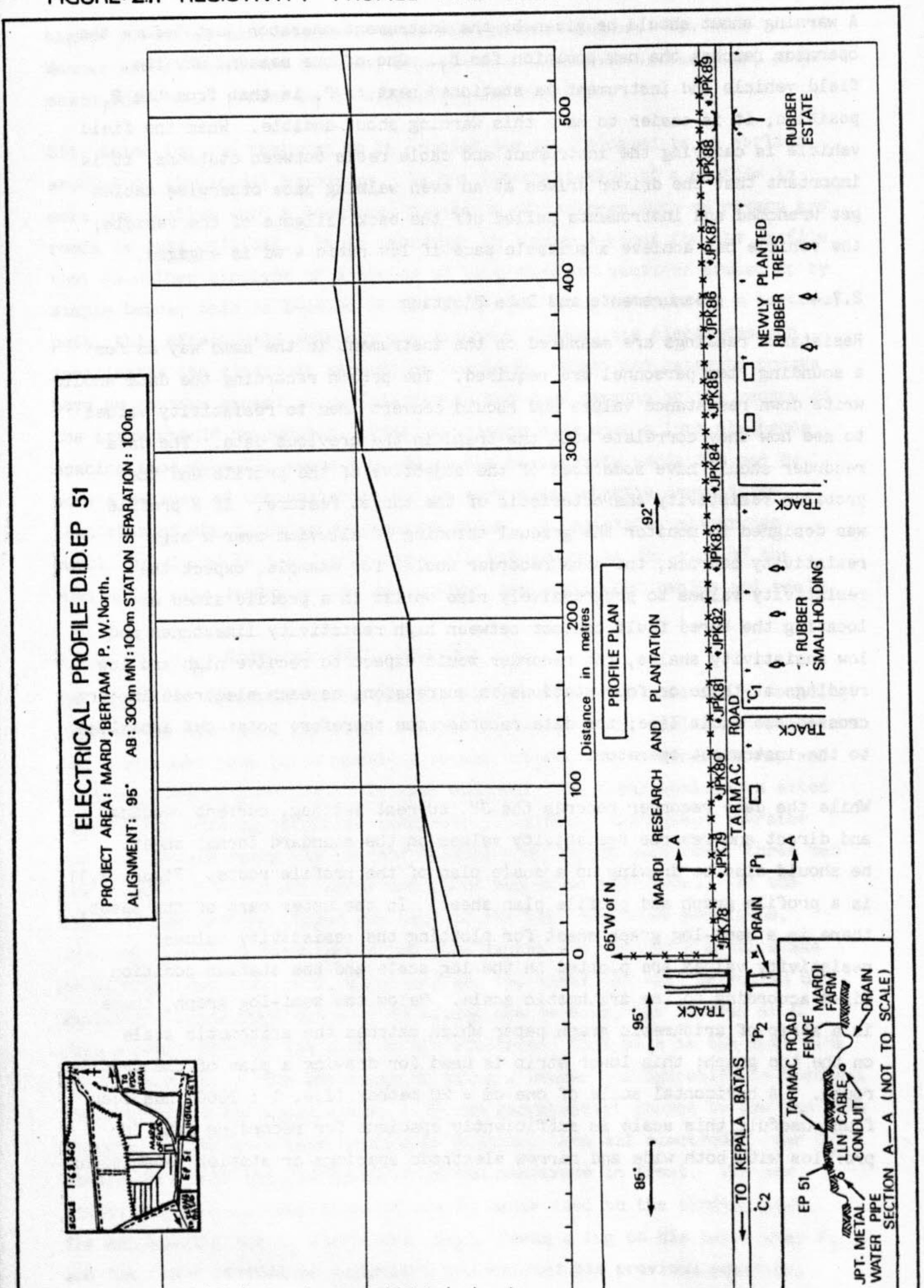
A warning shout should be given by the instrument operator just before the operator reaches the new position for P_1. One of the reasons why the field vehicle and instrument is stationed next to P_1 is that from the P_1 position, it is easier to make this warning shout audible. When the field vehicle is carrying the instrument and cable reels between stations, it is important that the driver drives at an even walking pace otherwise cables get wrenched and instruments pulled off the back tailgate of the vehicle; the vehicle can achieve a suitable pace if low ratio 4 wd is engaged.

2.7.4 Measurements and Data Plotting

Resistance readings are measured on the instrument in the same way as for a sounding; two personnel are required. The person recording the data should write down resistance values and should convert them to resistivity values to see how they correlate with the trend in the previous data. The data recorder should have some idea of the objective of the profile and the probable resistivity characteristic of the target feature. If a profile was designed to monitor the gradual thinning of alluvium over a high resistivity bedrock, the data recorder would, for example, expect the resistivity values to progressively rise whilst in a profile aimed at locating the bured fault contact between high resistivity limestones and low resistivity shales, the recorder would expect to receive high and low readings at three or four stations in succession, as each electrode in turn crossed the fault line; the data recorder can therefore point out anomalies to the instrument operator.

While the data recorder records the SP, current setting, current reading and direct and reverse resistivity values on the standard format sheet, he should also be drawing up a scale plan of the profile route. Figure 2.11 is a profile graph and profile plan sheet. In the upper part of the sheet, there is a semi-log graph sheet for plotting the resistivity values; resistivity values are plotted on the log scale and the station position fixed according to the arithmetic scale. Below the semi-log graph, there is a strip of arithmetic graph paper which matches the arithmetic scale on the top graph; this lower strip is used for drawing a plan of the profile route. A horizontal scale of one cm = 20 metres (i.e. 1 : 2000) has been found useful; this scale is sufficiently spacious for recording data for profiles with both wide and narrow electrode spacings or station separations.

FIGURE: 2.11 RESISTIVITY PROFILE AND PLAN PROFILE



The object of the plan is to fix the position of each measurement station in relation to permanent and obvious features along the profile alignment; this is done so that the position of each station can be reploteed on a larger scale map and also so that the station position can be identified by either the geophysics crew, the hydrogeologist or the drillers.

The compilation of the plan can become time-consuming if the plotter is too ambitious. He should merely aim to fix the position of each or every other station in relation to some recognisable feature.

This discussion of field procedure emphasizes the need for either a geophysicist and a technician, or two technicians to take resistivity readings and plot the data; plotting by one person leads to slow progress and often to suspect data.

2.7.5 Examples

Examples of profiles made by the JPT geophysics section are given below:

- in Figure 2.12 an east-west profile run in the Bukit Chuping area of Perlis is interpreted as a Chuping limestone block downfaulted against meta shale rocks of the Kubang Pasu Formation
- an intepretation of a profile run north-south across a valley in the Bukit Gantang district, Perak which is flanked by granite hills (Figure 2.13). Resistivity was used to estimate alluvial thickness in the valley.
- an interpretation of an east-west profile across on area in the Chuping Gula Estate, Perlis (Figure 2.14). The area was known to contain isolated upstanding outcrops of Chuping limestone and a subdued outcrop of meta sedimentary rocks. Resistivity was used to attempt detection of hidden downfaulted limestones; the limestone is the principle aquifer in the area.
- an interpretation of profiles run along a track cutting metasedimentary rocks of the Semanggol Formation, in Padang Terap Gula Estate; geological control was available at close by road cuttings (Figure 2.15). Profiling was able to give gross indications of sand or shale lithology although subsequently, such lithologies in this metamorphic rock were shown by drilling to be unrelated to permeability and aquifer potential.

FIGURE 2.12 PROFILE INTERPRETATION: BUKIT CHUPING

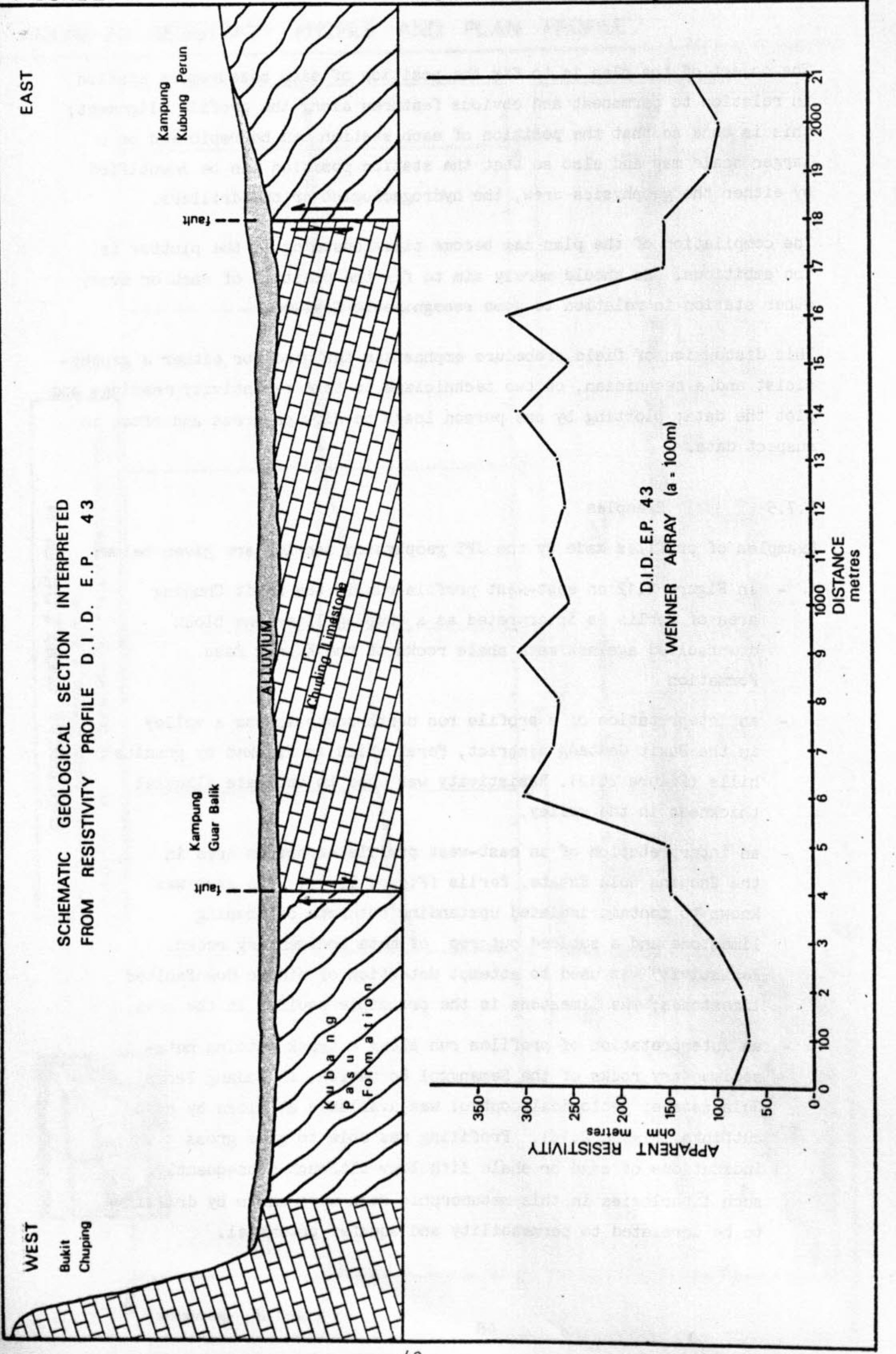


FIGURE 2.13 PROFILE INTERPRETATION: BUKIT GANTANG

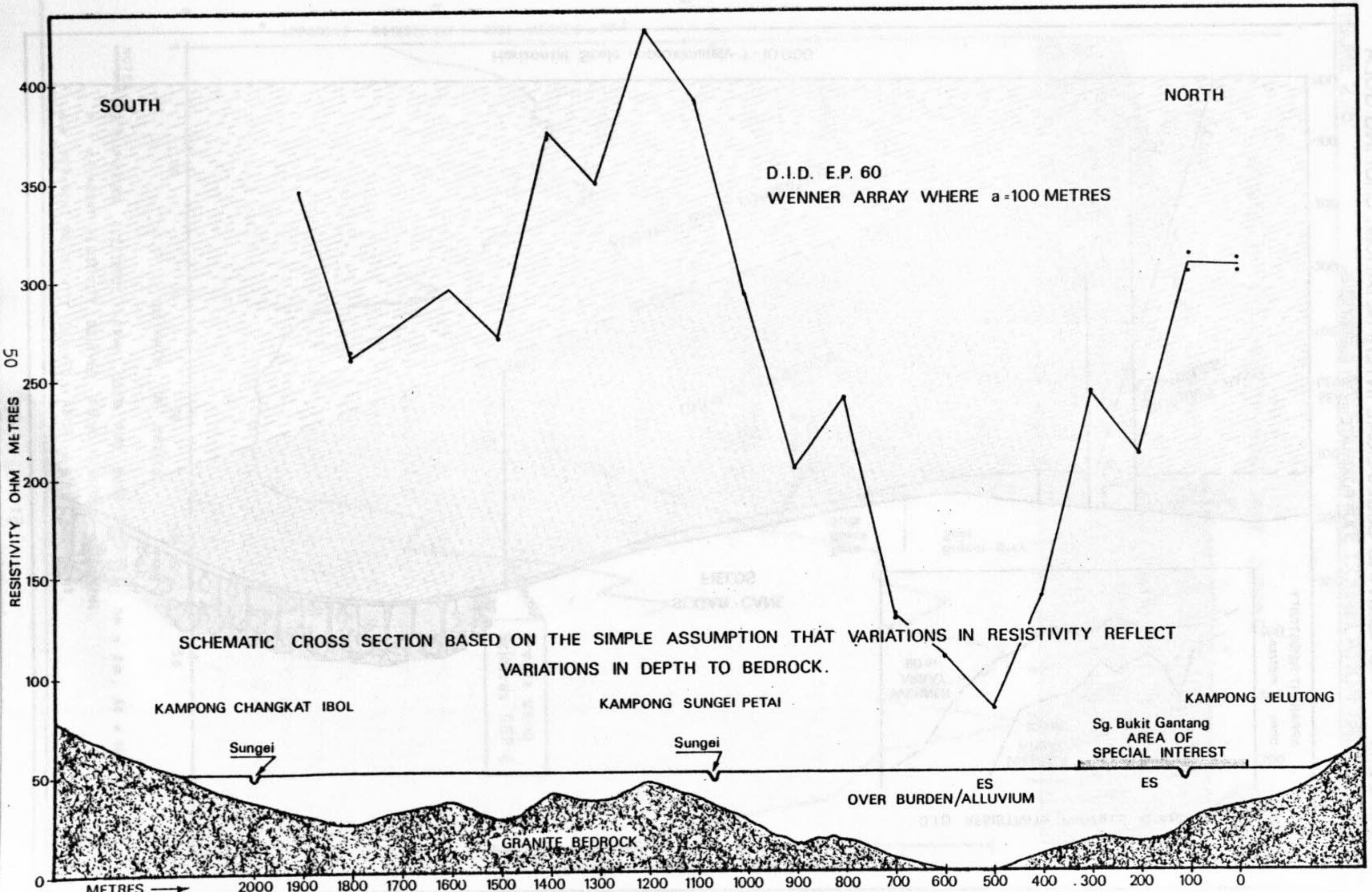
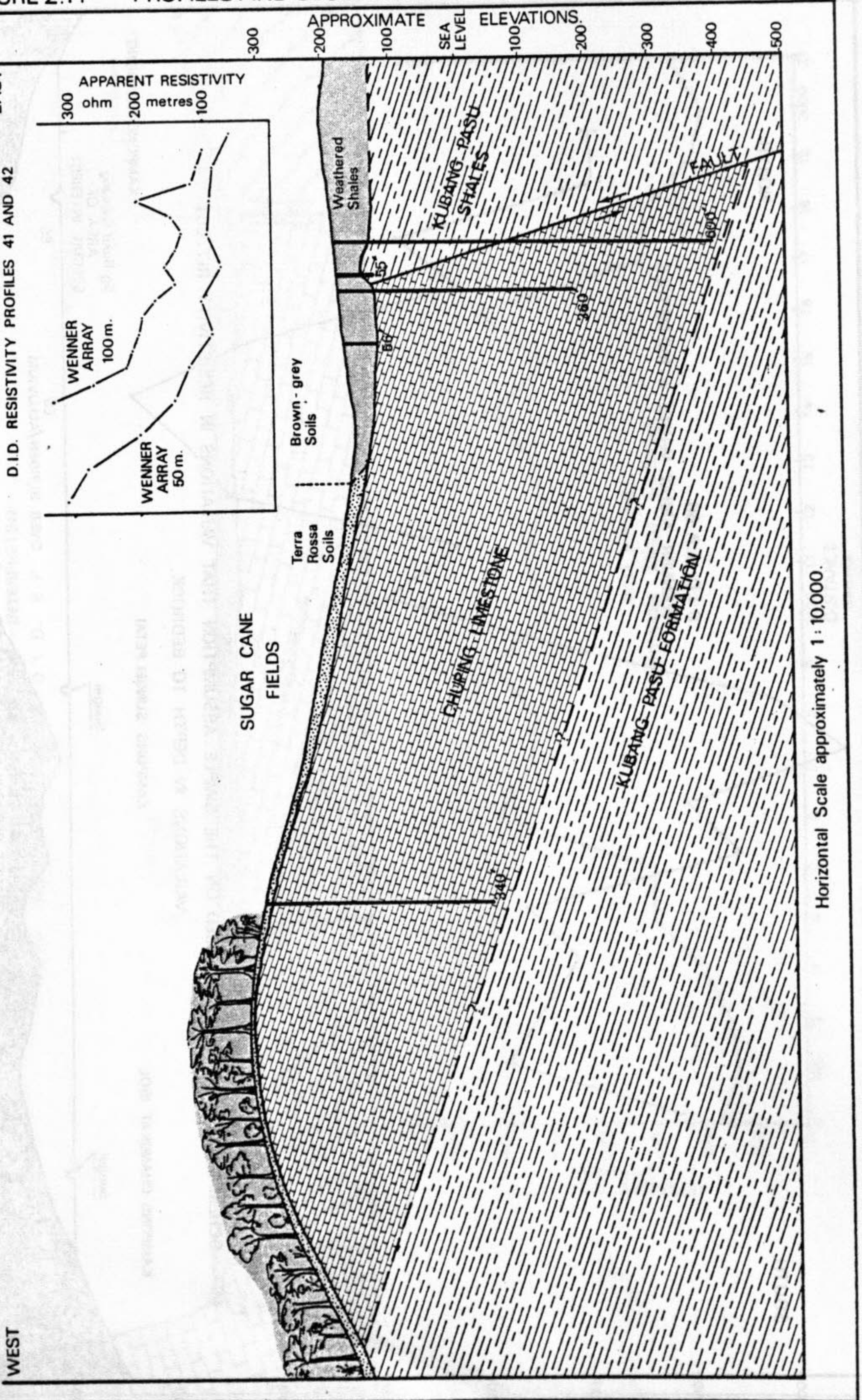
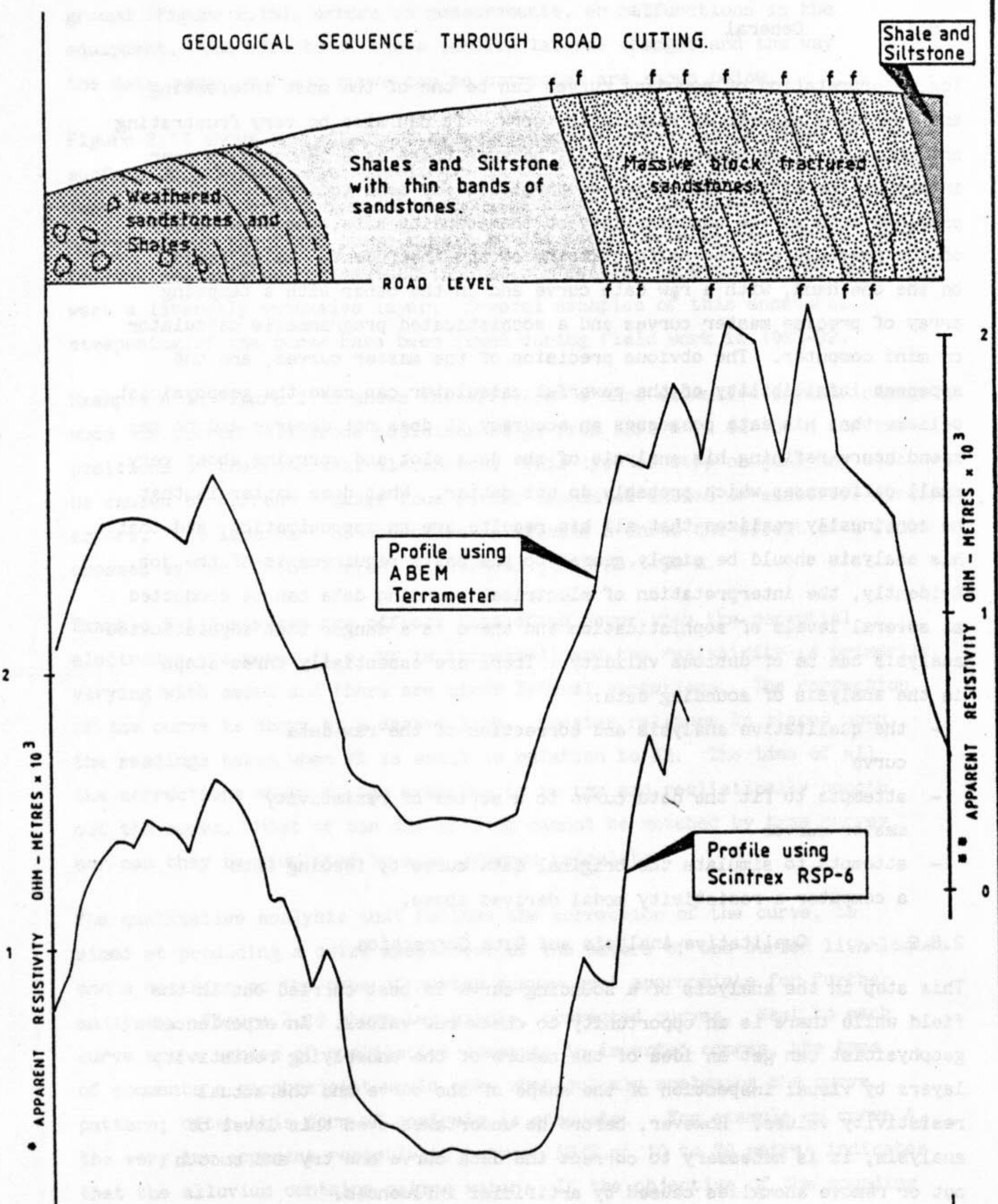


FIGURE 2.14 PROFILES AND GEOLOGICAL INTERPRETATION: CHUPING GULA



Horizontal Scale approximately 1 : 10,000.

FIGURE 2.15 PROFILE AND ROCK TYPE CORRELATION: PADANG TERAP



NOTE: SCHLUMBEGER ELECTRODE ARRAY USED FOR BOTH PROFILES; MN = 2m, AB = 10m.

* APPARENT RESISTIVITY DERIVED FROM SCINTREX EQUIPMENT.

** APPARENT RESISTIVITY DERIVED FROM ABEM EQUIPMENT.

2.8 Interpretation of Sounding Curves

2.8.1 General

The interpretation of sounding curves can be one of the most interesting and rewarding aspects of resistivity work. It can also be very frustrating and cause disillusionment, primarily because there is a tendency to over interpret the data. The office geophysicist may tend to forget field problems such as the unsuitability of the sounding site, the uncertainty of the readings and the malfunctioning of the instrument. He is faced, on the one hand, with a raw data curve and on the other with a tempting array of precise master curves and a sophisticated programmable calculator or mini computer. The obvious precision of the master curves, and the apparent infallibility of the powerful calculator can make the geophysicist believe that his data possesses an accuracy it does not deserve and he may spend hours refining his analysis of the data plot and worrying about very small differences which probably do not matter. What does matter is that he continually realizes that all his results are an approximation, and that his analysis should be simply geared to the basic requirements of the job. Evidently, the interpretation of electrical sounding data can be conducted at several levels of sophistication and there is a danger that sophisticated analysis can be of dubious validity.. There are essentially three steps in the analysis of sounding data:

- the qualitative analysis and correction of the raw data curve
- attempts to fit the data curve to a series of resistivity master curves
- attempts to simulate the original data curve by feeding into a computer a resistivity model derived above.

2.8.2 Qualitative Analysis and Data Correction

This step in the analysis of a sounding curve is best carried out in the field while there is an opportunity to check raw values. An experienced geophysicist can get an idea of the nature of the underlying resistivity layers by visual inspection of the shape of the curve and the actual resistivity values. However, before he undertakes even this level of analysis, it is necessary to correct the data curve and try and smooth out or remove anomalies caused by artificial influences.

The sounding curve is often distorted by lateral heterogeneities in the ground (Figure 2.16), errors in measurements, or malfunctions in the equipment. The effects of these shallow lateral changes and the way the data curve or field curve can be corrected are shown below.

Figure 2.17 shows the effect of a small patch of gravel at or close to the surface or a buried metal pipeline. In this case, a buried high resistivity lens of sand or gravel directly below the centre of the sounding causes a sharp peak on the field curve. Next to the field curve is the normal curve for the resistivity contrast (20:250:20 ohm metres) expected if the lens were a laterally extensive layer. Several examples of this unnatural steepening of the curve have been found during field work in 1981-82.

Example A in Figure 2.18 shows the effect of a more extensive surface change when the current electrode positions range from AB/2 = 10 to 100 m for two positions of the potential electrodes. This type of step or jump may also be caused by current leakage from poorly insulated cables or electrode spacing errors. The last part of the curve in Example A shows the effect of a fault crossed by one of the current electrodes at AB/2 = 150 m.

Example B illustrates the offsets that often occur when the potential electrodes are moved (i.e. MN is increased) and the resistivity is primarily varying with depth and there are minor lateral variations. The correction of the curve is shown by a dashed line. Greater reliance is placed upon the readings taken when MN is small in relation to AB. The idea of all the corrections shown in the examples is to try and realistically smooth out the curve. Most of the distortions cannot be matched by type curves, nor can they be simulated by a programmed calculator.

The qualitative analysis that follows the correction of the curve, is aimed at producing a quick assessment of the nature of the buried lithologies and a decision on the type of master curves most appropriate for further analysis. Figure 2.19 shows two simple, corrected curves. Next to each curve are a series of qualitative comments in inverted commas, the type of comments a geophysicist would make when quickly analysing the curve pattern; often this form of analysis is adequate. For example on curve A, the very low apparent resistivity between AB/2 of 10 to 70 metres indicates that the alluvium contains saline water. If the objective of the sounding is to find a suitable site for a borehole for irrigation supply, the geophysicist can without further analysis say that curve A indicates that

FIGURE 2.16 ELECTRICAL SOUNDINGS: DISTORTION CAUSED BY LATERAL HETEROGENEITY

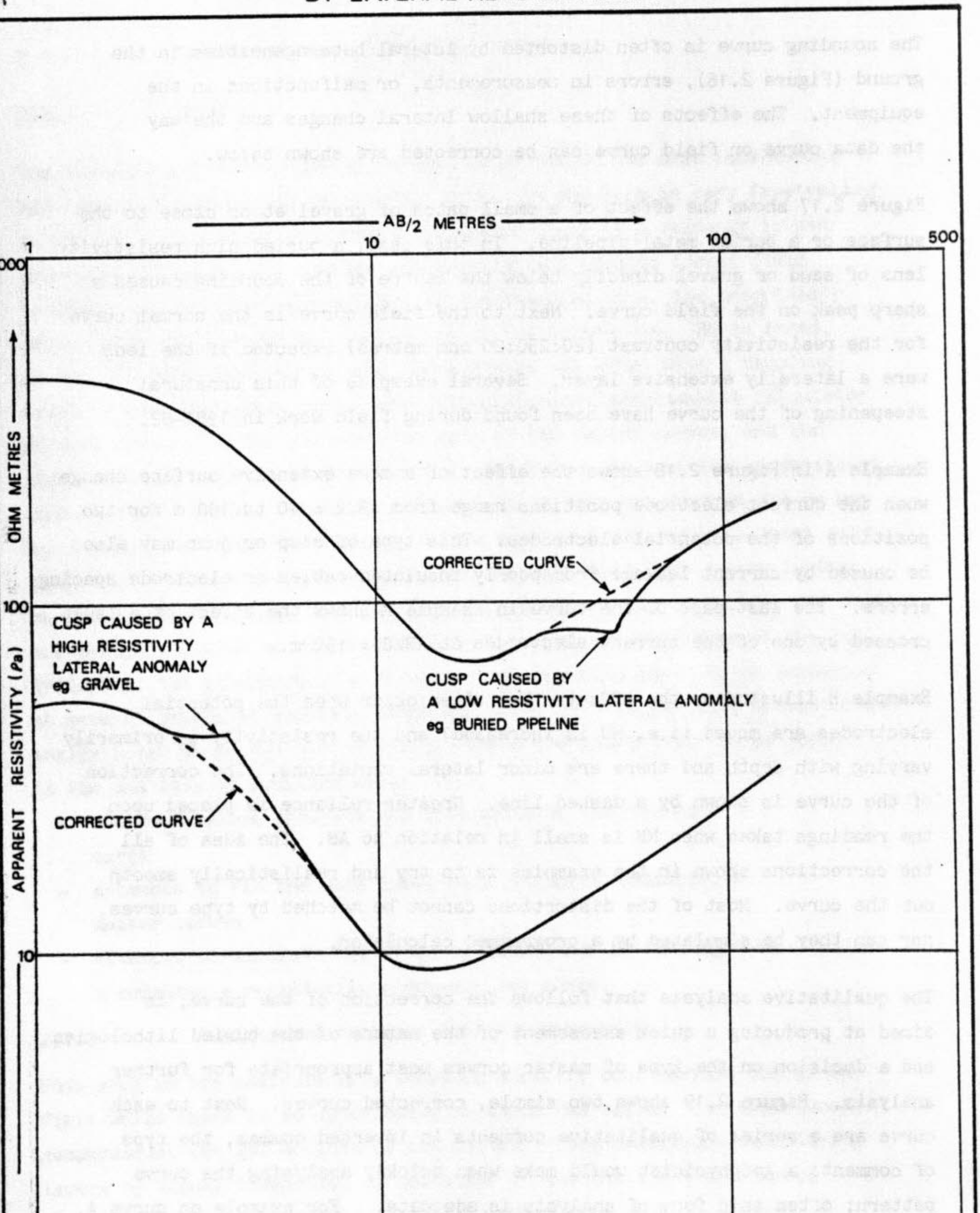


FIGURE 2.17 ELECTRICAL SOUNDINGS: DISTORTION CAUSED BY RESISTIVE LENS

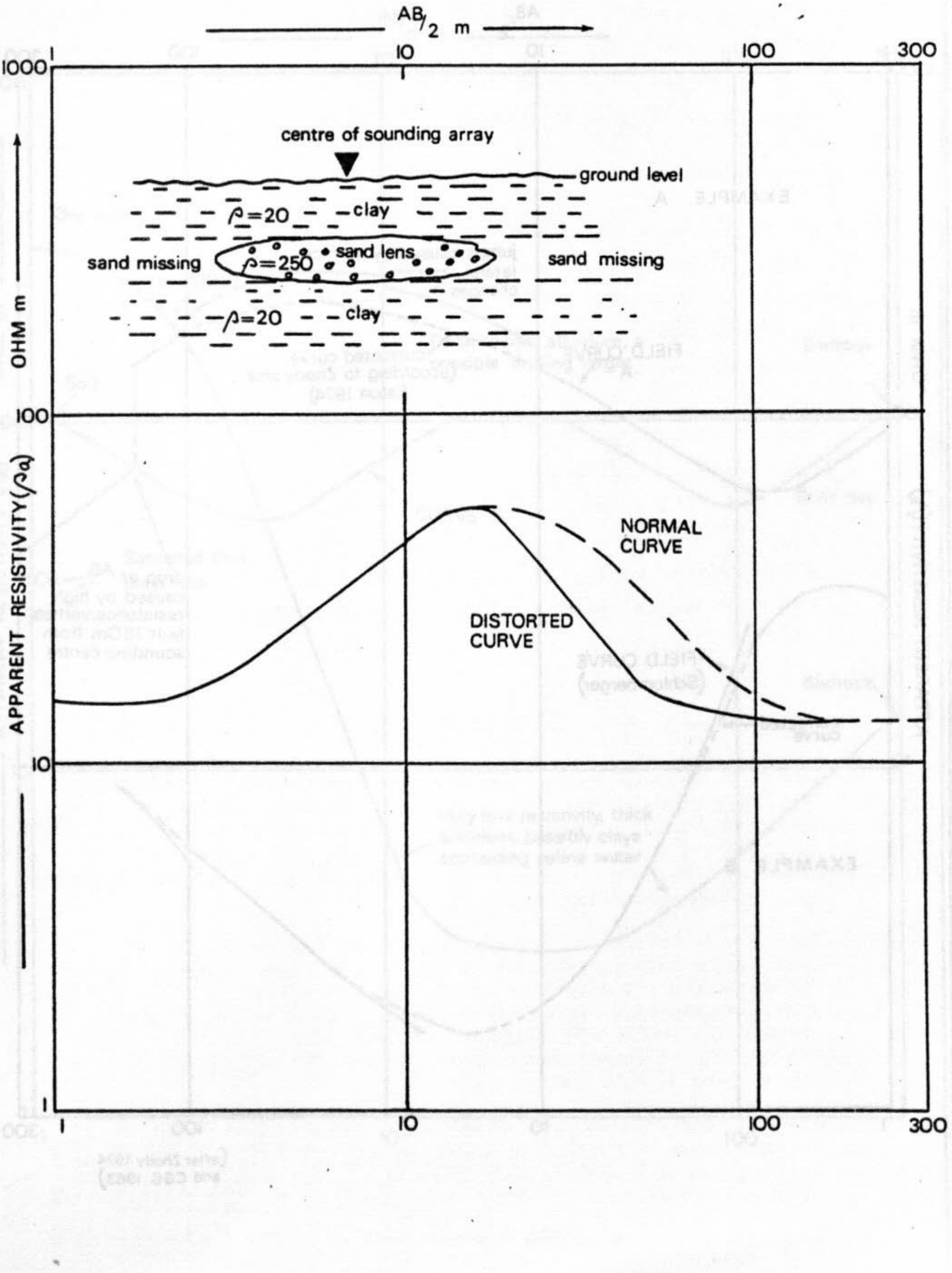
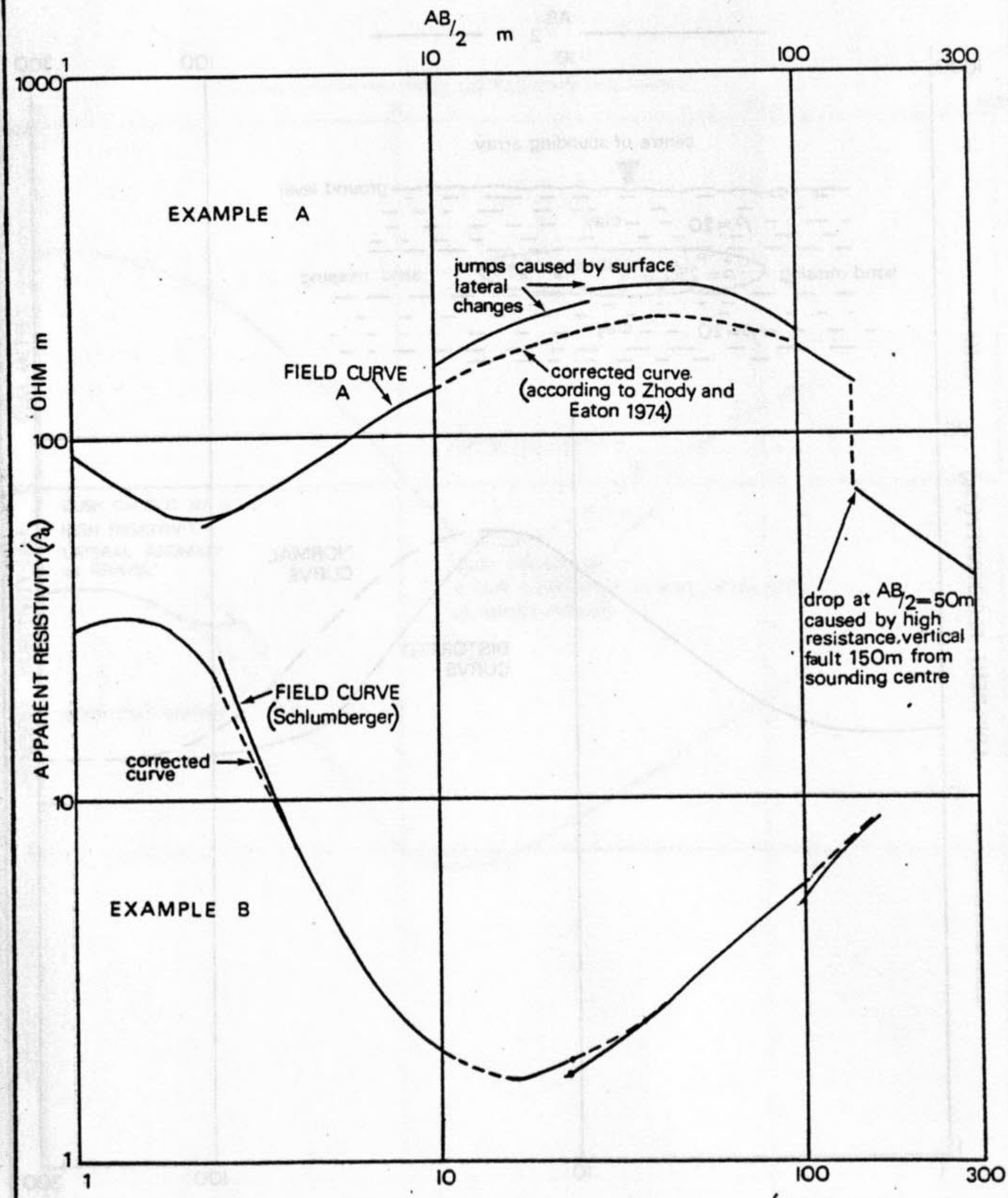
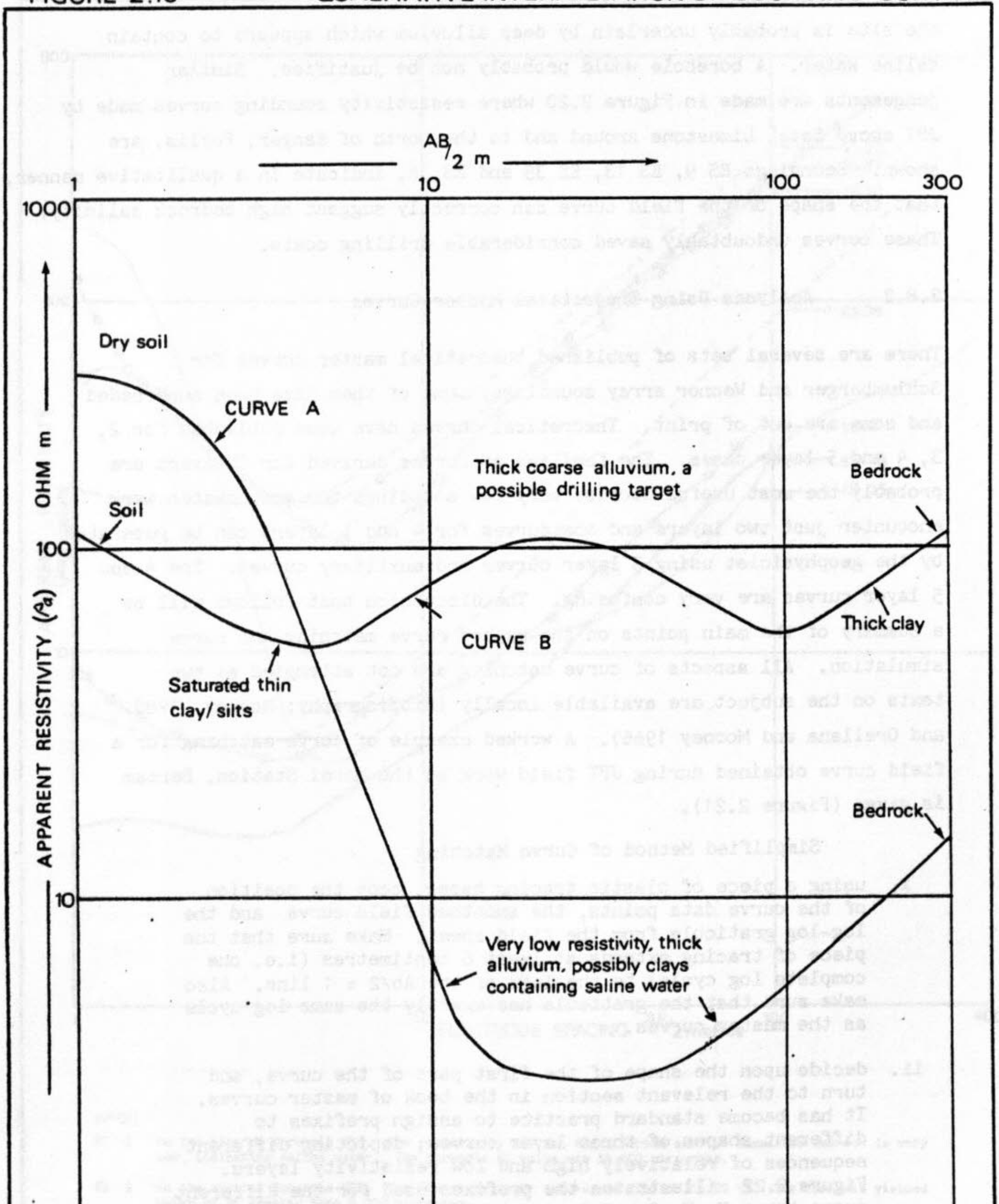


FIGURE 2.18 ELECTRICAL SOUNDINGS
DISTORTIONS CAUSED BY ELECTRODE DISPLACEMENT



(after Zhody 1974 and CGG 1963)

FIGURE 2.19 QUALITATIVE INTERPRETATION OF SOUNDING CURVES



the site is probably underlain by deep alluvium which appears to contain saline water. A borehole would probably not be justified. Similar judgements are made in Figure 2.20 where resistivity sounding curves made by JPT above Setul Limestone around and to the north of Kangar, Perlis, are shown. Soundings ES 9, ES 13, ES 35 and ES 36, indicate in a qualitative manner, that the shape of the field curve can correctly suggest high bedrock salinity. These curves undoubtably saved considerable drilling costs.

2.8.3 Analysis Using Theoretical Master Curves

There are several sets of published theoretical master curves for Schlumberger and Wenner array soundings; some of them have been superseded and some are out of print. Theoretical curves have been published for 2, 3, 4 and 5 layer cases. The families of curves derived for 3 layers are probably the most useful because very few soundings for groundwater work encounter just two layers and the curves for 4 and 5 layers can be generated by the geophysicist using 3 layer curves and auxiliary curves. The 4 and 5 layer curves are very confusing. The discussion that follows will be a summary of the main points on the method curve matching and curve simulation. All aspects of curve matching are not attempted as two texts on the subject are available locally (Bibliography; Mooney, 1980 and Orellana and Mooney 1966). A worked example of curve matching for a field curve obtained during JPT field work at the Mardi Station, Bertam is given (Figure 2.21).

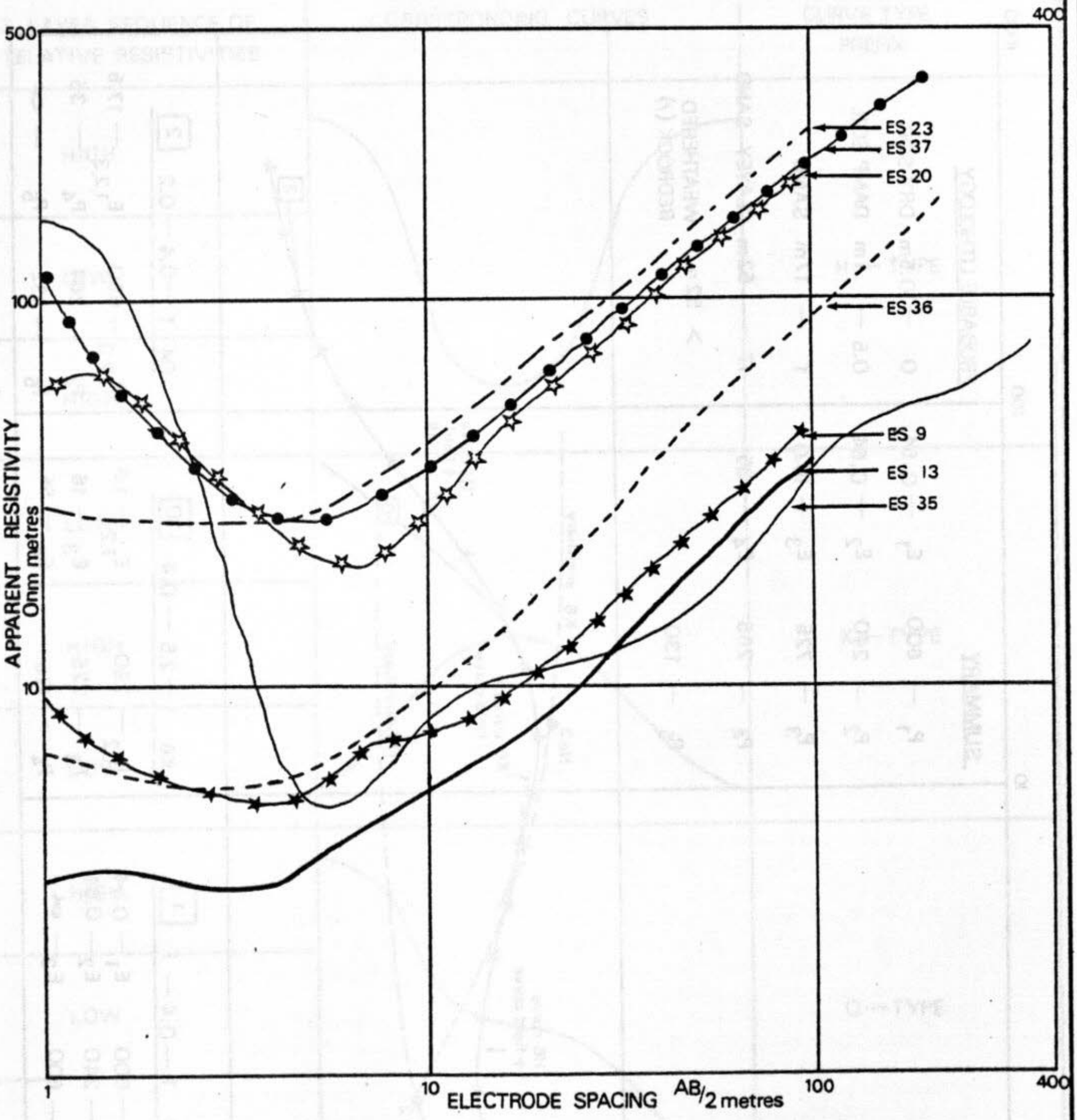
Simplified Method of Curve Matching

i. using a piece of plastic tracing paper, copy the position of the curve data points, the smoothed field curve and the log-log graticule from the field sheet. Make sure that the piece of tracing extends at least 6 centimetres (i.e. one complete log cycle) to the left of the Ab/2 = 1 line. Also make sure that the graticule has exactly the same log cycle as the master curves.

ii. decide upon the shape of the first part of the curve, and turn to the relevant section in the book of master curves. It has become standard practice to assign prefixes to different shapes of three layer curves, depicting different sequences of relatively high and low resistivity layers. Figure 2.22 illustrates the prefixes used for the different curve types H, K, Q and A.

iii. try to fit the early part of the field curve to one of the master curves. Always keep the coordinate axes of the graticule on the tracing paper parallel to the axes on the master curve.

FIGURE 2.20 RESISTIVITY SOUNDINGS -SETUL LIMESTONE



NOTES:

ES 13 on the site of Borehole WB1. The resistivity for the first layers and the limestone bedrock is very low, indicating saline water. The borehole EC value was 24,000 micromhos.

ES 9 on the site of Borehole WB2. The low resistivity values are similar to ES 13. The borehole yielded water of EC greater than 6,000 micromhos.

ES 35 prospective borehole site just north of Kangar. Intentions to drill abandoned; even though the curves between AB2 10 to 300 m. indicate solution cavities, the low resistivity indicates saline groundwater.

ES 36 on prospective bore site near Kg. Hujong Bukit. No drilling as slight decrease in gradient beyond AB/2 of 50 m indicates saline water in the limestone.

ES 20 on the site of Borehole PM3. High overburden and bedrock resistivity suggests no salinity problem. Borehole yield had EC of 480 micromhos.

ES 37 on the site of Borehole PM6. No salinity problem expected; bore produced water of EC less than 500 micromhos.

ES 23 on prospective borehole site west of Kg. Penghulu Ja'afar. No salinity problem expected.

FIGURE 2.21 CURVE MATCHING WORKED EXAMPLE: DID ES 38

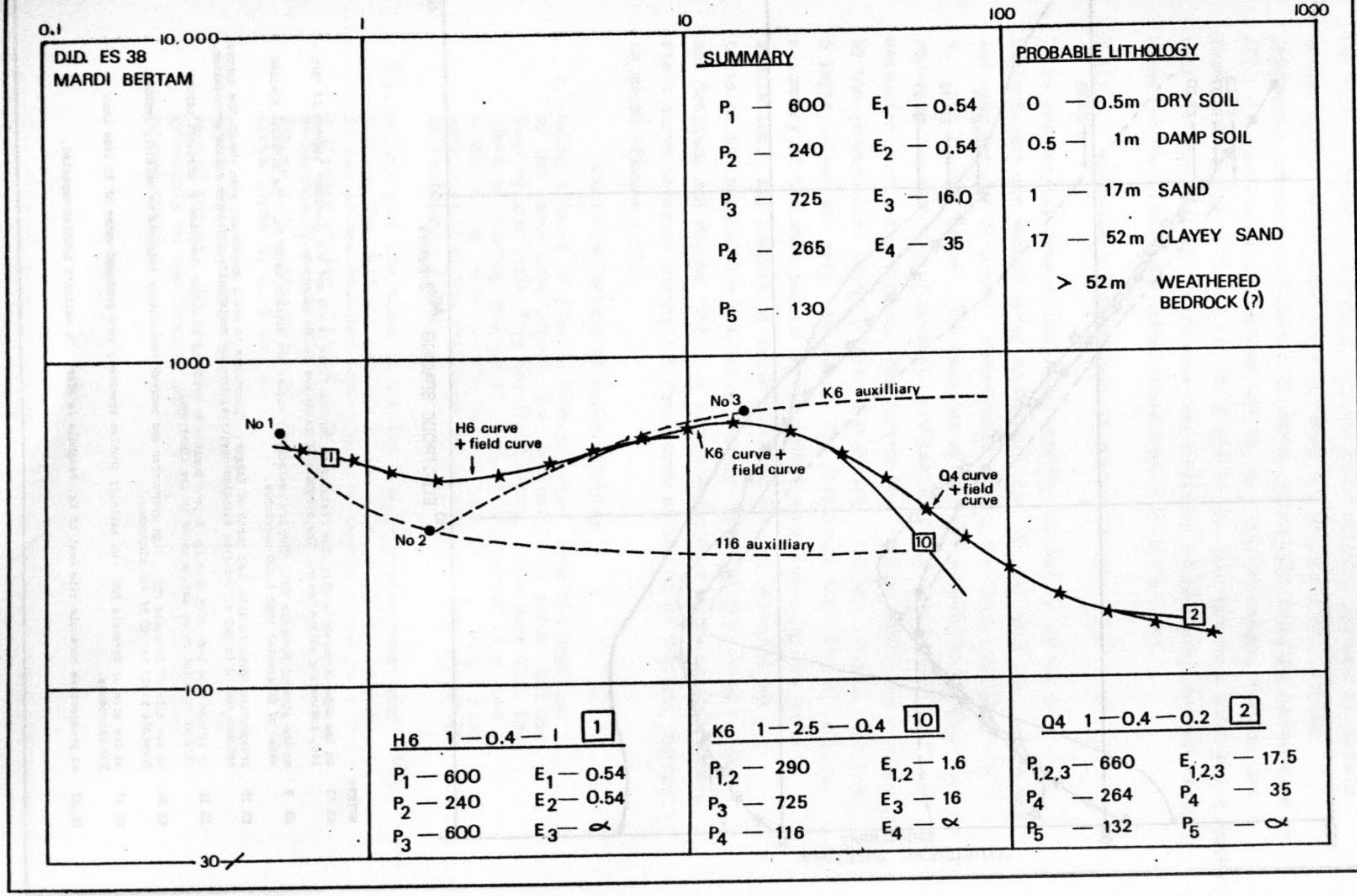
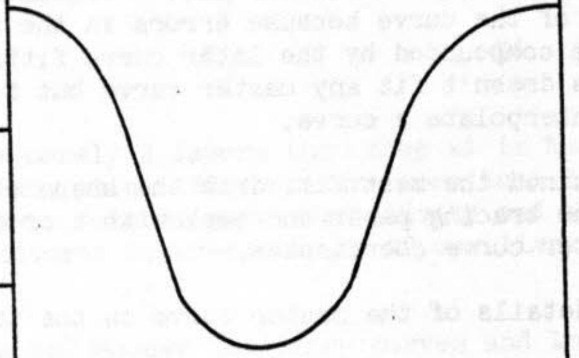
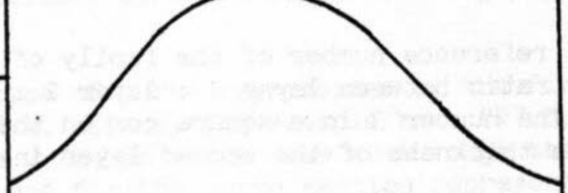
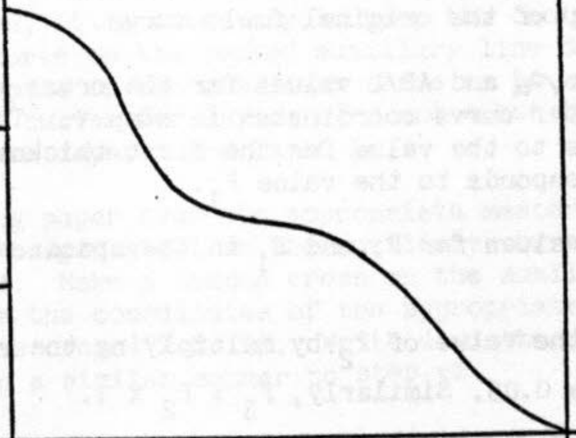
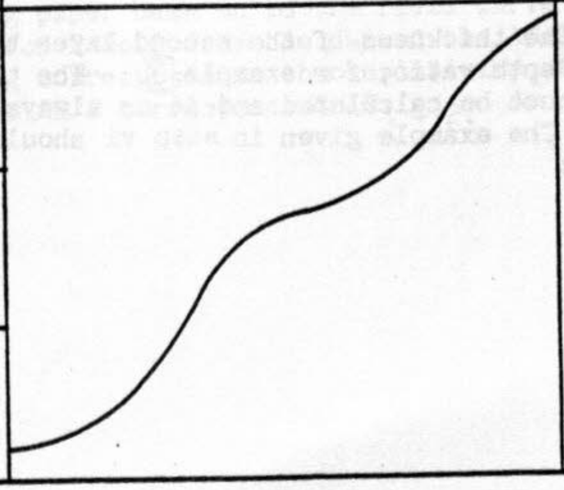


FIGURE 2.22

KEY TO 3 LAYER CURVE PREFIXES

3 LAYER SEQUENCE OF RELATIVE RESISTIVITIES	CORRESPONDING CURVES	CURVE TYPE PREFIX
HIGH LOW HIGH		H — TYPE
LOW HIGH LOW		K — TYPE
HIGH LOW LOWER		Q — TYPE
HIGHER HIGH LOW		A — TYPE

iv. attempt a best fit of the field curve and then try it on another set of curves where the ratio of resistivity values for the three layers is either higher or lower. For example, if the fit seems good on H-20, try H-19 and H-21 as these curves may yield an even better fit. Concentrate on the early part of the curve because errors in the analysis of this section are compounded by the later curve fitting. If the field curve doesn't fit any master curve but falls between two then interpolate a curve.

v. having obtained the best fit, draw the shape of the master curve on the tracing paper and mark with a cross the origin of the master curve coordinates.

vi. write the details of the master curve on the tracing paper as follows:

Example H-20 1:0.05:1 $\boxed{3}$

and below write:

P_1 = E_1 =

P_2 = E_2 =

P_3 = E_3 =

H-20 is the reference number of the family of curves with a resistivity ratio between layer 1 : layer 2 : layer 3 of 1:0.05:1. The number 3 in a square box on the curve is the ratio of the thickness of the second layer in relation to the first.

vii. take the tracing paper off the master curves and place it back on top of the original field curve.

viii. read of the ρa and AB/2 values for the cross made at the origin of the master curve coordinates in step v. The AB/2 value corresponds to the value for the first thickness E_1. The ρa value corresponds to the value P_1.

ix. enter the values for P_1 and E_1 in the spaces shown in step vi.

x. calculate the value of P_2 by multiplying the value of P_1 by, for example 0.05. Similarly, $P_3 = P_2 \times 1$.

xi. calculate the thickness of the second layer by multiplying E_1 by the depth ratio, for example $\boxed{3}$. The thickness of layer 3 cannot be calculated and it is always regarded as infinity. The example given in step vi should be filled in as follows:

H-20 1:0.05:1 $\boxed{3}$

P_1 = 120 E_1 = 0.3

P_2 = 6 E_2 = 0.9

P_3 = 120 E_3 = $\propto$

If the curve is merely 3 layers then step xi is the end of the curve matching analysis. If there are more than 3 layers, auxiliary curves are used to determine the resistivity and thickness of the fourth and subsequent layers; the use of auxiliary curves follows on from step xi.

xii. Locate Orellana and Mooney auxiliary curves and locate the auxiliary curve for master curve H-20. It is not identified by its reference number; instead, it is identified along the right hand axis by the ratio of P_2/P_1. In the previous example this ratio is 0.05.

xiii. take the tracing paper off the field curve sheet and lay it on the auxiliary curve diagram. Place the cross made in step v on the origin of the auxiliary curve coordinates. Keep all the axes parallel and trace with preferably a pecked line the shape of the 0.05 auxiliary curve on to the tracing paper; label the curve.

xiv. decide on the type of curve represented by the field curve for layers 2, 3 and 4. It may for example be a "k" curve as in curve B, Figure 2.22, in which case go through the K-type master curves and try and fit the curve section representing layers 2, 3 and 4.

To fit the curve, it is necessary to keep the coordinate origin of the master curve on the pecked auxiliary line on the tracing paper. The purpose of the auxiliary curve is to allow the interpreter to match curves for layers 2, 3 and 4 but take into account the ratio between layer 1 and layer 2.

xv. move the tracing paper over the appropriate master curves but with the coordinate origin still on the auxiliary curve, until a good fit is achieved. Make a second cross on the auxiliary curve at the point where the coordinates of the appropriate master curve cross; mark this cross "X No.2". Write down the details of the master curve in a similar manner to step vi.

xvi. move the tracing paper back on to the field curve sheet and read off the values for X No.2 from the ρ_a and the AB/2 ordinates. The value for ρ_a corresponds to the resistivity of layers 1 and 2. The value of AB/2 corresponds to E_{1+2}.

Below is an example:

K-19 1-20-1 $\boxed{0.7}$

$P_{1+2} = 38$ $E_{1+2} = 1.1$

$P_3 = 760$ $E_3 = 0.77$

$P_4 = 38$ $E_4 = \propto$

A rough check on the accuracy of the value for P_{1+2} can be made by multiplying the P_1 value in the example in step xi by E_1 and multiplying the value of P_2 by E_2; then adding these two multiples together and dividing them by the value of E_{1+2} given above; the result should be roughly the same as the value for P_{1+2}. If there is a 5th layer, then follow the procedure in step xii to xv for auxiliary curves except in this example use the auxiliary curve for K-19 where $P_2/P_1 = 20$.

Curve matching in this manner can be successful for a field curve with up to 6 layers. However, the accuracy of the values for layers 4, 5 and 6 depend upon the accuracy of the match for the upper layers. At each level of interpretation errors accumulate and therefore, the values derived for layers 5 and 6 are likely to be unreliable. An apparent 8 layer analysis should never be undertaken!

Once values for ρ and E have been derived for each layer, they should be assessed and if the data appears reliable, the interpreter should attempt the simulation of the field curve on a programmable calculator.

2.8.4 Curve Simulation on the Programmable Calculator

The objective of the curve simulation on the programmable calculator is to check the results of the curve matching process described in Section 2.8.3. The check is performed by entering the calculated values of P and E; the machine then generates the form of the resistivity curve that should arise from the input data. Ghosh (1971) published a method of generating resistivity curves for horizontally stratified materials and a Texas Instruments Ti 59 calculator has been programmed to generate curves on the basis of Ghosh's principles.

The programme is written upon magnetic storage cards; the method of programme and data input is shown (Table 2.2). A copy of the print out at the end of the programme is given (Table 2.3). The printer produces AB/2 and ρ_a coordinates three times per log cycle at an AB/2 of 1, 2.15, 4.64, 10 m. This is suitable for matching with the original field curve when the peaks and troughs are not too pronounced. It sometimes leads to ambiguity when there are several thin layers evident in the log cycle AB/2 from 1 to 10 m.

The output data from the printer should be plotted on a photocopy of the original field curve sheet as in this way, the two curves can be compared. If the curve generated by the calculator does not fit the corrected field curve, the interpreter has to decide how to modify his input data model. The decision is not straightforward because of the principle of equivalence. This is that each part of the curve is a representation of a layer with a thickness and a resistivity, in relation to the other layers above or below.

Any adjustments of the input model therefore involve a change in several parameters. The process of adjustment is made by trial and error but below are some crude guidelines:

i. modify one parameter at a time

ii. obtain a good fit with the early data curve first, then proceed to make modifications to fit the remaining curve

iii. if you want to move the generated curve upwards, this can be done by increasing either the thickness of the layer or the resistivity. To reduce a section of the curve, reduce either the thickness or the resistivity.

iv. remember that the alteration of the shape of the curve in one section will cause a change in the shape of the curve on both sides.

v. whilst gaining experience with curve matching on the calculator, it is worthwhile making several attempts to get the curve to match. However once the interpreter is familiar with the procedure, he should try and restrict himself to getting the best match possible within 5 attempts.

Table 2.2 INPUT OF PROGRAM AND DATA - T1 59

Step No.	Procedure	Enter	Display
1.	Press (3) (2nd) (Op) (17)		719.29
2.	Press (1) Read Card Side 1		1
3.	" (2) " " " 2		2
4.	" (3) " " " 3		3
5.	Key Reference Abscissa	(105) (STO) (23)	105
6.	Key Apparent Resistivity of Half Space	(ρa lowest layer) (A)	
	Prints ρa 'Rows' = ρ Half Space		
7.	R? Appears on Printer	ρ layer	(R/S)
8.	T? On Printer - Enter thickness t of next layer*	t' layer	(R/S)
9.	Repeat Steps 7) and (8) till all layers entered then when R? Appears on printer; press (2nd) 'A' - Outputs AB/2. Sample points and calculated ρa.		

Note: * computation for each layer lasts about 95 seconds.

t is the equivalent of E in the master curve matching terminology.

Table 2.3

Print Out For Sounding Curve Simulation - T1 59

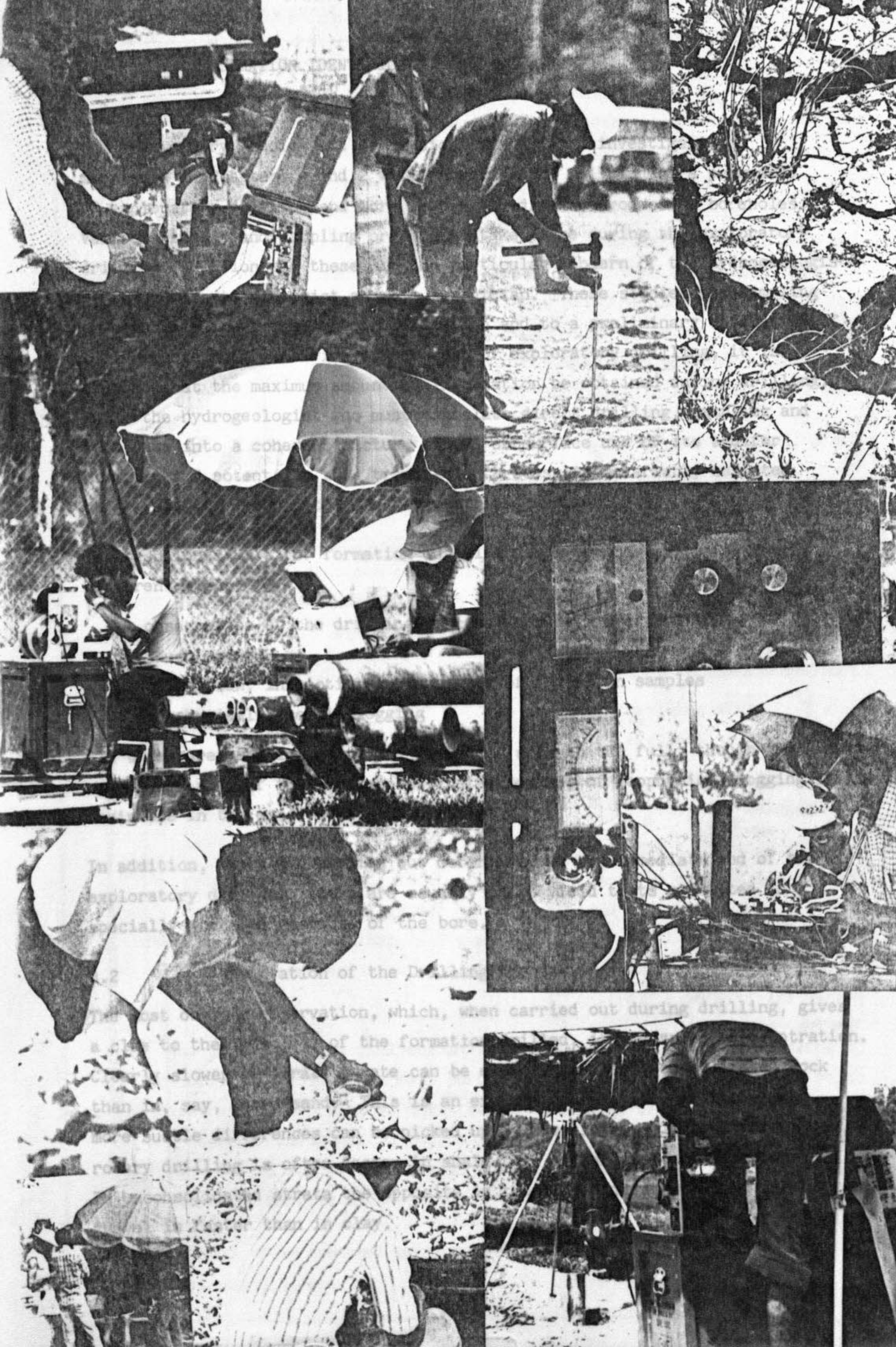
```
    210082.00                                    1.00
        60.00                                  734.93
         2.00
                                                 2.15
    1 - 145 - 0.13                             774.07
    2 - 5800 - 0.26
    3 -  26  - 18                                4.64
    4 - 20,000                                 297.46

     20000.00          DHS                      10.00
                        R?                      38.08
        26.00          RHD
                        T?                      21.54
        18.00           T                       38.55
                        R?
      5800.00          RHD                      46.42
                        T?                      70.86
         0.26           T
                        R?                     100.00
       145.00          RHD                     153.05
                        T?
         0.13           T                      215.44
                        R?                     323.69

                                               464.16
                                               675.83

                                              1000.00
                                              1383.97
```

The reason for this is the time factor involved. It takes at least 10 minutes to enter data for a 6 layer model. It takes a further 5-10 minutes to plot the new curve and analyse the results. Therefore only 3 to 4 runs can be completed in 1 hour.

2.9 Presentation of Results

Results can be presented in a written or diagramatic form which can draw together geophysical findings with geological and hydrogeological data to give a credible description of earth layer characteristics and configuration. How the geophysicist reports his results depends on who will later use these results. The latter will often be used by someone from outside the field of geophysics and in this case, there is little point in confusing him with a recital of calculated resistivity values. The geophysicist should therefore try and translate his analysis into the units and terminology appropriate to the discipline of the recipient. The geophysicist should always stress to others that his equipment merely enables him to measure values of subsurface resistivities; interpretation and the translation of resistivity values into rock types invariably depends on the availability of existing geological and drilling data. It follows that resistivity can seldom be useful without control from existing boreholes.

During 1981-1982 several reports have been written on the results of resistivity work in different areas. A review of those reports will show that there is no rigid format as objectives, methods and available subsurface control have varied in each case. These reports are assembled, together with susbsequent bore drilling and testing data, in the Area Investigations volume.



3. FORMATION IDENTIFICATION AND TESTING

3.1 General

The preliminary studies and geophysical resistivity investigations mentioned in Chapters 1 and 2 will invariably be followed by a programme of exploratory drilling and bore testing leading to production boreholes. Various studies and sampling procedures take place during the exploratory drilling operation and these are the particular concern of the hydrogeologist or the bore site geologist and his technician. These studies and sampling procedures allow formation identification and to a preliminary estimation of aquifer potential. In view of the cost of exploratory drilling, it is crucial that the maximum amount of information be obtained during drilling. It is the hydrogeologist who must translate direct drilling, sampling and test data into a coherent picture of the subsurface and of its aquifer geometry and potential; this translation process is invariably assisted by the surface geophysical studies described earlier.

The techniques used for formation identification are basically of three different kinds:

- observation of the drilling process; rotary air or mud flush and DHH
- collection, inspection and analysis of formation samples
- geophysical borehole logging

All three are very important and should be used to the full, though the expense of some of the more sophisticated methods of geophysical logging limits their use in the water well industry.

In addition, tests are carried out during and at the immediate end of the exploratory drilling; these are usually short yield tests conducted in specially isolated sections of the bore.

3.2 Observation of the Drilling Process

The most obvious observation, which, when carried out during drilling, gives a clue to the lithology of the formation drilled, is the speed of penetration. Clearly slower penetration rate can be expected in hard, consolidated rock than in, say, loose sand. This is an extreme example but, in fact, much more subtle differences can be picked up. Thus, in consolidated formations, rotary drilling is often faster in shale than in sandstone or limestone. In unconsolidated strata the opposite is true and drilling in sand and/or gravel is faster than in clay.

There are no absolute rules on the speed of penetration of different materials, as it would obviously vary with different equipment, drilling method, bit size and geometry and penetrated depth. However, an accurate penetration log can be most helpful in accurate delineation and identification of the drilled strata and such logs should always be kept. The normal practice in the JPT drilling section is to note the penetration achieved per unit time. The time or depth units are chosen on the basis of local experience; clearly a longer time/depth unit is appropriate for hard rocks than for unconsolidated formation. A minute per metre penetration scale has been found satisfactory. The penetration rate log is normally recorded on the composite bore log; an example from bore BG 2, Bukit Gantang, Perak is given (Figure 3.1a) from a bore drilled by rotary and mud flush.

The following characteristics are indicated:

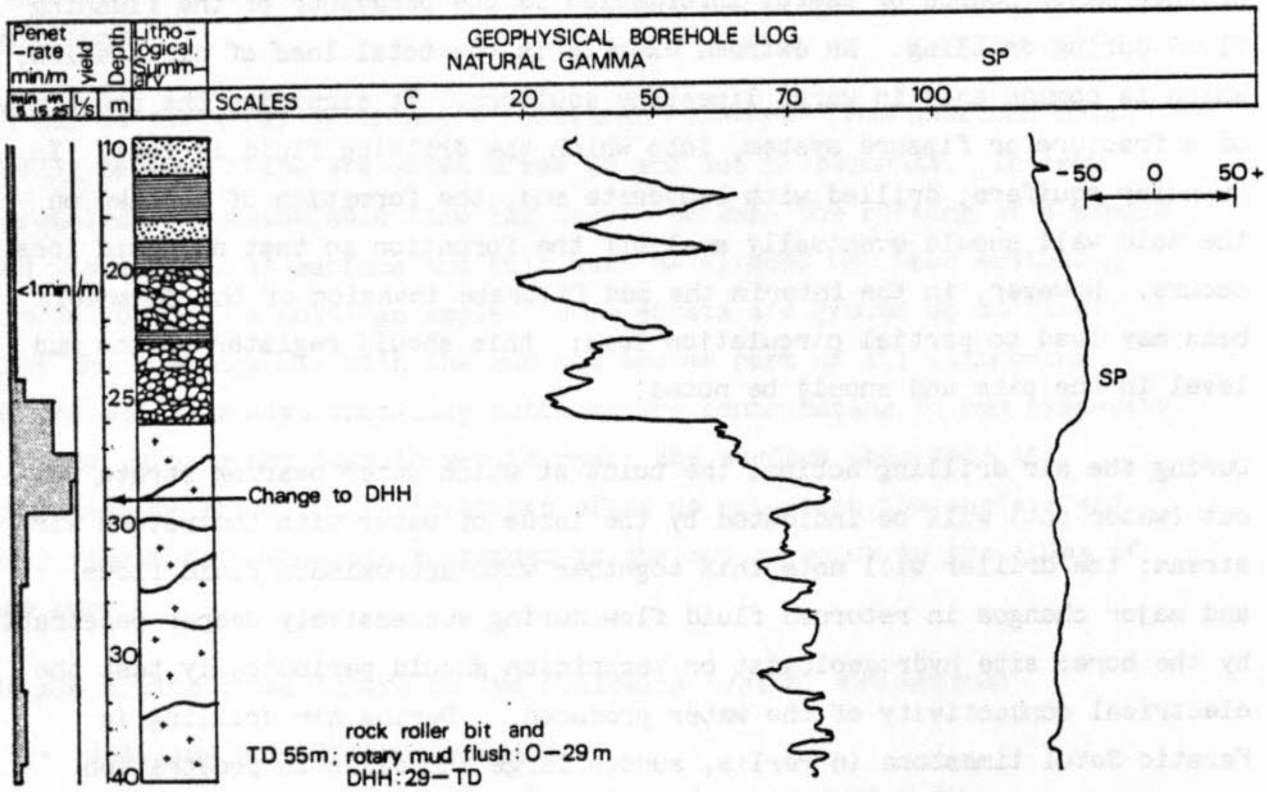
- slight but significant decreases in penetration rate opposite shaly sections; where the bit tooth conformation is wrong, shales and clays are extremely difficult to drill because of bit balling and blockage. It is sometimes possible, in thin sand-shale sections, to accurately correlate small penetration rate variations with gamma logs and caliper logs.
- overall 2-6 min/m penetration rate in eluvial sands and clays
- 5-15 min/m penetration rates, typical of weathered bedrocks in Peninsular Malaysia.
- a fairly sudden increase in this rate to 30-50 + min/m in fresh granite bedrock.

Drilling method and tools can markedly change penetration rate and the hydrogeologist must maintain close liaison with the driller in order to correctly interpret rate changes shown on the drillers log. Another example, from bore A1, Kg. Alma, Seberang Prai indicates how penetration changes in a hard rock when the DHH method is used instead of a rock roller bit (Figure 3.1(b)).

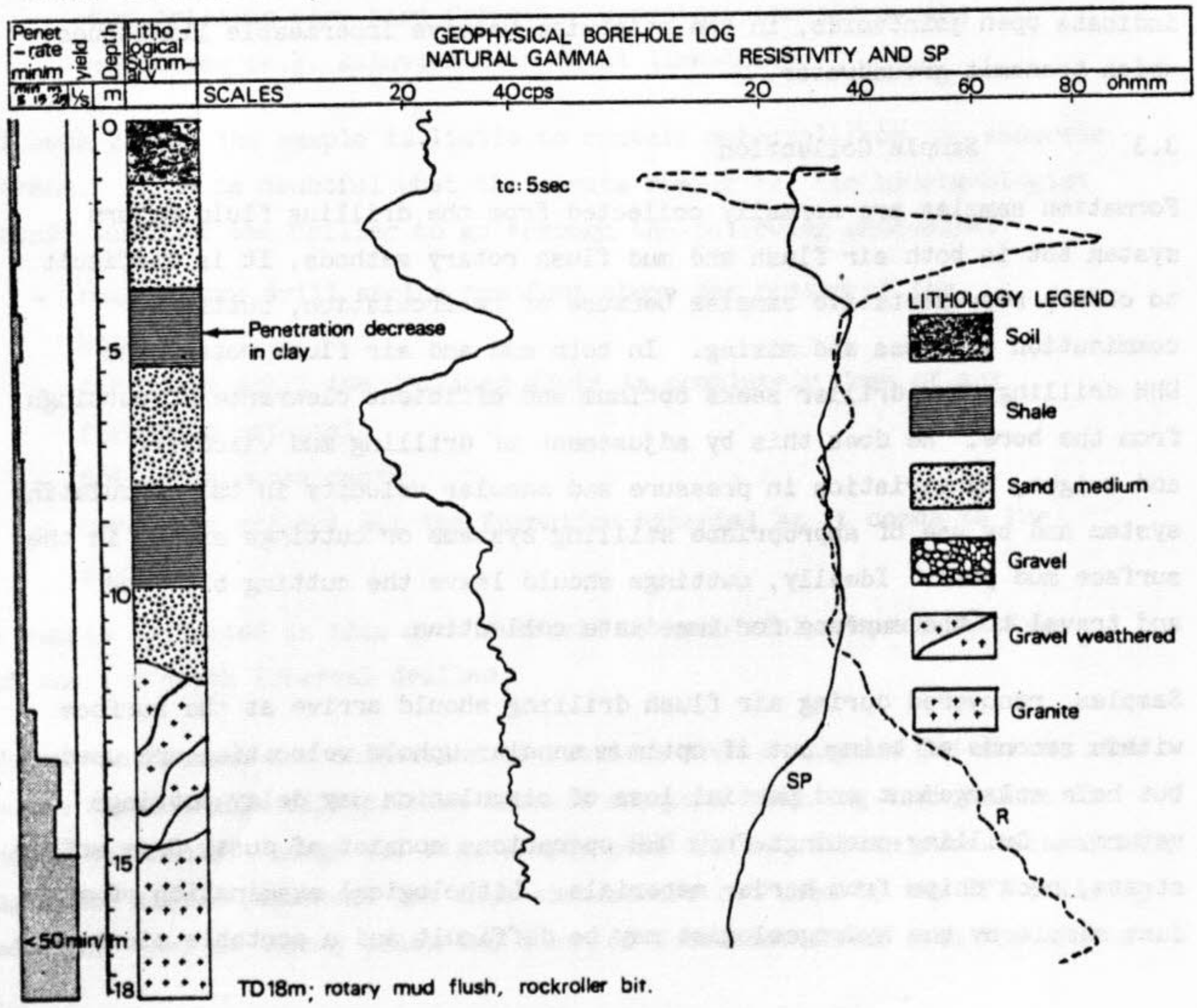
In addition to keeping an accurate penetration log, a good driller can often obtain valuable relevant information from the general behaviour of the drill, particularly in shallow holes. For example, in unconsolidated formations drilled with a drag bit, the action of the drill is much smoother in clays than in sands and coarse gravels; in the latter, vibrations of the drill string can often be actually seen at the surface.

FIGURE 3.1 EXAMPLES OF COMPOSITE BORE LOGS

A. ALMA A1 COMPOSITE BORE LOG EXTRACT



B. BUKIT GANTANG BG 2: COMPOSITE BORE LOG EXTRACT



Note: Bore construction detailsomitted.

Another major source of useful information is the behaviour of the flushing fluid during drilling. An extreme example is the total loss of circulation, which is common only in karst limestone aquifers. It signifies the penetration of a fracture or fissure system, into which the drilling fluid is lost. In granular aquifers, drilled with bentonite mud, the formation of mudcake on the hole wall should eventually seal off the formation so that no fluid loss occurs. However, in the interim the mud filtrate invasion of the permeable beds may lead to partial circulation loss; this should register in the mud level in the pits and should be noted.

During the air drilling action, the point at which water bearing strata are cut (water cut) will be indicated by the issue of water with the return air stream; the driller will note this together with approximate fluid flows and major changes in returned fluid flow during successively deeper penetration by the bore; site hydrogeologist or technician should periodically test the electrical conductivity of the water produced. During air drilling in Karstic Setul limestone in Perlis, sudden large increases in penetration rate (or sometimes a sudden short drop of the drill string) have been associated with the sudden appearance of water in the bore; these events indicate open jointvoids, in the otherwise massive impermeable limestones, which transmit groundwater.

3.3 Sample Collection

Formation samples are normally collected from the drilling fluid return system but in both air flush and mud flush rotary methods, it is difficult to obtain representative samples because of recirculation, cuttings, comminution and loss and mixing. In both mud and air flush rotary and DHH drilling, the driller seeks optimum and efficient clearance of cuttings from the bore. He does this by adjustment of drilling mud viscosity and weight, by variation in pressure and annular velocity in the circulating system and by use of appropriate stilling systems or cuttings sieves in the surface mud pits. Ideally, cuttings should leave the cutting bit face and travel to the surface for immediate collection.

Samples recovered during air flush drilling should arrive at the surface within seconds of being cut if optimum annular uphole velocities are used but hole enlargement and partial loss of circulation may delay cuttings return. Drilling cuttings from DHH operations consist of dust, from softer strata, rock chips from harder materials. Lithological examination of a dust sample by the hydrogeologist may be difficult and a portable microscope may be needed; the driller should be asked where possible, to obtain larger chippings.

It may be difficult to interpret cuttings recovered from mudflush holes mainly because these are often mixed up and out of sequence. In deep boreholes, a considerable time lag occurs between the cutting of a sample and its arrival at surface and this must be allowed for when assigning a true depth to a cuttings ample. Some strata are ground up so finely that the cuttings mix with the mud and become part of it; thickening of the mud is a sign that clay cuttings are contributing to mud viscosity. A large cutting may take longer to reach the surface than does its immediate neighbour whilst cuttings often do not reach the surface and will remain down-the-hole suspended in the mud or stuck to the sides of the bore.

Sample mixing often occurs in the following typical situations:

- drilling thin bedded deposits,
- when penetrating formation boundries (e.g. alluvium to weathered bedrock)
- when drilling vary hard formation beneath a heavily caving overburden (e.g. alluvium over Setul limestone)

In such cases, the sample is liable to contain material from two separate strata. If he is doubtful what the strata really is, the hydrogeologist should instruct the driller to go through the following procedure:

- suspend the drill string one foot above the bottom of the hole
- circulate until the drilling fluid is completely free of any formation material
- drill one metre depth
- carefully collect all the formation material as it comes to the surface.

A sample collected in this manner should be completely representative of the 1 m depth interval drilled.

Prompt settlement and collection of cuttings from a viscous mud stream also poses considerable difficulties because adequate stilling tank volume is never available. Large oil rigs simply pass the mud stream over mechanically agitated sieves (shakers) but such devices are not used by the JPT drilling section; they do anyway allow the loss of formation fines from the sample which in alluvial sand aquifers, is undesirable. In most water bore drilling, the formation cuttings are allowed to settle in some kind of a sampling box (or sampling pit) inserted into the mud return channel. Inevitably, in drilling unconsolidated formations with a thick and viscous mud, there is some separation of the drilled material into finer and coarser fractions while travelling upwards with the mud column. Further, the samples are contaminated with the mud and have to be washed clean with water at the surface. It is not possible to do that without, at the same time, washing away some of the finer constituents of the formation. Therefore, often the sampled formation appears coarser than it really is. Sampling methods thought appropriate to the JPT Drilling Section are as follows:

- collect samples 'little and often', for each metre drilled
- hold a very fine sieve (200 mesh) under the mud return flow pipe for a few seconds at a time and constantly empty contents into a bucket
- use several buckets to catch the sample and mud together and allow settlement; dilute if mud is very viscous
- trap cuttings in a long baffled trough set into the mud channel system.

During the drilling of exploratory boreholes into eluvial and alluvial sediments at Cangkat Jong and Bukit Gantang, Perak, the hydrogeological section demonstrated a considerable loss of silt-fine sand fraction from the samples. Samples from the mud return, described as coarse rather clean 1-2 mm sands, were subsequently shown by careful sampling and coring, to be non-representative. True formation was an ill sorted and immature m-c sand with a high kaolin clay-silt fraction.

In routine sampling, drill cuttings are normally collected at 1 metre depth intervals and at every change of formation (both in direct and reverse circulation drilling). The samples should be carefully labelled and laid out in special boxes for inspection and geological description. Samples should be placed in the sample box according to some fixed sequence; left to right from the top downwards and so on, book fashion is suggested. They should then be packed in separate plastic bags (again clearly labelled) for transport to a laboratory or office.

Undisturbed formation samples or cores have been taken on several occasions by the JPT drilling section, to obtain a true and uncontaminated sample of a formation whose exact nature could not be resolved by inspection of cuttings from the mud stream. Whilst cores can indicate true formation lithology, grain size and composition, they tend to suffer compression and distortion during the coring and core extraction process; this means that permeability values derived from core permeability tests may differ markedly from permeabilities derived from bore pumping tests.

Good core recovery can be expected from coring consolidated strata or clays but recovery is usually poor in the non-cohesive, unconsolidated sands and gravels which form good aquifers; rather such materials can only be sampled by use of a bailer and line. The fact that the coring conducted in possible aquifer zones at Bukit Gantang and Cankat Jong in Perak was mainly successful in core recovery indicates (even before core extrusion and examination) that the material sampled was of little potential as an aquifer; such good core recovery indicated a cohesive material rich in kaolinitic clay-silt.

3.4. Sample Description and Analysis

3.4.1 Lithologic Description

All formation samples collected should be described on site by a competent technician, hydrogeologist or a geologist. In each case, the following characteristics should be described and recorded on a descriptive lithological log:

Hard Rock

- Rock types in sample, by <u>percentage</u>. Describe each type with respect to the following characteristics
- major mineral constituents (quartz, felpar etc.), accessory minerals (mica, limonite etc). Use of 0.1 N hydrochloric acid, in field dropper bottle, allows identification of lime (calcium carbonate) or dolomite in the sample; hot HCL fizzes dolomite.
- colour, texture, hardness - Fracture cleavage or break pattern of rock fragments as this could effect rock porosity.
- other observations; limonitised fracture planes in limestones as indicators of groundwater flow, size of rock cuttings and whether abraded.

Soft Rock - Alluvium

- in addition to the above, soft, granular unconsolidated sediments should be described in terms of grain composition, grain size, degree of sorting (or grading), degree of roundness and angularity of individual grains, the presence of accessory or matrix material (clay, limonite) upon the grain surface
- where granular aggregates occur in the sample (i.e. sand grains inconpletely broken by the drill action) record any intergranular cement or matrix.

Much of the necessary description is qualitative and subjective, particularly the individual's perception of percentages, degree of sorting and angularity. Text books on the petrology of sedimentary and meta-igneous rocks contain numerous rock classifications to which the hydrogeologist must refer. Classifications of angularity and roundness are found in good text books (.e.g Pettijohn, P.F; Sedimentary Rocks) whilst the American Institute of Petroleum Geologists has published standard colour charts.

For accurate description, samples must be examined with a x 8 or x 10 hand lens or, preferably, with a low power binocular microscope. Whatever methods are adopted, the end result should be a description, of the whole sample, which is concise and understandable to workers not directly involved in the same investigation. The description of sample, in terms of component percentage, is then used in conjunction with geophysical borehole logs and information acquired from observation of the drilling process, to give an interpretative geological log which, in the JPT, is plotted on a common scale, with geophysical logs, penetration rate and bore construction details, upon a composite bore log. Examples are given elsewhere (Figure 3.1, 4.7).

The distinction should be made between the two types of lithologic description used by the hydrogeologist, namely:

- the lithologic description of the whole sample recovered from the drill. This description should be recorded in percentage terms wihtout interpretation or judgement
- the interpreted lithologic log which appears in the composite bore log. This log is the hydrogeologists opinion of the true nature of the subsurface strata and is based both on the lithologic percentage description, the geophysical bore logs and drill evidence.

3.4.2 Sample Analysis

The most important analysis of cuttings samples of loose granular material is grain size analysis, ideally done by dry sieving 0.5 to 1 Kg of sample through a number of standard sieves with different sizes of openings. The results are plotted on to graph paper, of linear or log linear type, in terms of cumulative percentage of sample passing the standard size openings used. The standard graph as used by JPT is given (Figure 3.2) together with some typical grain size curves. Results are plotted in this way to gain a statistical picture of grain size distribution, which is relevant to the water bearing properties of the formation and to borehole design.

With the help of such graphs, a granular formation can be described in terms of grading, sorting and grain size distribution by use of the following parameters:

Median Grain Size

- that size which divides the sample into two equal parts, one containing all larger grains, the other containing all smaller grains. Also known as the 50% size or D_{50}.

Effective Grain Size

- sometimes used in technical literature for 10% size (D_{10}); that is, that size which 90% of the sample is larger than.

Upper and Lower Quartiles

- 75% and 25% sizes (D_{75} and D_{25}), respectively.

Uniformity Coefficient

- The ratio of 60% size to 10% size (D_{60}/D_{10}); a measure of the variety of grain sizes in a sample. Sometimes given as the ratio D_{90}/D_{10}.

Coefficient of Sorting

- Defined as the square root of the ratio of the upper to lower quartiles (D_{75}/D_{25}); again a measure of the variety of grain sizes in a sample.

The grain size classification used by the JPT is also shown on Figure 3.2.

The degree of sorting of a sample is commonly described in terms of the uniformity coefficient as follows:

FIGURE 3.2 EXAMPLE OF PARTICLE SIZE DISTRIBUTION GRAPH

Borehole location : BUKIT GANTANG

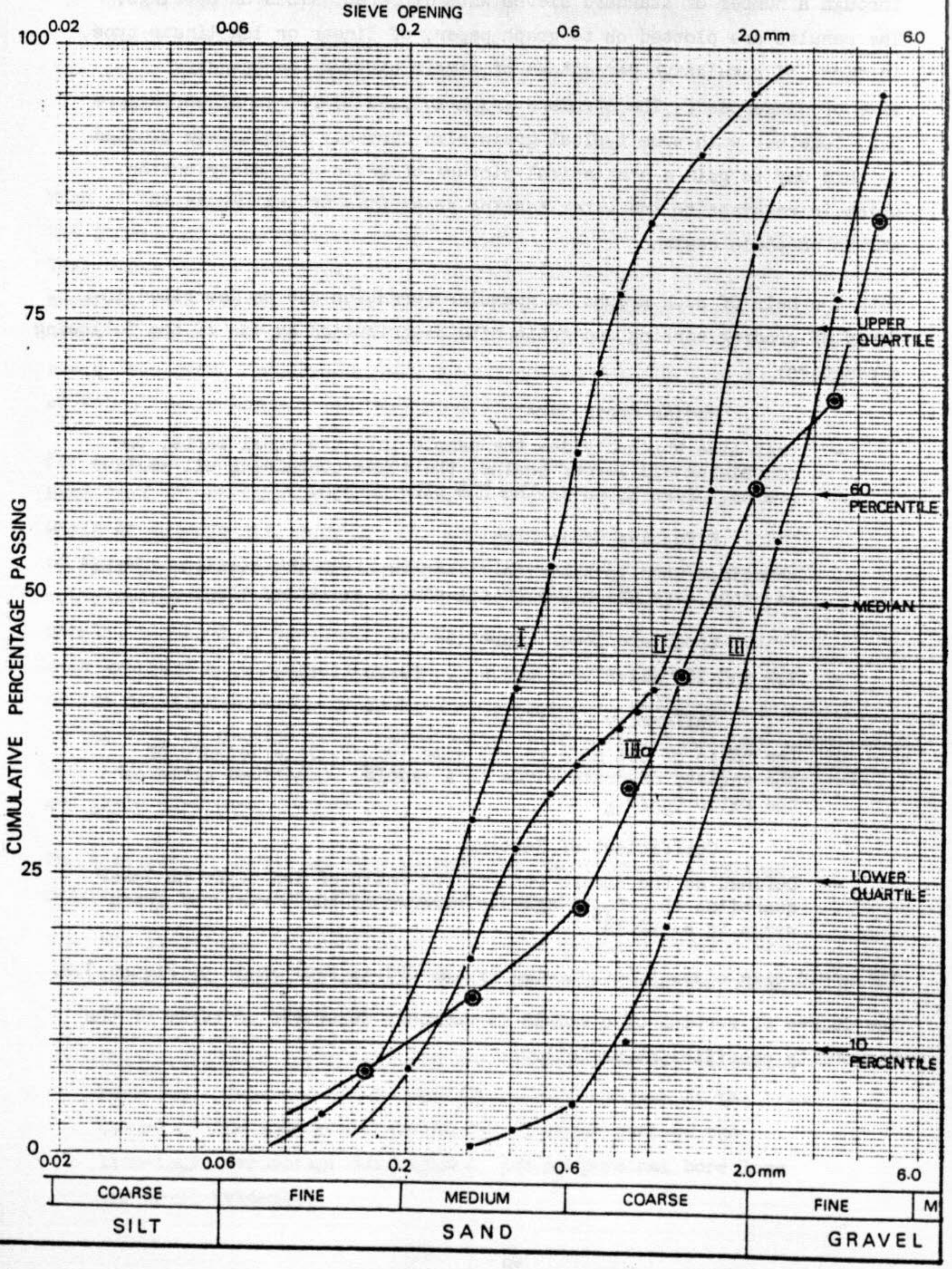
Borehole number : BG IA (Curves III, IIIa only)

Total depth :

Screened interval :

Casing :

Sample no.	Depth interval (m)	D_{60}/D_{10} Uniformity coefficient
I	EXAMPLE ONLY	3.35
II		6.7
III	12.20m	3.27
IIIa	17.1–17.3(CORE)	10.5



Uniformity Coefficient	Description
< 2	Well sorted
2 to 4	Moderately well sorted
4 to 8	Poorly sorted
> 8	Very poorly sorted

The main practical use of accurate grain size data is for borehole design, specifically for the design of artificial gravel packs and screen slot sizes (Chapter 5). However, aquifer grain size is also related to permeability; the simplest relation, claimed by some writers, simply states that permeability is proportional to the effective grain size. Another simple formula, after Hazen, says:

$$(D_{10})^2 = K \text{ cm/sec}$$

Where D_{10} is the 10 percentile size
and K is permeability.

At best, such formulae give only a very approximate of true permeability; values so derived often differ considerably from pump test derived permeabilities.

Most formation grain size distribution curves are single S-shape (plot I in Figure 3.2). However, in some cases the plots come out as a double S-shaped curve (plot II in Figure 3.2): this is rare and in such cases it is probable that the sample represents a mixture of two different formations. Curves I and III indicate moderately well sorted medium and coarse sands (UC's of 3.35 and 3.27 respectively). Extreme loss of the fine sand-silt fraction from the drill samples in BG 1A has made curve III an erroneous representation of true grain size and sorting. Subsequent core analysis indicated a very poorly sorted argillaceous sand (Curve IIIa).

The use of grain size curves in borehole design is shown elsewhere (Figure 5.7).

3.5 Geophysical Logging

3.5.1 Introduction

Electrical and radioactive properties of geological strata are related to formation lithology and groundwater quality. These properties can be measured by techniques known as electric and radioactivity logging. The basic equipment, used for such logging, consists of a power source, a reel of armoured co-axial cable to which a probe is attached, a measuring circuit, including amplifiers, and a recorder capable of continuous recording. The probe is lowered into a borehole and signals from it are amplified and recorded at the surface.

The JPT groundwater section geophysically logs exploratory boreholes on a routine basis using SIE T450-E logging instruments; the equipment allows caliper, natural gamma, spontaneous potential-resistivity (SP-R or electric logs) and temperature logs to be run in bores up to 450 m depth. The logs obtained are used together with sample lithologic description and drilling data, to make proper formation identifications and to judge formation boundaries, screenable zones and transmissive cracks in hard rock aquifers. All information including the logs is incorporated in composite bore logs; some examples are shown elsewhere (Figures 3.1, 3.8 and 3.10).

The JPT has logged over 50 bores and the majority of the logged section has been in largely air or DHH drilled bores in fresh and deeply weathered hard rocks, such as metamorphosed shaly-quartzites (e.g. Singa and Semanggol Formations in Kedah-Perlis), hard limestones (e.g. Setul limestone in Perlis) and granites. The caliper or section gauge log has been the more useful log in these hard rocks since it directly identifies rock cracks or fissures which constitute the transmission zones in the hard rock aquifer. Gamma and electric log response in hard rocks is often poor; these logs have given better results in mud drilled alluvial or alluvial materials where the log can be used to delineate sand and clay beds.

3.5.2 Electric Logging

A normal electric log consists of a spontaneous potential (SP) curve and one or more resistivity curves; the JPT instrument will run only one resistivity curve, the point resistivity (PR). Schematic circuits for SP and PR logging are shown (Figure 3.3). For electric logging, the drilled bore has to be uncased and has to contain electrically conductive fluid; the fluid will commonly be a bentonite-water mixture in rotary mud flush drilling. However, much of the JPT drilling work has been by the rotary air or water flush or air operated down-hole hammer (DHH) methods which means that logs have often had to be run in a bore filled with water (usually formation water of EC less than 400 micromho). In such bores, electric logs of rather poor quality are obtained. Electric (SP) logs can be run in cased bores where they are used to make a log of casing collars or threads.

FIGURE 3.3 SCHEMATIC CIRCUITS FOR MEASURING SP AND PR

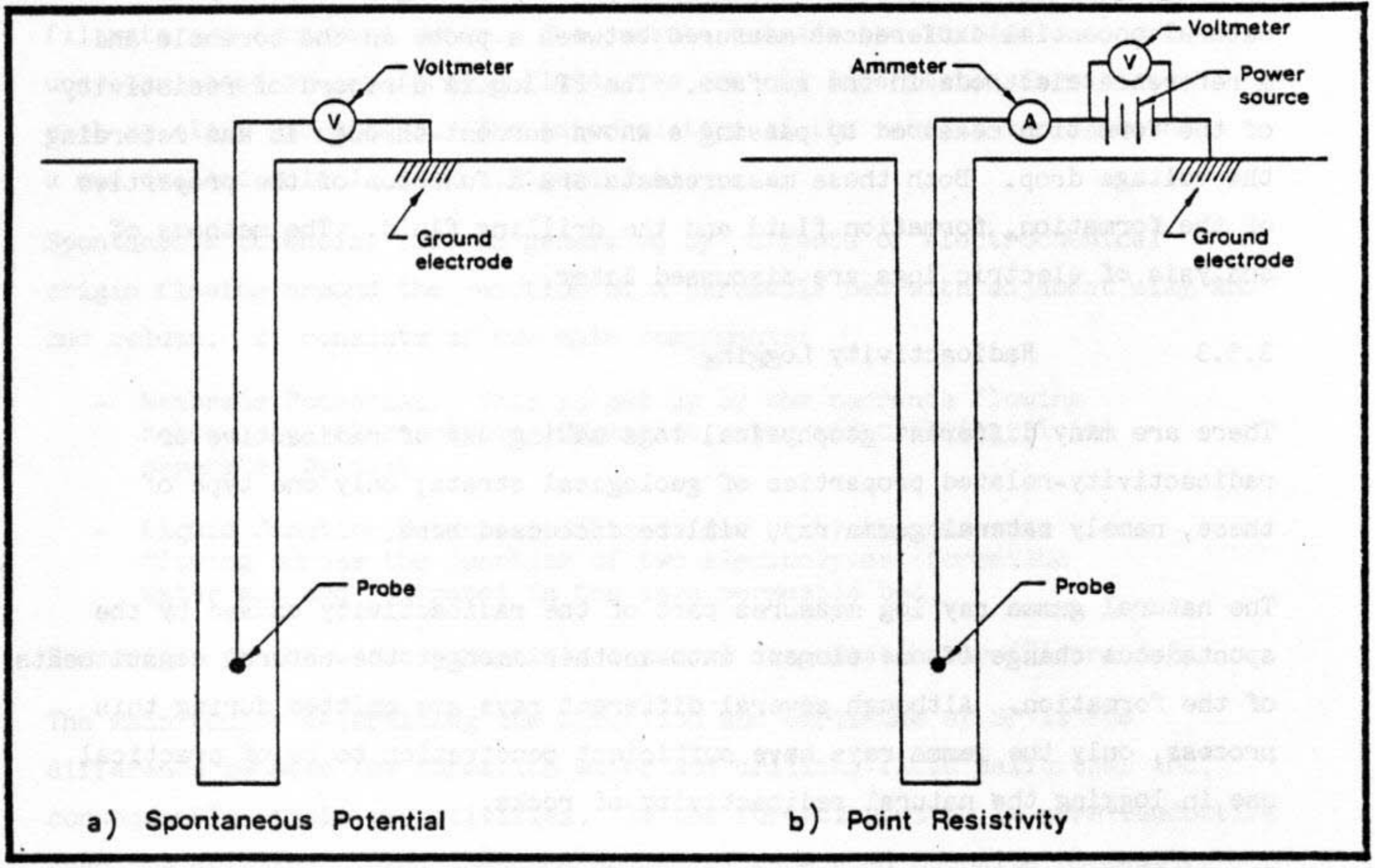
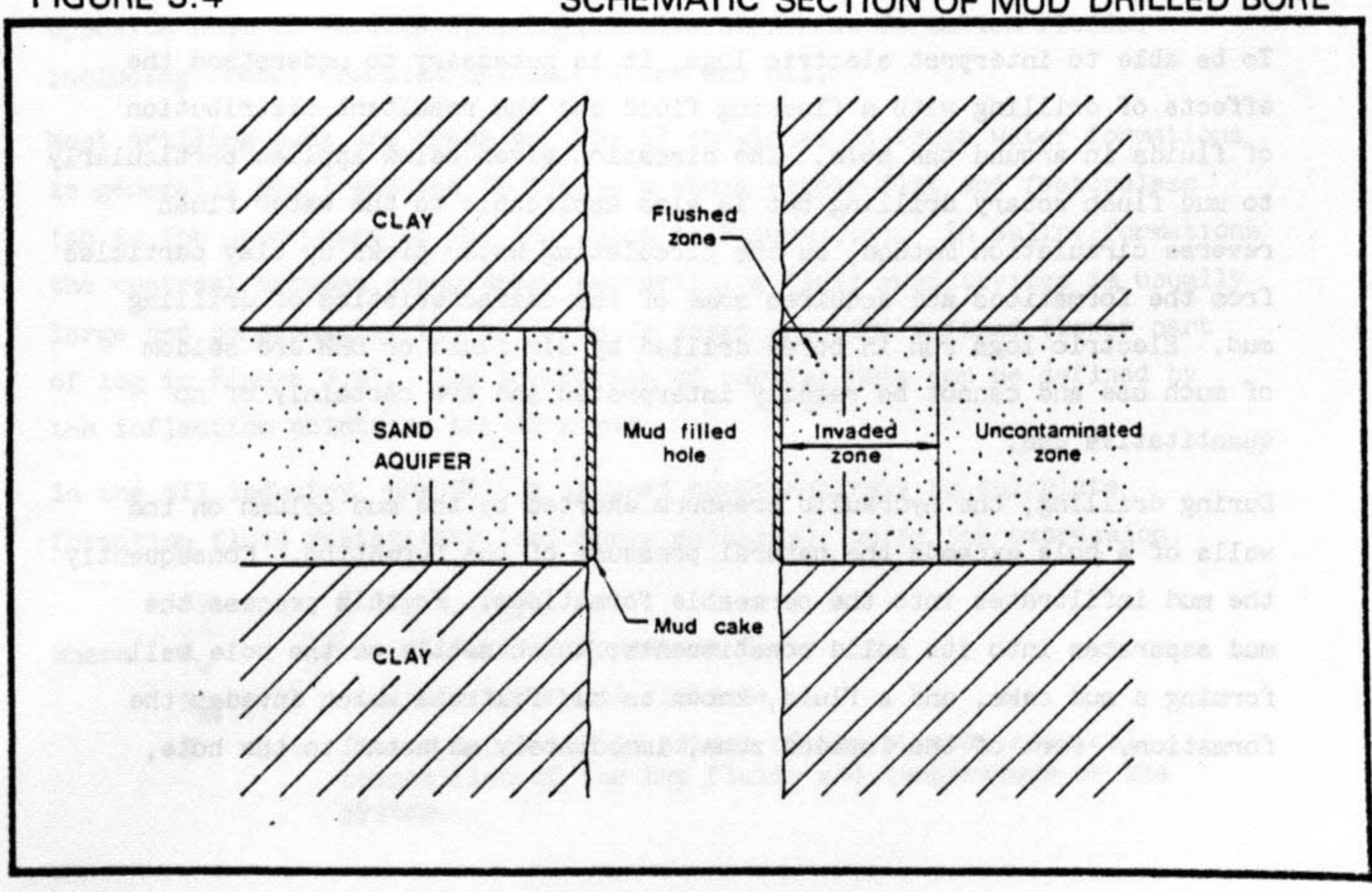


FIGURE 3.4 SCHEMATIC SECTION OF MUD DRILLED BORE



Sp and PR logs are recorded simultaneously. The SP log is a record of natural potential differences measured between a probe in the borehole and a reference electrode in the surface. The PT log is a record of resistivity of the formation measured by passing a known current through it and recording the voltage drop. Both these measurements are a function of the properties of the formation, formation fluid and the drilling fluid. The methods of analysis of electric logs are discussed later.

3.5.3 Radioactivity Logging

There are many different geophysical logs making use of radioactive or radioactivity-related properties of geological strata; only one type of these, namely natural gamma ray, will be discussed here.

The natural gamma ray log measures part of the radioactivity caused by the spontaneous change of one element into another amongst the natural constituents of the formation. Although several different rays are emitted during this process, only the gamma rays have sufficient penetration to be of practical use in logging the natural radioactivity of rocks.

In contrast to electric logging, gamma ray measurements do not require a bore full of a conductive fluid and can be successfully used in boreholes lined with casing of steel or any other material.

3.5.4 Interpretation of Electric Logs

To be able to interpret electric logs, it is necessary to understand the effects of drilling with a flushing fluid and the resultant distribution of fluids in around the hole. The discssion given below applies particularly to mud flush rotary drilling but is also applicable to the water flush reverse circulation method, as the circulating water picks up clay particles from the formations and acquires some of the characteristics of drilling mud. Electric logs run in bores drilled by air flush or DHH are seldom of much use and cannot be readily interpreted and are certainly of no quantitative use.

During drilling, the hydraulic pressure exerted by the mud column on the walls of a hole exceeds the natural pressure of the formation. Consequently the mud infiltrates into the permeable formations. In this process the mud separates into its solid constituents, which settle on the hole wall forming a mud cake, and a fluid, known as mud filtrate which invades the formation. Part of the invaded zone, immediately adjacent to the hole, is completely flushed out by mud filtrate. The rest contains a mixture of filtrate and formation water. The strata outside the invaded zone are uncontaminated by any foreign fluids, as are all impermeable formations such as clays and shales. The interrelation of the various zones around a mud drilled hole is shown (Figure 3.4).

Spontaneous potential (SP) is generated by currents of electrochemical origin flowing around the junction of a permeable bed with adjacent clay and mud column. It consists of two main components:

- Membrane Potential: This is set up by the currents flowing between two electrolytes (formation water and drilling fluid) separated by clay

- Liquid Junction Potential: This is set up by the currents flowing across the junction of two electrolytes (formation water and mud filtrate) in the same permeable bed.

The origin of these two potentials is shown diagramatically (Figure 3.5).

The main factor determining the direction and amplitude of SP is the difference between the formation water and drilling fluid salinities and, consequently, their resistivities. If the formation water is more conductive than the drilling fluid, then the potential generated opposite permeable beds is negative (that is, shows a shift to the left with conventional logging polarity). Similar beds, containing water less conductive than the drilling mud, usually (but not always) generate a positive potential anomaly. Figure 3.6 shows an idealised electric log with SP and PR which might be expected opposite beds of various lithologies with different formation fluids, including fresh, brackish and salt water and oil.

Most drilling muds are fresh and the SP developed in fresh water formations is generally small and the SP log as a whole rather flat and featureless (as in the upper part of the log shown in Figure 3.6). In saline formations the contrast between groundwater and drilling fluid resistivites is usually large and consequently the SP curve is sharp and well defined (lower part of log in Figure 3.6). The boundaries of various beds can be defined by the inflection points on the SP curve.

In the oil industry, the SP log is used quantitatively to calculate formation fluid resistivity (and hence salinity), using the expression:

$$SP = K \log R_w/R_{mf}$$

where R_w = resistivity of formation fluid

R_{mf} = resistivity of mud filtrate

K = factor depending on nature of the strata, chemical composition of the two fluids and temperature of the system.

FIGURE 3.5 ORIGIN OF SPONTANEOUS POTENTIAL CURRENTS

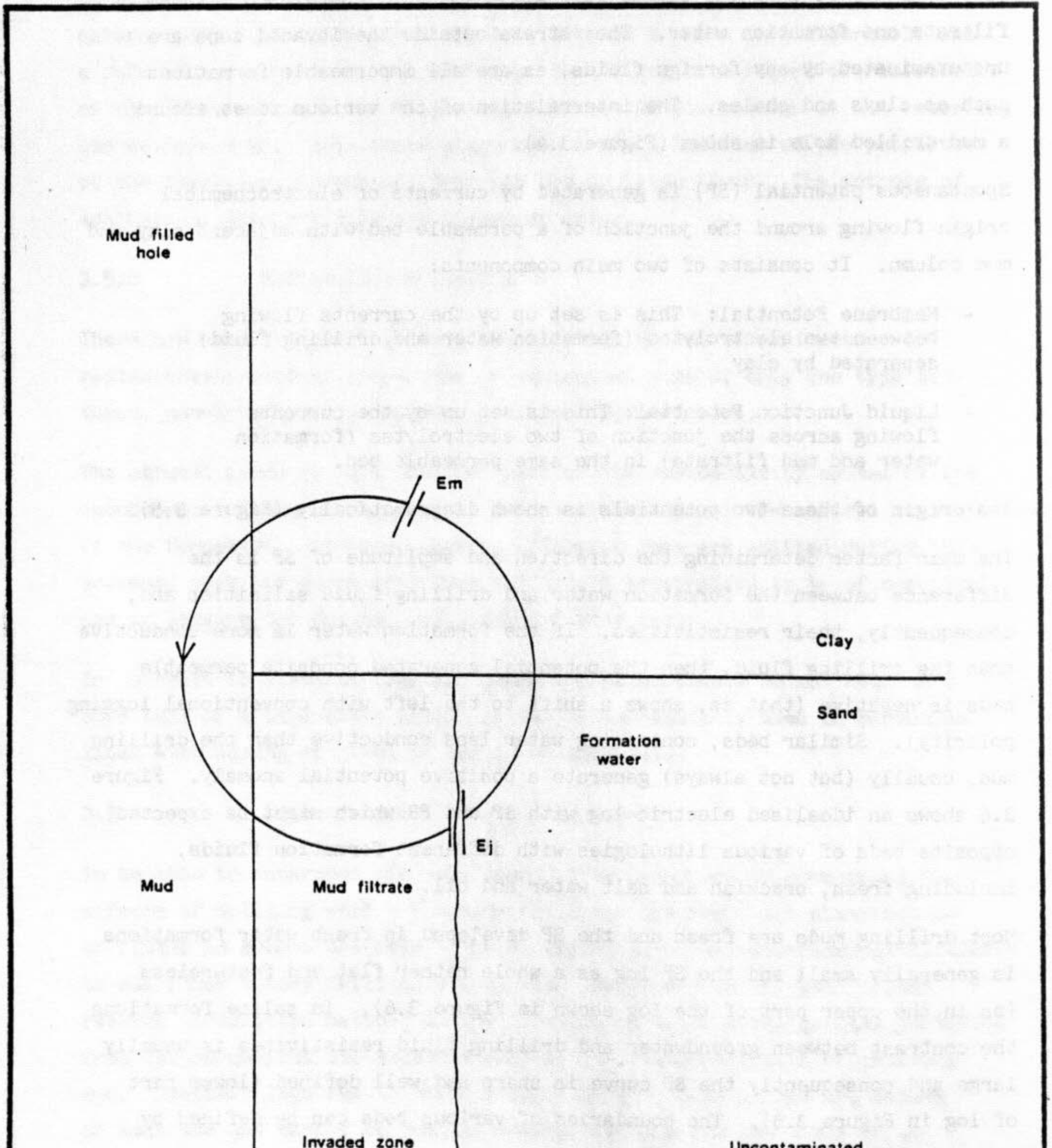
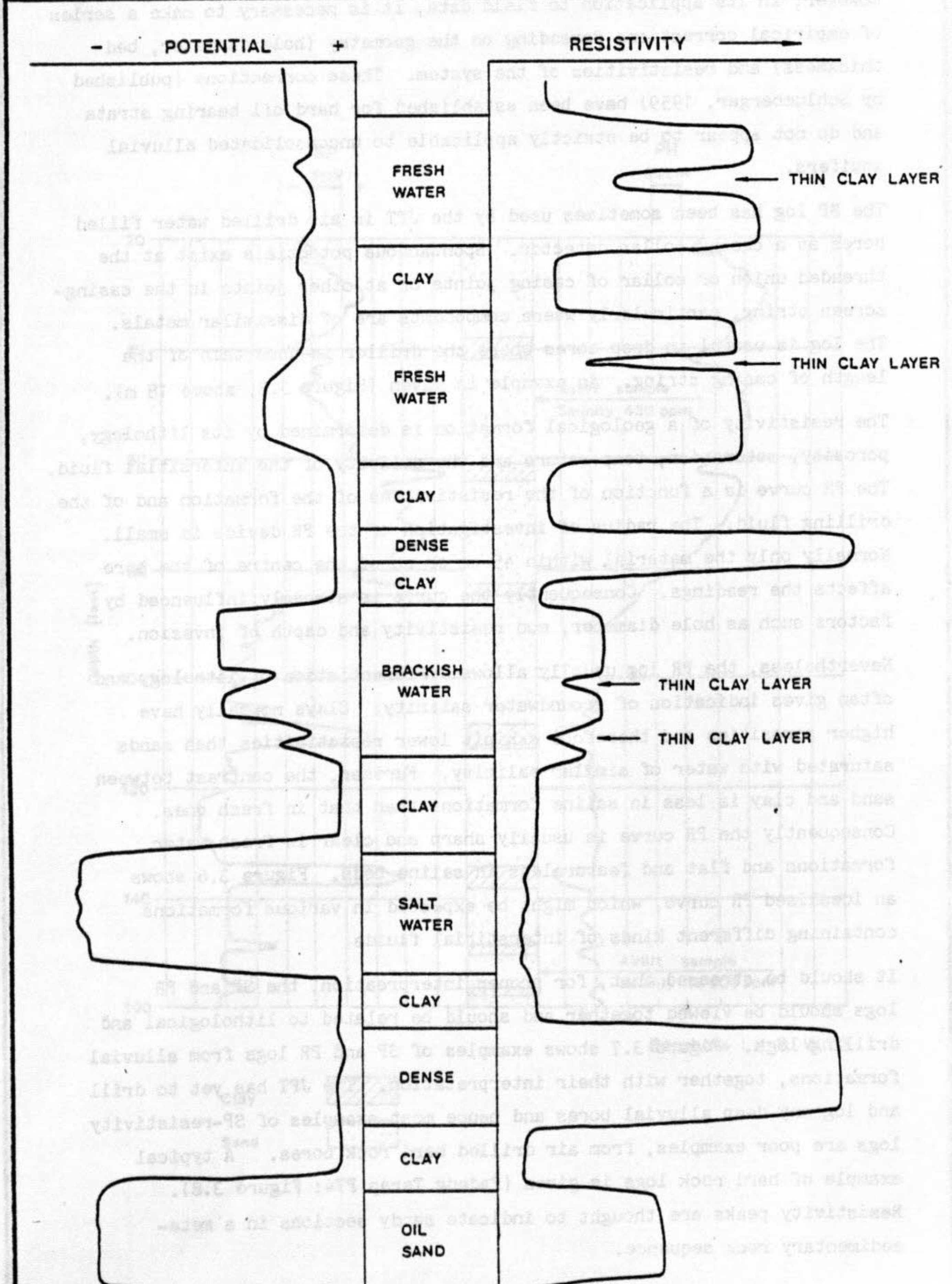


FIGURE 3.6 IDEALISED ELECTRIC LOG



The above equation is approximate but it has been rigorously derived. However, in its application to field data, it is necessary to make a series of empirical corrections depending on the geometry (hole diameter, bed thickness) and resistivities of the system. These corrections (published by Schlumberger, 1959) have been established for hard oil bearing strata and do not appear to be strictly applicable to unconsolidated alluvial aquifers.

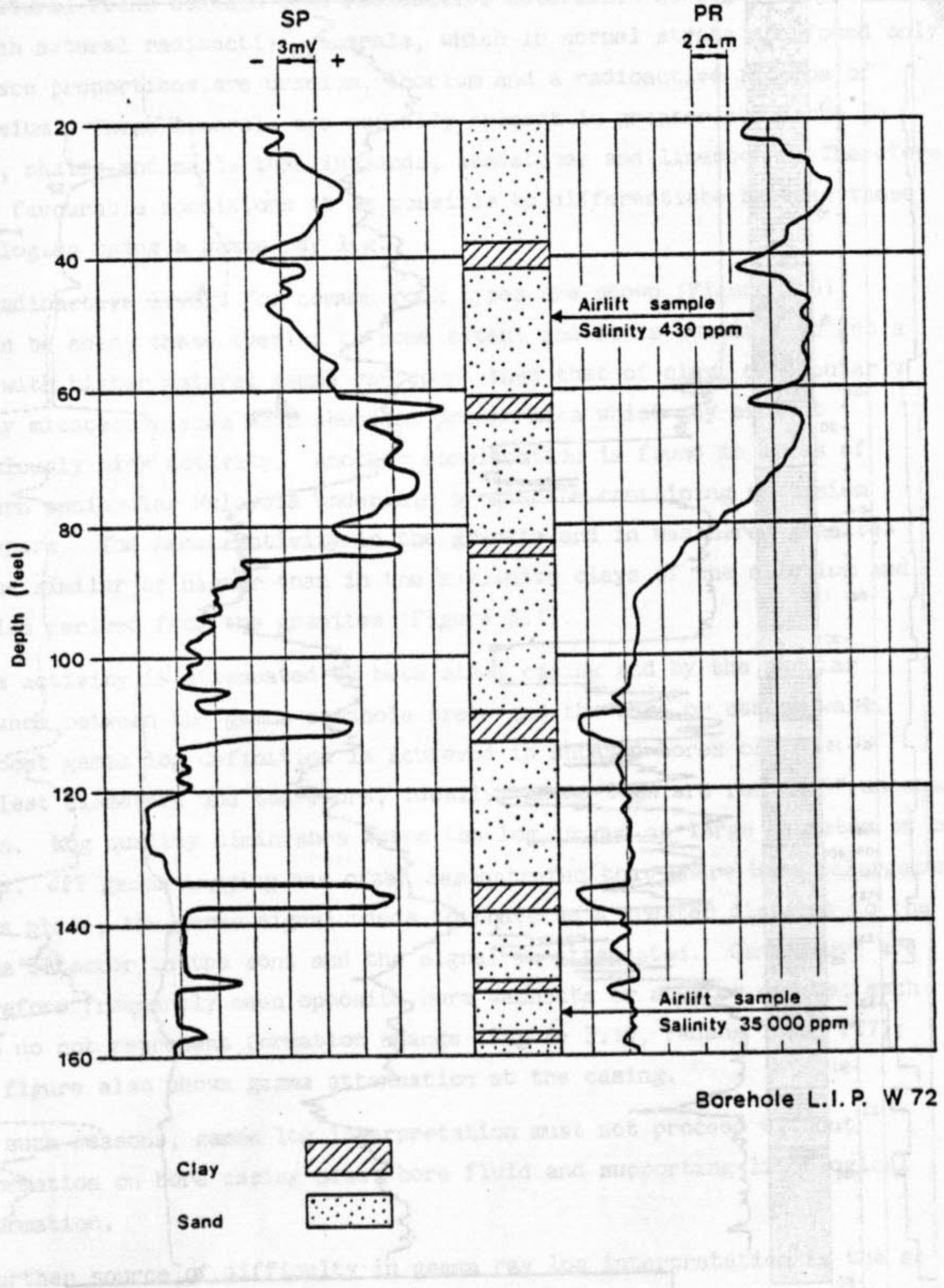
The SP log has been sometimes used by the JPT in air drilled water filled bores as a casing collar detector. Spontaneous potentials exist at the threaded union or collar of casing joints or at other joints in the casing-screen string, particularly where components are of dissimilar metals. The log is useful in deep bores where the driller is uncertain of the length of casing string. An example is given (Figure 3.8, above 18 m).

The resistivity of a geological formation is determined by its lithology, porosity, saturation, temperature and the salinity of the intersitial fluid. The PR curve is a function of the resistivities of the formation and of the drilling fluid. The radius of investigation of the PR device is small. Normally only the material within 45 cm or so of the centre of the bore affects the readings. Consequently the curve is strongly influenced by factors such as hole diameter, mud resistivity and depth of invasion.

Nevertheless, the PR log usually allows differentiation of lithology and often gives indication of groundwater salinity. Clays normally have higher porosities and therefore exhibit lower resistivities than sands saturated with water of similar salinity. Further, the contrast between sand and clay is less in saline formations than that in fresh ones. Consequently the PR curve is usually sharp and clear in fresh water formations and flat and featureless in saline beds. Figure 3.6 shows an idealised PR curve, which might be expected in various formations containing different kinds of interstitial fluids.

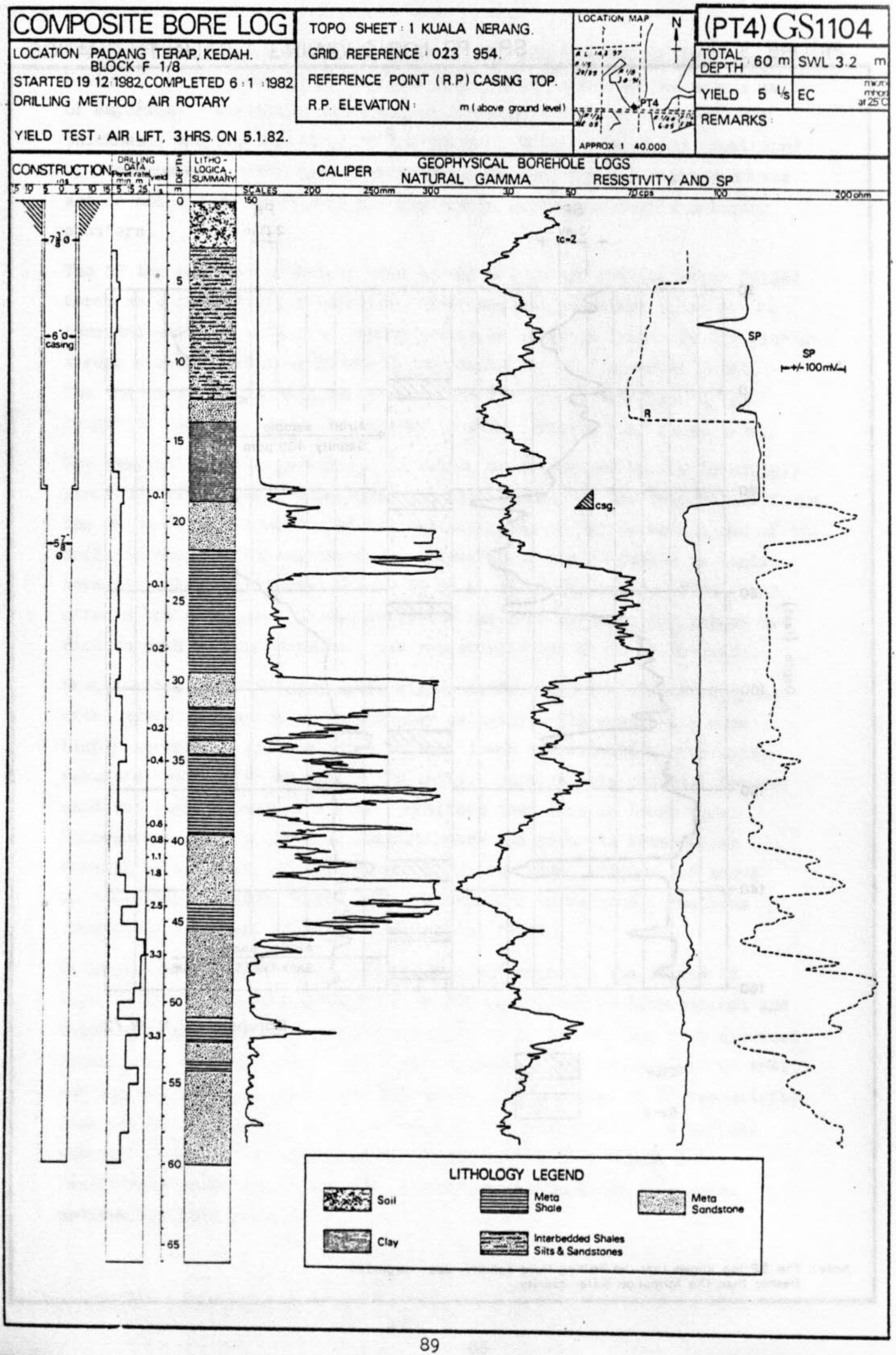
It should be stressed that, for proper interpreation, the SP and PR logs should be viewed together and should be related to lithological and drilling logs. Figure 3.7 shows examples of SP and PR logs from alluvial formations, together with their interpretation. The JPT has yet to drill and log any deep alluvial bores and hence most examples of SP-resistivity logs are poor examples, from air drilled hard rock bores. A typical example of hard rock logs is given (Padang Terap PT4; Figure 3.8). Resistivity peaks are thought to indicate sandy sections in a meta-sedimentary rock sequence.

FIGURE 3.7 SP – PR LOG SHOWING A SALINITY CHANGE



Note: The SP log shows that the drilling fluid salinity was marginally fresher than the formation water salinity.

FIGURE 3.8 LOG EXAMPLES.



COMPOSITE BORE LOG

LOCATION PADANG TERAP, KEDAH. BLOCK F 1/8
STARTED 19 12 1982, COMPLETED 6 1 1982
DRILLING METHOD AIR ROTARY
YIELD TEST AIR LIFT, 3 HRS. ON 5.1.82.

TOPO SHEET 1 KUALA NERANG.
GRID REFERENCE 023 954.
REFERENCE POINT (R.P) CASING TOP.
R.P. ELEVATION: m (above ground level)

(PT4) GS1104
TOTAL DEPTH 60 m
SWL 3.2 m
YIELD 5 l/s
EC micro mhos at 25°C
REMARKS:

The SP log, above 19 m, is acting as a casing collar log. In an earlier example, (Figure 3.1), the resistivity curve increases through the weathered zone and into a fresh granite bedrock.

3.5.5 Interpretation of Gamma Ray Logs

All natural rocks contain some radioactive material. However, the levels of such natural radioactive minerals, which in normal strata are found only in trace proportions,are uranium, thorium and a radioactive isotope of potassium. These minerals are normally present in greater abundance in clays, shales and marls than in sands, sandstones and limestones. Therefore, under favourable conditions it is possible to differentiate between these lithologies using a gamma ray log.

The radioactive levels for common rock types are shown (Figure 3.9). As can be seen, these overlap to some extent and it is possible to get a sand with higher natural gamma ray count than that of clay, particularly highly micaceous sands with abundant potash mica which may exhibit anomalously high activity. Another complication is found in areas of eastern peninsular Malaysia underlain by granite containing potassium feldspars. The gamma activity in the granite and in weathered granites may be similar or higher than in the kaolinite clays of the alluvium and eluvium derived from the granites (Figure 3.1).

Gamma activity is attenuated by both steel casing and by the annular distance between the gamma downhole probe and the bore or casing wall. The best gamma log definition is achieved in uncased bores of the smallest diameters and therefore, ideally, gamma logs are run in 6" uncased bores. Log quality diminishes where the log is run in large diameter or cased bores. JPT gamma logging has often demonstrated that where bore enlargement takes place, the gamma signal needs to traverse a greater distance to the gamma detector in the sond and the signal is attenuated. Gamma lows are therefore frequently seen opposite bore washouts or aquifer cracks; such lows do not represent formation change (Figure 3.10, Padang Terap PT7); the figure also shows gamma attenuation at the casing.

For such reasons, gamma log interpretation must not proceed without information on bore casing size, bore fluid and supporting lithological information.

A further source of difficulty in gamma ray log interpretation is the so called statistical variation or statistical noise. The intensity of radiation, which affects the logging instrument is not constant, but fluctuates due to the random number of atomic distintegrations at any given instant.

FIGURE 3.8 LOG EXAMPLES.

FIGURE 3.9 ROCK RADIOACTIVITY

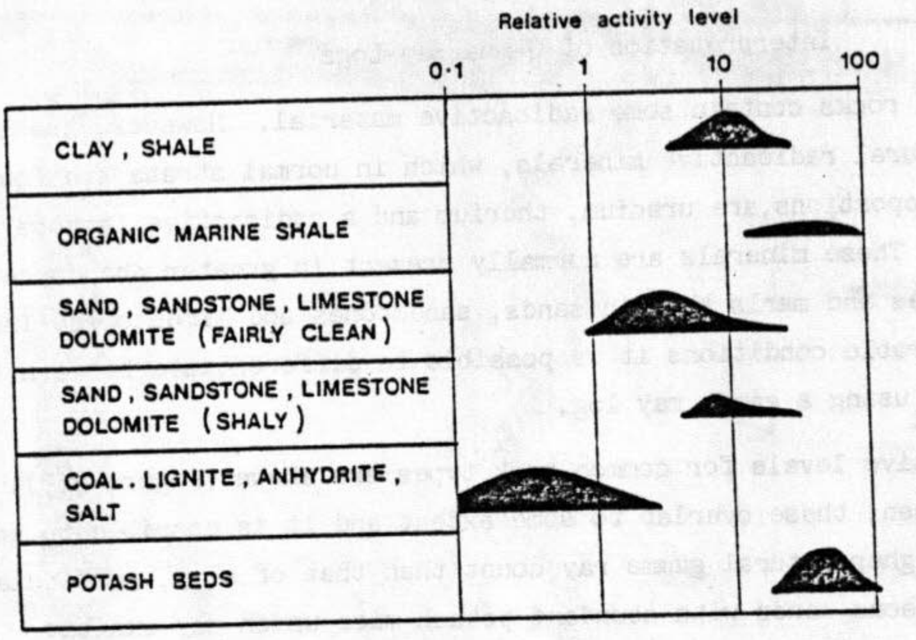


FIGURE 3.11 GAMMA LOGS RUN AT DIFFERENT SPEEDS AND TIME CONSTANTS

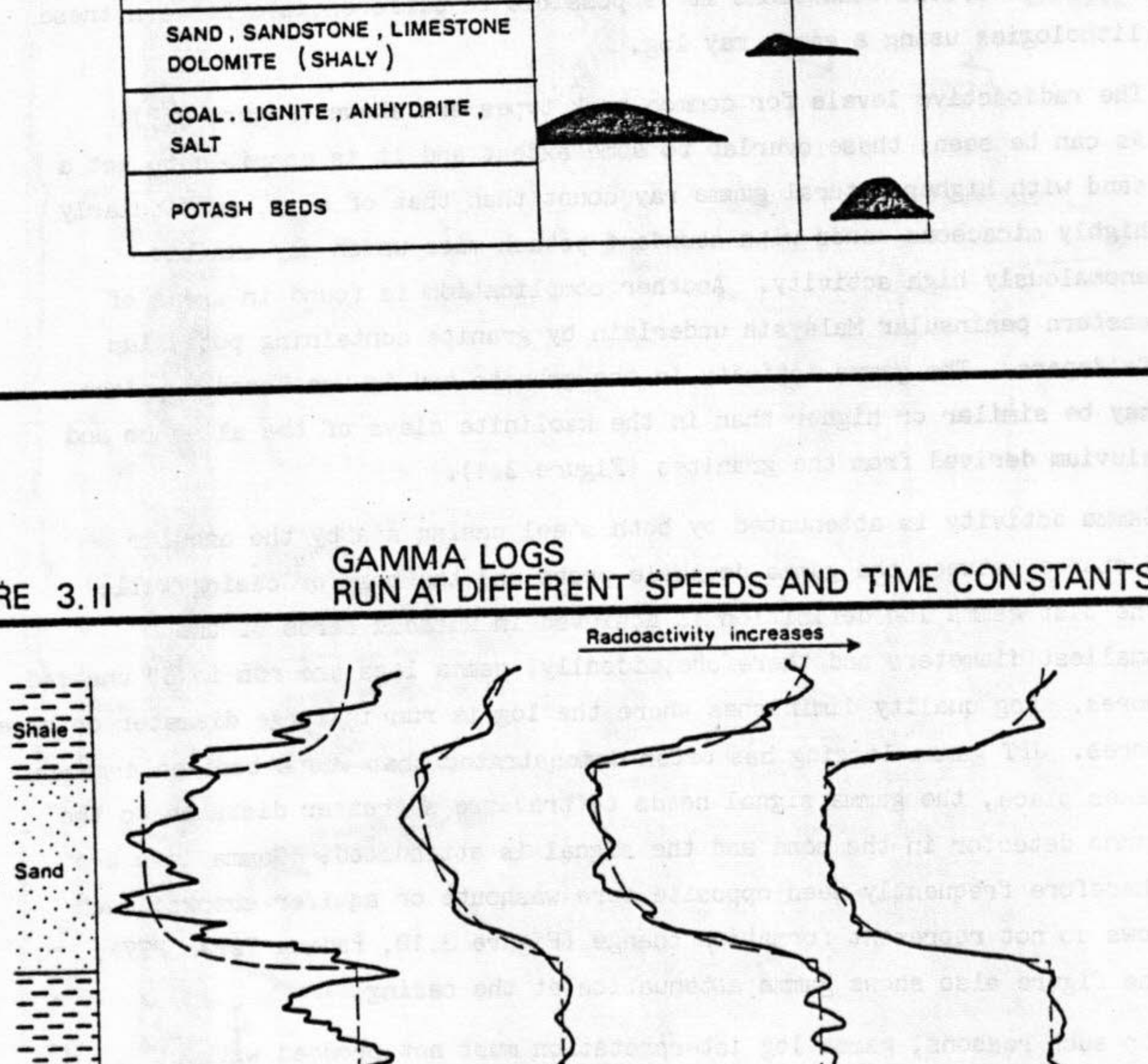
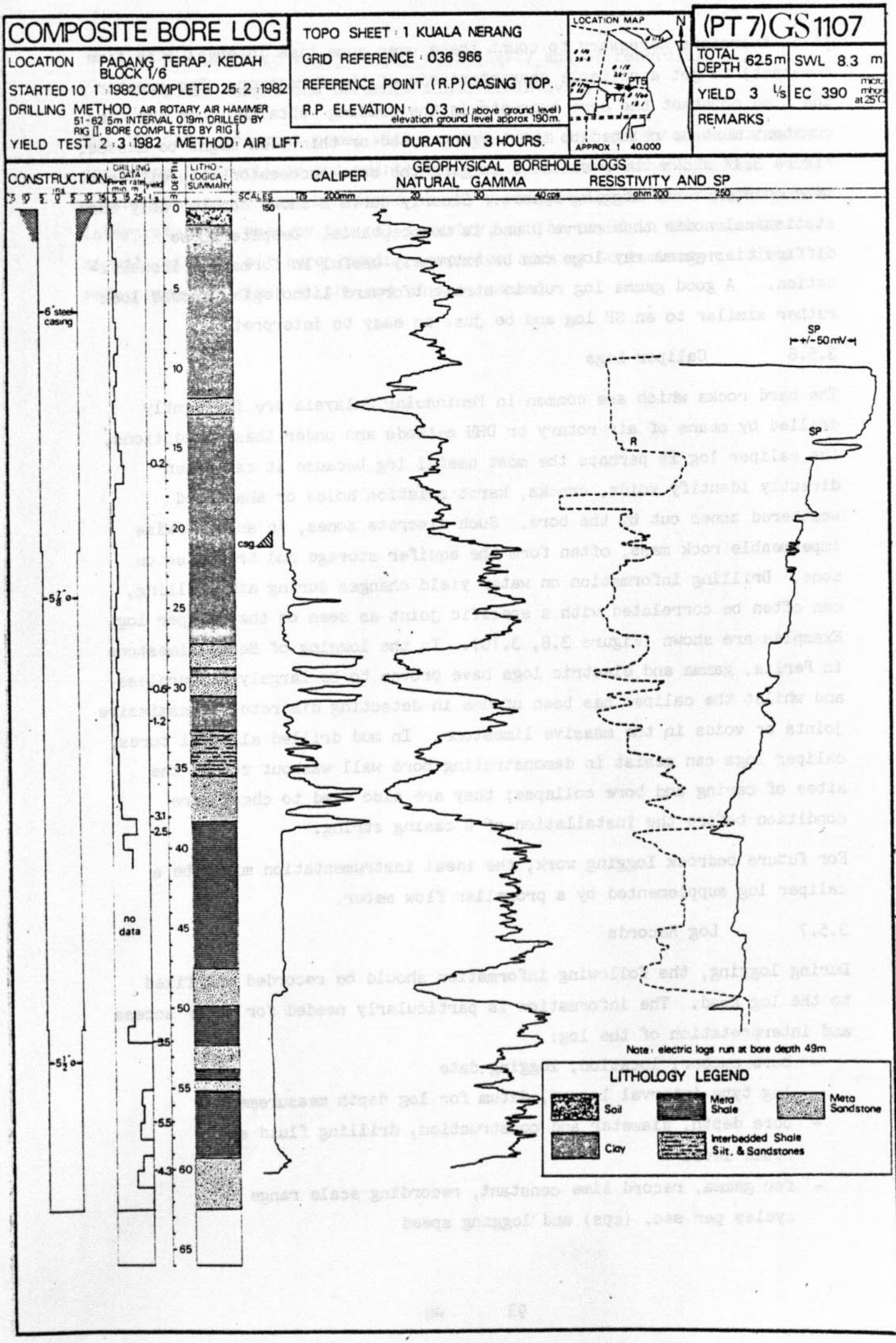


FIGURE 3.10 LOG EXAMPLES.



It is therefore necessary to count these over some time interval (the time constant) to get a reliable characteristic of the formation. The greater the time constant the more accruate is the reading obtained, but the time constant must be related to the logging speed or thin beds might be missed. Figure 3.11 shows the results of logging the same succession with different time constants and logging speeds. Clearly curve A shows considerably more statistical noise than curve D and is unacceptable. Despite these difficulties gamma ray logs can be extremely useful in formation identification. A good gamma log run in straightforward lithologies should look rather similar to an SP log and be just as easy to interpret.

3.5.6 Caliper Logs

The hard rocks which are common in Peninsular Malaysia are frequently drilled by means of air rotary or DHH methods and under these conditions, the caliper log is perhaps the most useful log because it can often directly identify voids, cracks, karst solution holes or shattered weathered zones cut by the bore. Such discrete zones, in an otherwise impermeable rock mass, often form the aquifer storage and transmission zone. Drilling information on water yield changes during air drilling, can often be correlated with a specific joint as seen on the caliper log. Examples are shown (Figure 3.8, 3.10). In the logging of Setul limestone in Perlis, gamma and electric logs have proven to be largely featureless and whilst the caliper has been of use in detecting discrete, transmissive joints or voids in the massive limestone. In mud drilled alluvial bores, caliper logs can assist in demonstrating bore wall washout zones, the sites of caving and bore collapse; they are also used to check bore condition before the installation of a casing string.

For future bedrock logging work, the ideal instrumentation might be a caliper log supplemented by a propeller flow meter.

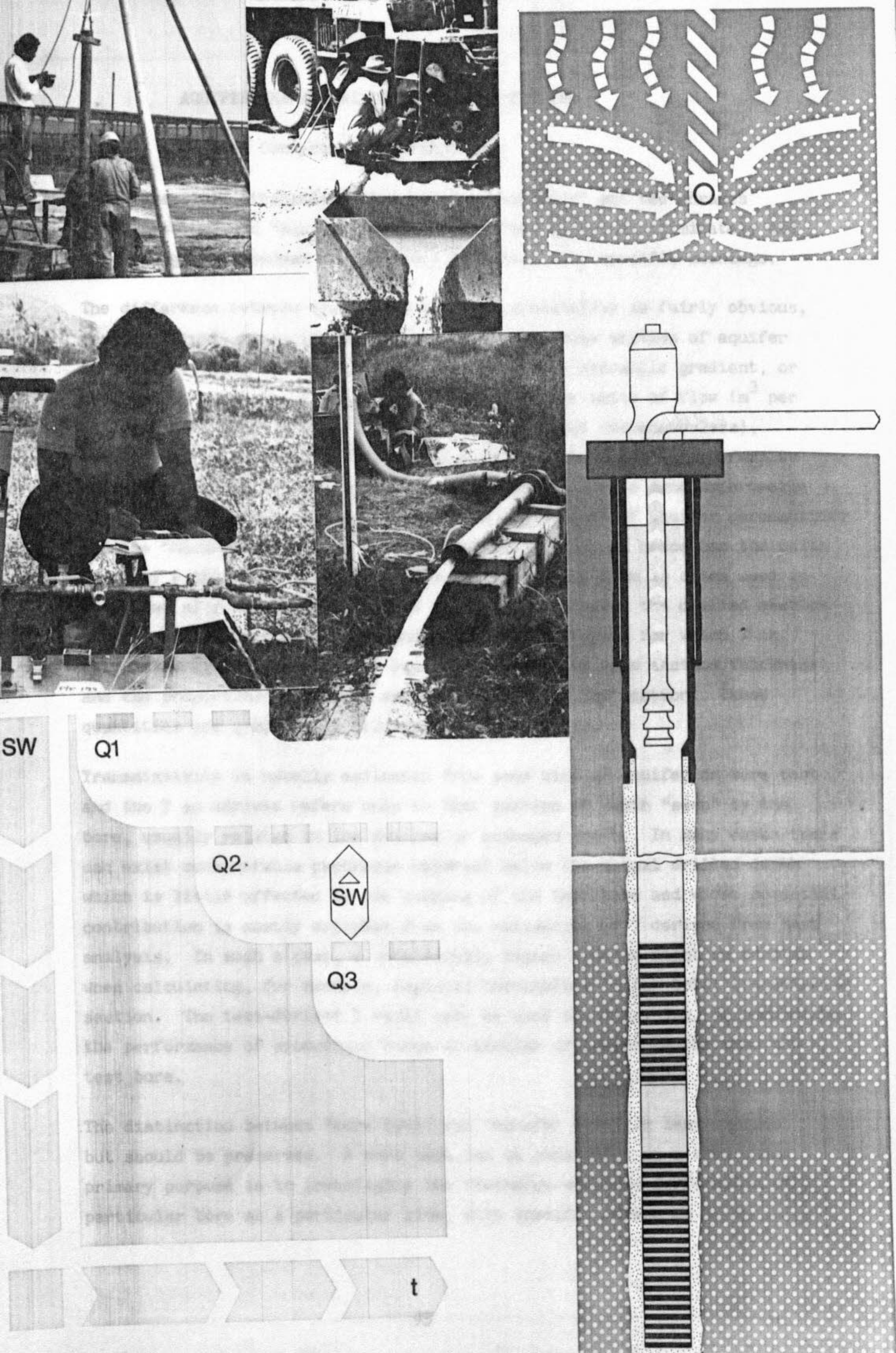
3.5.7 Log Records

During logging, the following information should be recorded and fixed to the log head. The information is particularly needed for later access and interpretation of the log:

- bore number, location, logging date
- log type interval logged, datum for log depth measurement
- bore depth, diameter and construction, drilling fluid and fluid level
- for gamma, record time constant, recording scale range in cycles per sec. (cps) and logging speed

- for electric logs, record resistivity and SP recording scale ranges and borehole fluid resistivity.

The JPT SIE instrument comes with comprehensive and easily understandable operating instructions and maintenance manuals. More detail on the interpretation of logs mentioned above, particularly the more quantitative interpretations used by the oil industry, can be found in the various manuals of the oil bore logging companies such as Schlumberger. Some references are given at the end of this manual.


SW
Q1
Q2
SW
Q3
t

4. AQUIFER CHARACTERISTICS AND WELL TESTING

4.1 General Concepts and Definitions

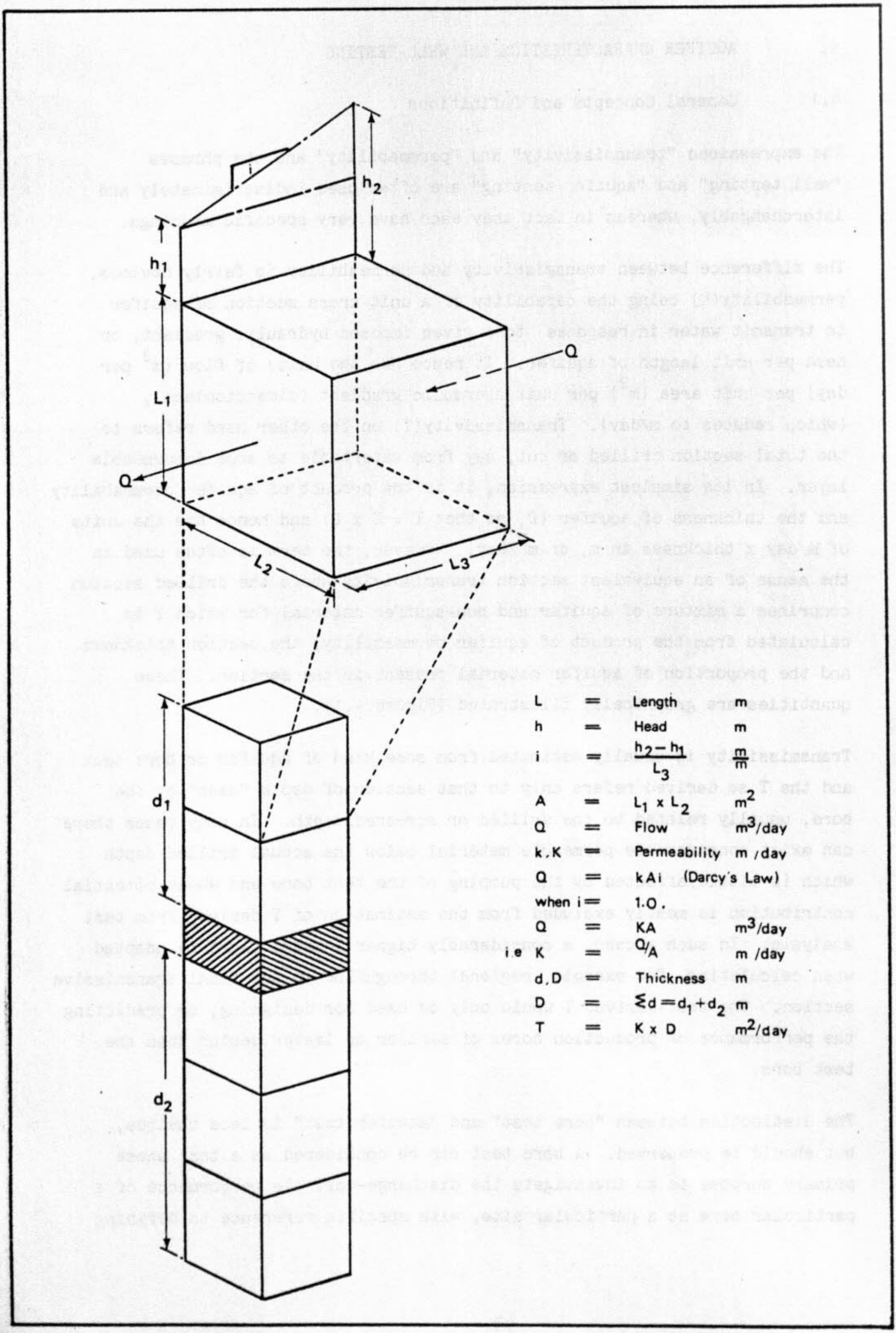
The expressions "transmissivity" and "permeability" and the phrases "well testing" and "aquifer testing" are often used indiscriminately and interchangably, whereas in fact they each have very specific meanings.

The difference between transmissivity and permeability is fairly obvious, permeability(K) being the capability of a unit cross section of aquifer to transmit water in response to a given imposed hydraulic gradient, or head per unit length of aquifer. It hence has the units of flow (m^3 per day) per unit area (m^2) per unit hydraulic gradient (dimensionless), (which reduces to m/day). Transmissivity(T) on the other hand refers to the total section drilled or cut, say from watertable to some impermeable layer. In its simplest expression, it is the product of aquifer permeability and the thickness of aquifer (D, so that $T = K \times D$) and hence has the units of m/day x thickness in m, or m^2/day. However, the term is often used in the sense of an equivalent section transmissivity where the drilled section comprises a mixture of aquifer and non-aquifer material for which T is calculated from the product of aquifer permeability, the section thickness and the proportion of aquifer material present in the section. These quantities are graphically illustrated (Figure 4.1).

Transmissivity is usually estimated from some kind of aquifer or bore test and the T so derived refers only to that section of depth "seen" by the bore, usually related to the drilled or screened depth. In many cases there can exist considerable permeable material below the actual drilled depth which is little affected by the pumping of the test bore and whose potential contribution is mostly excluded from the estimation of T derived from test analysis. In such a case, a considerably higher T value might be adopted when calculating, for example, regional throughflow in the total transmissive section. The test-derived T would only be used for designing, or predicting the performance of production bores of similar or lesser depths than the test bore.

The distinction between "bore test" and "aquifer test" is less obvious, but should be preserved. A bore test can be considered as a test whose primary purpose is to investigate the discharge-variable performance of a particular bore at a particular site, with specific reference to defining

FIGURE 4.1 PERMEABILITY AND TRANSMISSIVITY



the location of the major aquifer zones and the so called 'efficiency' of the bore. (Note that efficiency is not a preferred term since its absolute value changes with discharge and so can never be uniquely defined).

An aquifer test can be considered as a test, carried out by pumping a bore, whose prime function is to evaluate the characteristics of the overall aquifer in the general region of a bore site; such tests are usually done in conjunction with the simultaneous recording of groundwater behaviour in the neighbourhood of the bore, by means of specially constructed piezometers or observation bores.

Terms used throughout this chapter are now defined below. Most of these definitions can also be found in standard texts on hydrogeology.

<u>Coefficient of Permeability</u> of a material, as used in hydrogeology, is defined as the rate of flow of water through a unit cross sectional area of the material under a unit hydraulic gradient. It has the dimensions of velocity (for example, m/day). It is a measure of the ease which fluids can move through the material.

<u>Coefficient of Transmissivity</u> of an aquifer is defined as the rate of flow of water through a cross section of unit width of the aquifer, under a unit hydraulic gradient. It is numerically equal to the product of average permeability and aquifer thickness.

<u>Coefficient of Storage</u> of an aquifer is the volume of water which is released from storage per unit surface area per unit head loss normal to that surface. It is dimensionless and often expressed both as a decimal fraction and as a percentage.

<u>Specific Capacity</u> of a bore is its discharge divided by its drawdown.

<u>Specific Drawdown</u> of a bore is its drawdown per unit discharge. It is the reciprocal of specific capacity.

<u>Bore</u>; the words borehole, well or tubewell are often used instead. All have slightly different connotations in different countries and almost any definition will be disputed. In this manual, <u>bore</u> is a deep vertical cylindrical structure, which is usually lined and may be screened and which may have been constructed for groundwater exploration or production purposes.

The word well may often be used instead of bore. Where used in this manual, the word well means a drilled well (not a shallow dug well) which may be lined and screened. A tubewell invariably is a production bore.

Bores can be qualified as of exploration or production status depending on original purpose and projected end use. Where results are favourable, exploration bores may be redesignated as production bores.

Confined and Unconfined Aquifers A confined aquifer is a permeable formation containing water under pressure, confined at the top by a layer of restricted permeability. In an unconfined aquifer there is no such confining layer and the upper surface of the water body is free (the phreatic surface or the water table). The main difference is that confined flow can be horizontal whereas unconfined flow always has both horizontal and vertical components and the direction of flow varies with depth (Figure 4,2, 4.4 and 4.5).

Leaky Aquifers are aquifers which, when pumped, are replenished from either above or below by seepage through confining beds of low permeability. (Figure 4.6).

Partially Penetrating bore is a bore which taps less than the full thickness of the aquifer. The ratio of screened bore depth to total aquifer depth is termed the penetration factor (Figure 4.3).

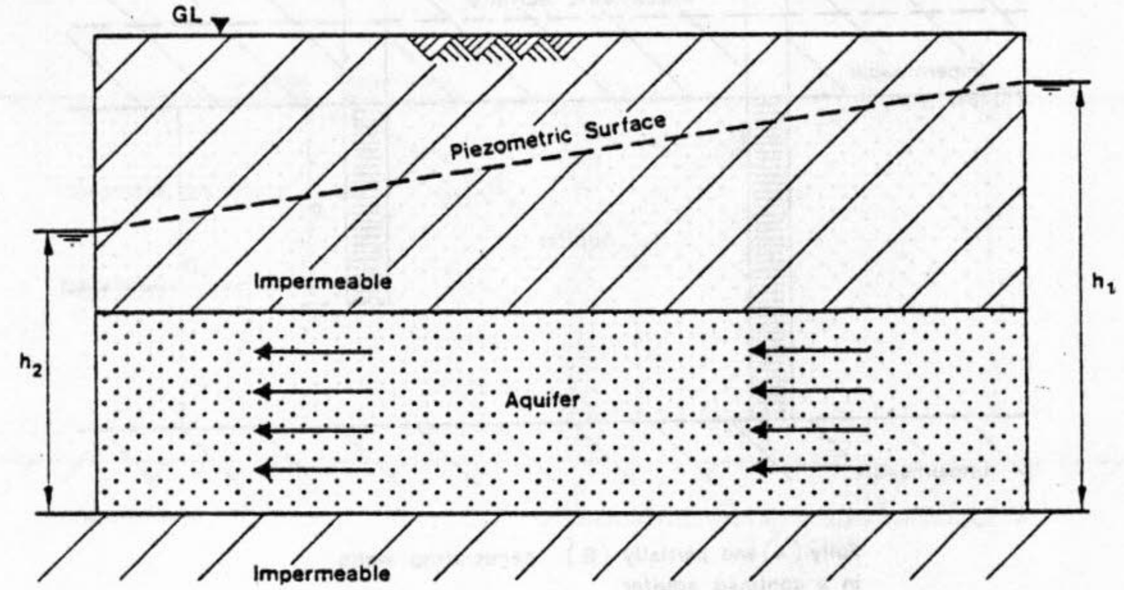
Non-equilibrium The condition when the method of analysis (such as the Theis equation) recognises that the rate of drawdown in the aquifer varies both with time and distance with respect to the bore; complex functions are necessary to describe this variance.

Semi-equilibrium The condition when for distances close to the bore or for long pumping times, the complex time-drawdown functions can be simplified using the expression that $\frac{ds}{dlogt}$ = Constant.

Equilibrium (or steady-state) The condition that should rarely if ever occur in theory, but which is common in practice, when the rate of change of drawdown, $\frac{ds}{dt}$, approximates zero or is constant at considerable distances from the well (Figure 4.4, 4.5 and 4.6).

FIGURE 4.2 CONFINED AND UNCONFINED FLOWS

a) Confined flow



b) Unconfined flow

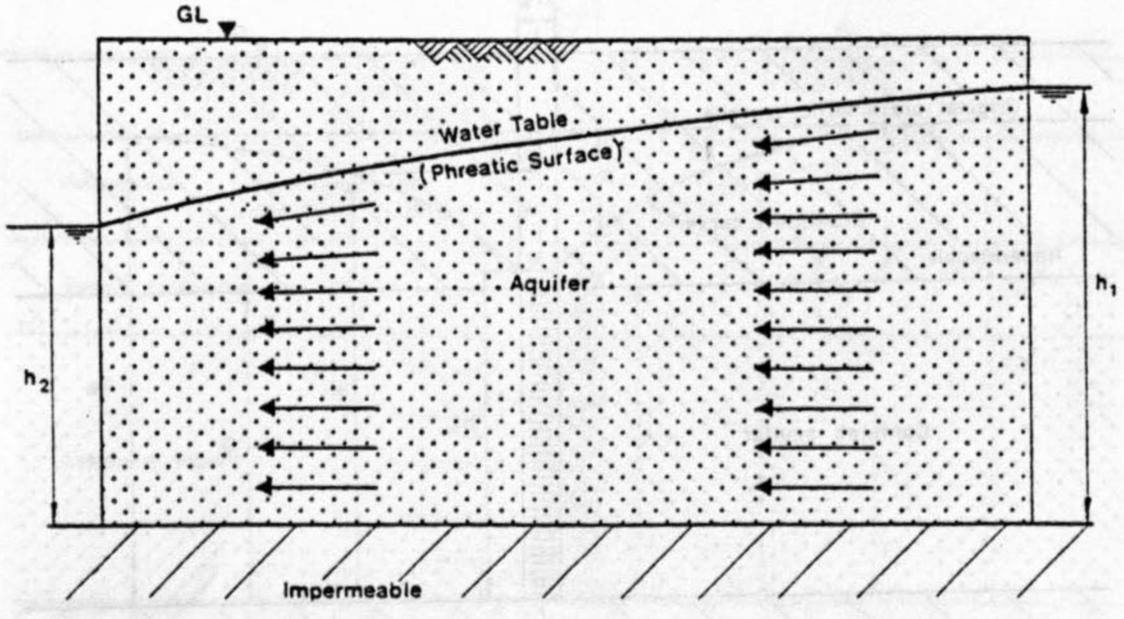
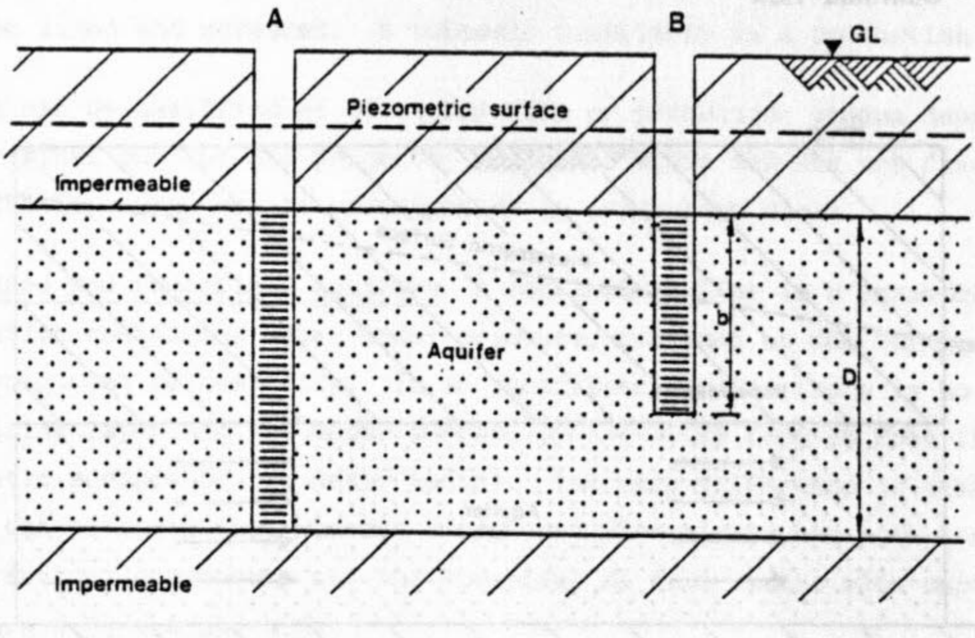


FIGURE 4.3 WELL PENETRATION



Fully (A) and partially (B) penetrating wells in a confined aquifer

FIGURE 4.4 STEADY STATE CONFINED FLOW

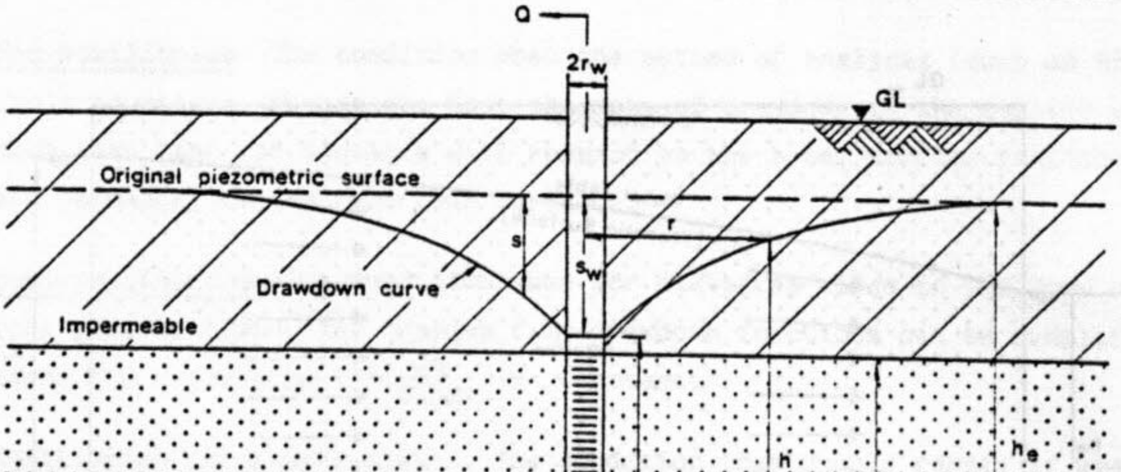


FIGURE 4.5

STEADY STATE FLOW-UNCONFINED AQUIFER

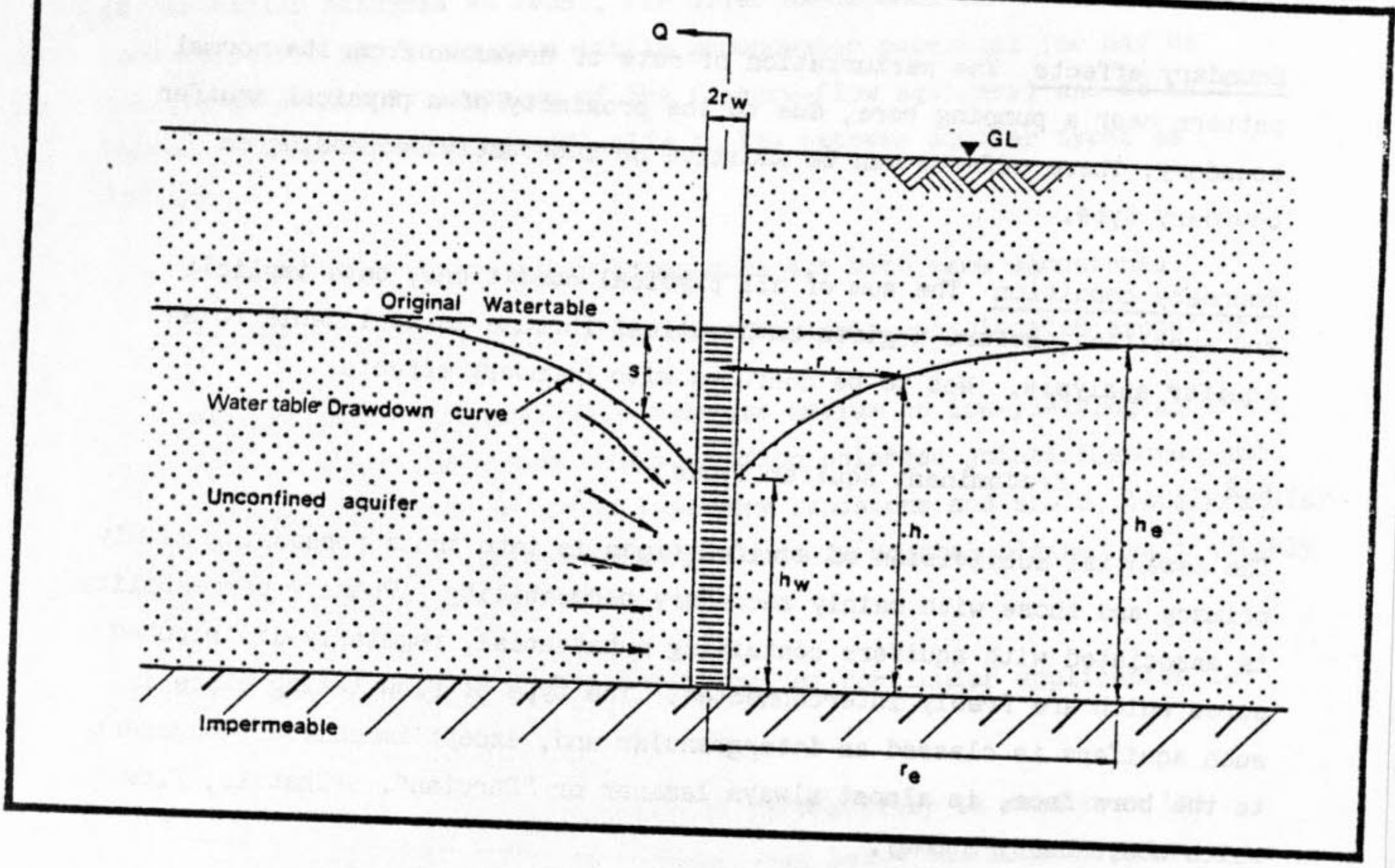
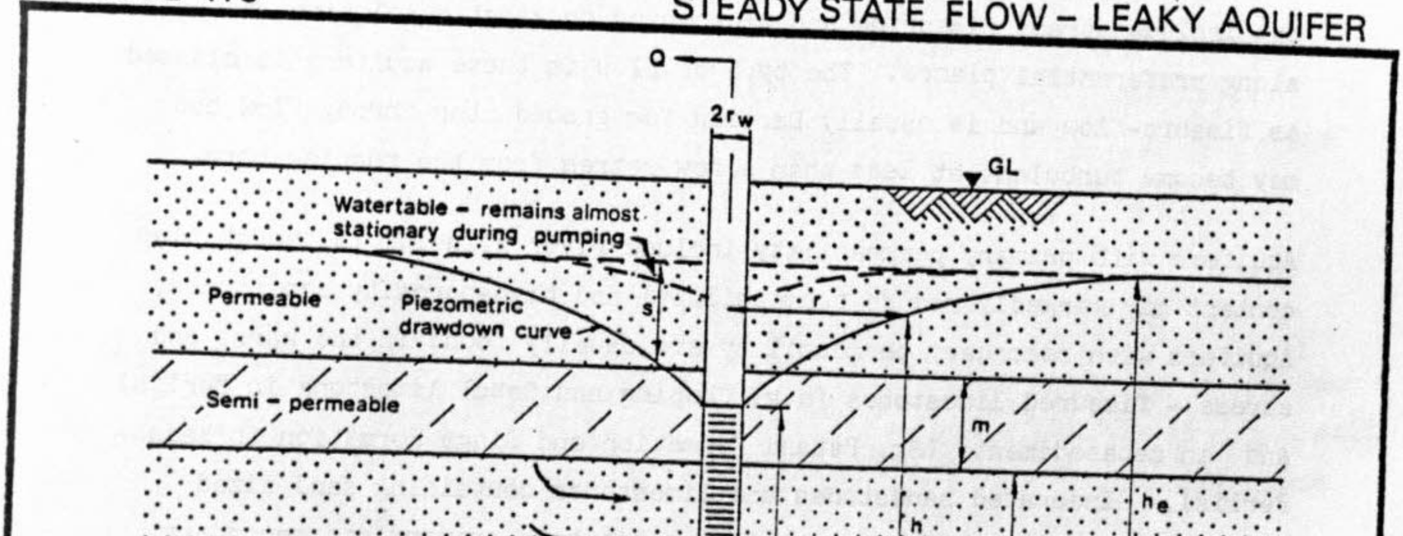


FIGURE 4.6

STEADY STATE FLOW - LEAKY AQUIFER



Boundary effects The perturbation of rate of drawdown from its normal pattern near a pumping bore, due to the proximity of a physical aquifer boundary; these effects can be additive or subtractive depending on boundary type.

Boundary condition The sum of all physical conditions, both implicit and specified, before a given mathematical formula is applicable to an aquifer analysis. Not to be confused with boundary effects.

4.2 Predominant Aquifer Types

The essential subdivision of aquifer types is into those possessing mainly primary and those with mainly secondary permeability. Primary permeability is associated with aquifers containing substantial, regularly-distributed pores which are freely interconnected. The type of flow taking place in such aquifers is classed as intergranular and, except immediately adjacent to the bore face, is almost always laminar or "Darcian". (That is, flow which obeys Darcy's Law).

Secondary permeability occurs where the pores are insubstantial, irregularly distributed and/or poorly connected, but where secondary transmission features, fissures or channels, have developed; such features can include stress fissuring and weathering or karstic solution channels along preferential planes. The type of flow in these aquifers is classed as fissure-flow and is usually Darcian for groundwater throughflow but may become turbulent at less than a few metres from the pumping bore.

Aquifers with primary permeability include river alluvium (although clay content may markedly reduce permeability) and high-porosity sandstones. Aquifers with secondary permeability are usually found in the karst and stress - fissured limestones (e.g. Chuping and Setul limestone in Perlis) and in metasediments (Sg. Petani Formation and Singa Formation in Kedah-Perlis). Indurated sandstones and limestones containing sand sized lime layers (calcarenytes) probably exhibit both primary and secondary permeability.

In Peninsular Malaysia at least, the older sandstones and metasediments (pre-Tertiary) appear to have little groundwater potential (or may be considered as poorer examples of the fissure-flow aquifers) and so this Manual addresses itself specifically to the extreme aquifer types as follows:

- fissured hard rock (mainly limestones but with same sandstones)
- intergranular soft rock (alluvium and poorly consolidated sandstone)

The fissured hard rocks generally comprise medium to low-yielding aquifers developed in karst limestones which solution enlarged joints/fissures and in joints and fractures in metasedimentary sandstone and shale. Intergranular soft rock aquifers are almost entirely found in coastal plains, particularly in the east coast alluvium.

A further useful aquifer type subdivision, of particular application is as follows:

- where the vertical distribution of permeability is even over the drilled section or is concentrated deep below the zone of drawdown and watertable natural fluctuation
- where the distribution of permeability is uneven, especially when it is highest in the potential zone of dewatering.

The distribution of permeability within an aquifer partly determines testing and testing methods. In soft rock or intergranular aquifers, the lateral variation of permeability from site to site is generally small whilst vertical variation can be related through observed lithology to the relatively large-scale depositional process involved in the formation of the rock. Permeability values derived for the region of the bore can therefore be reasonably extrapolated to adjacent regions by study of the lithology alone. The local permeability value can be applied to regional throughflow calculations whilst transmissivity changes with well depth change can be linearly interpolated. Aquifer tests have some value and are appropriate.

Almost opposite remarks apply to fissuured hard rock aquifers (except to solution karst limestones where transmissive systems are extremely well developed). This is because fissure and hence aquifer development is related to localised stress patterns and rock weathering features. Lateral permeability distribution in fissure aquifers is usually highly variable and the vertical variation is extreme, often being concentrated in two or three thin and very localised fissure zones. Aquifer test results cannot sensibly be used for regional and depth extrapolation, even with lithological control. The bore T value can rarely be applied to regional throughflow calculations and transmissivity itself is practically meaningless because of its variability. Typically, small variations in drilled depth can expose a new fissure system whilst water table declines either naturally or by pumping, lead to dewatering of upper yielding fissure zones and consequent large falls in transmissivity. Further, turbulent flow in thin fissure zones near the bore can easily confuse the simple theoretical determination of the actual T or K value concerned.

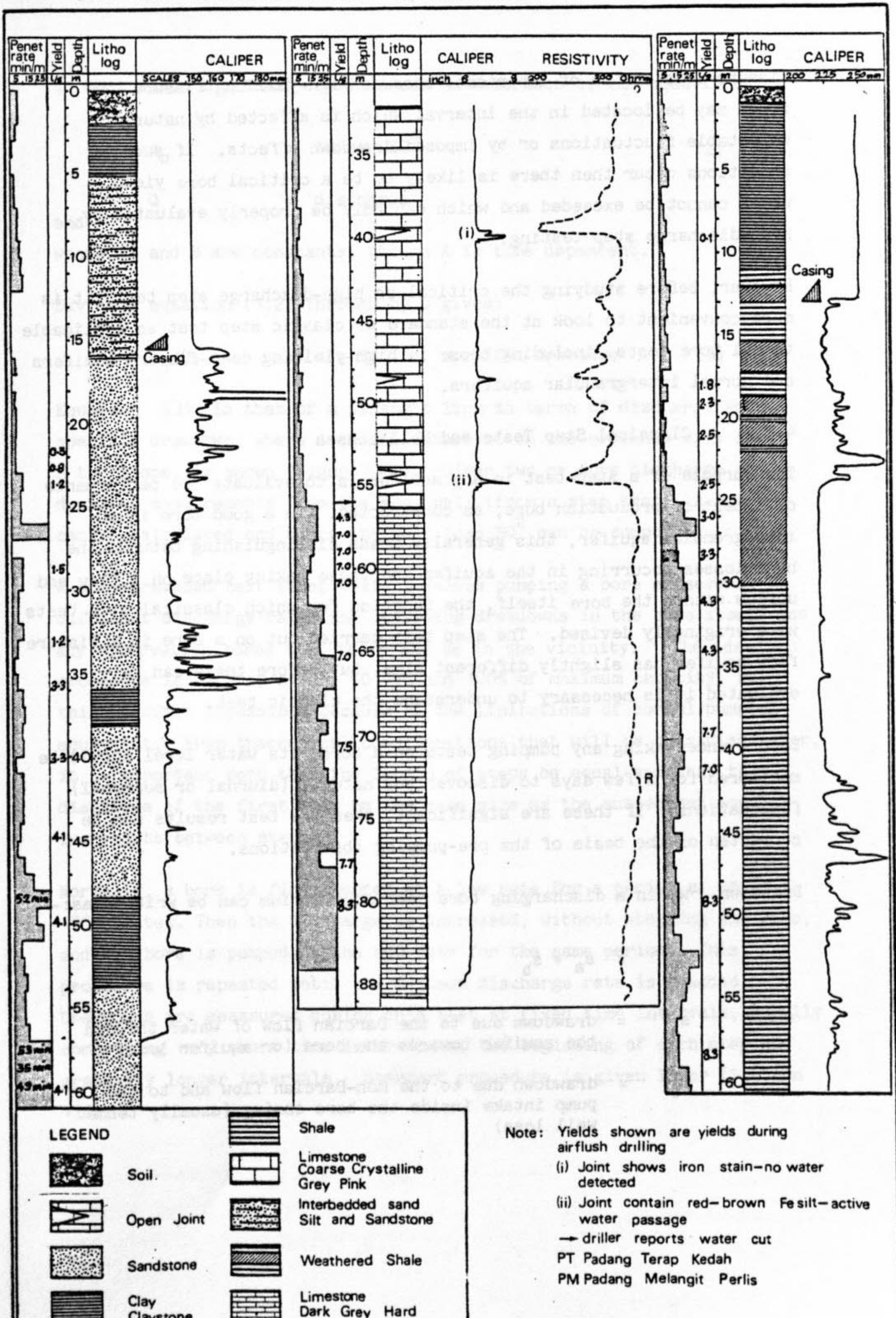
There is therefore little point in using piezometers in an aquifer of such high lateral variation since the piezometer may not intersect or terminate in the same fissure zone as the bore. A simple bore test, designed to investigate the permeability distribution and extent of the fissure zones, is more appropriate. Such tests can however be used, albeit only on a statistical rather than a determinant basis, to describe an aquifer's regional characteristics.

4.3 Shallow Fissured Aquifers

4.3.1 General

The depth distribution of yielding zones in a fissure-flow aquifer may determine the type of bore test adopted. Fissure distribution can often be predicted by a detailed study of the driller's operational log, particularly of sudden penetration rate increases and of water shows or yields relative to depth when drilling with air. It can be further evaluated by analysis of geophysical logs run after the completion of drilling and before any casing, especially of the caliper log (and neutron log if available). Examples are given from three exploratory air drilled bores in the Kedah-Perlis region (Figure 4.7). These examples indicate the often quite close correspondence between indentations on the caliper log and reported water shows or increases in water produced during air drilling.

FIGURE: 4.7 FISSURE ZONE DETECTION



Fissure location is of importance because high-yielding fissure zones may be located in the interval which is affected by natural watertable fluctuations or by imposed drawdown effects. If such conditions occur then there is likely to be a critical bore yield which cannot be exceeded and which can only be properly evaluated by high-discharge step testing.

However, before studying the critical or high-discharge step test, it is more convenient to look at the standard or classic step test as applicable to all bore tests, including those in high-yielding deep-fissure aquifers and normal intergranular aquifers.

4.3.2 Classical Step Tests and Well Losses

The purpose of a step-test in any aquifer is to evaluate the performance of a test or production bore, as constructed. In a good bore in an intergranular aquifer, this generally means distinguishing between the head losses occurring in the aquifer and those taking place on inflow and upflow within the bore itself, the function for which classical step tests were originally devised. The step test carried out on a bore in a fissure flow aquifer has slightly different aims, but before these can be evaluated it is necessary to understand the classic test.

Before undertaking any pumping tests on a bore, its water level should be monitored for a few days to discover any natural (diurnal or seasonal) fluctuations. If these are significant, then the test results can be corrected on the basis of the pre-pumping observations.

Drawdown, s_w, in a discharging bore near equilibrium can be written as:-

$$s_w = s_a + s_b \qquad (1)$$

where s_a = drawdown due to the Darcian flow of water through the aquifer towards the bore (or aquifer loss)

s_b = drawdown due to the non-Darcian flow and to the pump intake inside the bore casing (usually termed well loss)

In most cases the total drawdown can be described by the equation:-

$$s_w \quad = AQ + BQ^2 \qquad (2)$$

$$Q \quad = \text{discharge}$$

and

where A and B are constants, though A is time dependent.

Dividing equation (3.2) through by Q gives:

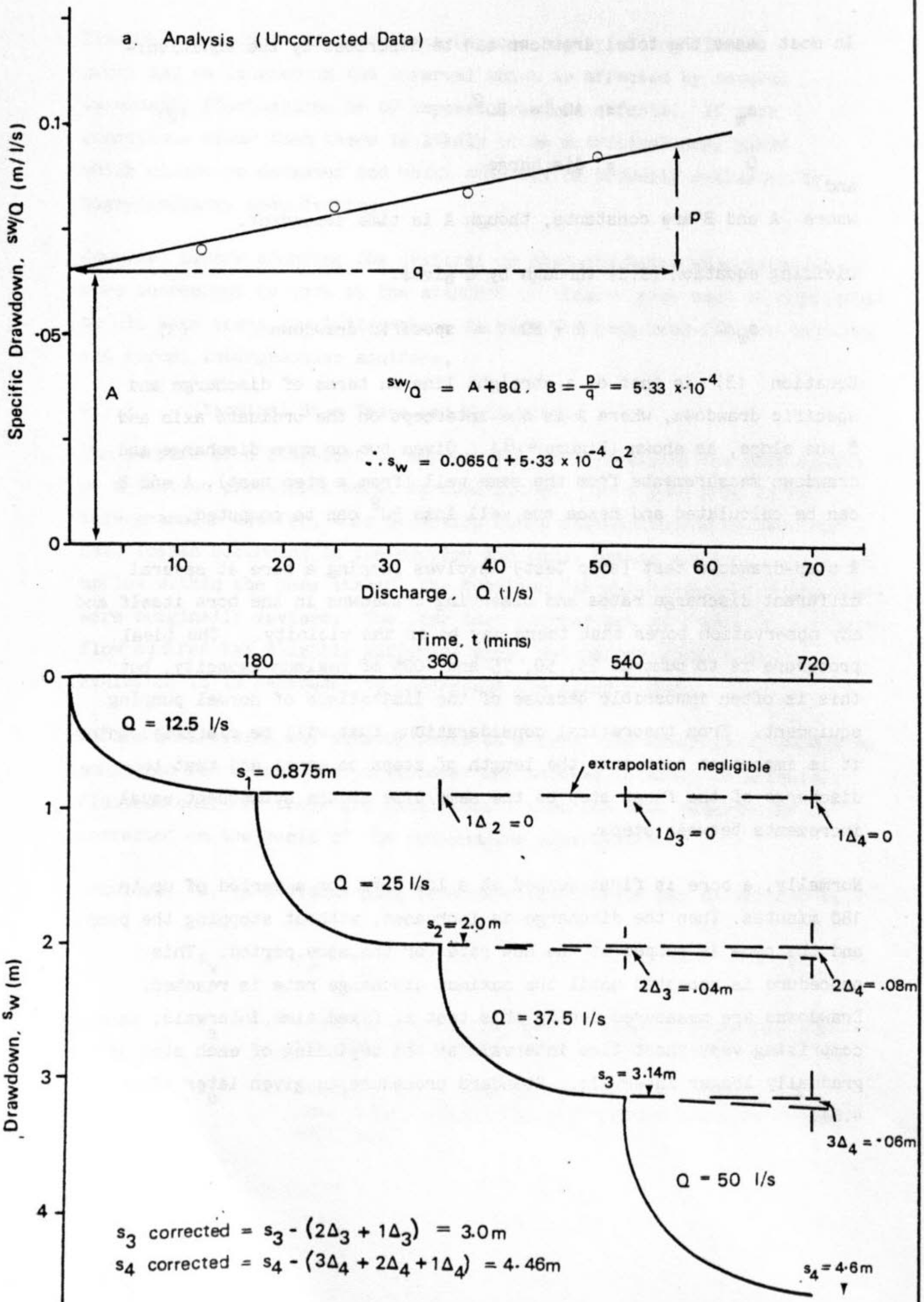
$$s_w/Q \quad = A + BQ \quad = \text{specific drawdown} \qquad (3)$$

Equation (3) is that of a straight line in terms of discharge and specific drawdown, where A is the intercept on the ordinate axis and B the slope, as shown (Figure 4.8). Given two or more discharge and drawdown measurements from the same well (from a step test), A and B can be calculated and hence the well loss BQ^2 can be computed.

A step-drawdown test (step test) involves pumping a bore at several different discharge rates and observing drawdowns in the bore itself and any observation bores that there may be in the vicinity. The ideal procedure is to pump at 25, 50, 75 and 100% of maximum capacity, but this is often impossible because of the limitations of normal pumping equipment. From theoretical considerations that will be clarified later, it is important both that the length of steps be equal and that the discharge of the first step be the same size as the subsequent equal increments between steps.

Normally, a bore is first pumped at a low rate for a period of up to 180 minutes. Then the discharge is increased, without stopping the pump, and the bore is pumped at the new rate for the same period. This procedure is repeated until the maximum discharge rate is reached. Drawdowns are meaasured during this test at fixed time intervals, usually comprising very short time intervals at the beginning of each step and gradually longer intervals. Standard procedure is given later (Section 4.6).

FIGURE 4.8 STEP TEST ANALYSIS



Application of equation (3) requires that the data ought to be derived from conditions where the time-variable is identical in each case and drawdown conditions are "Semi-equilibrium" (That is $\frac{ds}{dlogt}$ is constant). This would imply separate steps, each with a full recovery period between them so that the bore could be regarded as having been pumped for an equal time at each discharge. The step test as employed is clearly only an appromation of this sutation, but the principle of superimposition can be used to correct for this factor. This entials extrapolating the drawdown of each lower discharge step to estimate what the water level would have been had the bore continued pumping at the same rate for the next time increment. The drawdown used in analysis is then not the gross drawdown but the gross drawdown less the sum of the differential extrapolated drawdowns, as is also illustrated (Figure 4.8). The application of such a correction to the step test data is based on experience, but is related to the shape of the discharge/drawdown data plot obtained; the closer the steps come to "equilibrium", the less renecssary is the correction.

Nevertheless, the methods described (equation (3) and Figure 4.8) can be used to evaluate well-loss coefficient, 'B'. By looking at other discharges, we can see how efficient the bore will be or alternatively, how much of the drawdown is aquifer losses (unavoidable) and how much is well-losses (partly under the control of the designer and driller). For the bore used as an example, the losses would appear to be some 1.33 m at a discharge of 50 l/s, out of a total drawdown of 4.6 m. Some people would quote this bore as being (4.6-1.33) 4.6 = 100, or about 72.4% "efficient". This concept is misleading however, since the "efficiency" changes with discharge and the use of the word implies that all of the losses are avoidable which they are not. It is better simply to quote the well loss coefficient.

The losses comprise an entry loss plus upflow or 'pipe losses' within the bore. As described in section 5.4, entry loss can be minimised by proper design and construction procedures but never entirely eliminated. Upflow losses vary with bore total depth and with bore or screen and casing internal roughness, as well as varying proportionately with discharge squared. The whole subject is complex and open to some argument but it is considered that upflow losses can and should be maintained in the range 0.6 to 0.3 m for normal irrigation bores.

This can be achieved by using the correct finished bore, screen or casing internal diameter. The recommended target design discharges for various materials to limit upflow losses are given below. (Table 4.1).

Table 4.1

Target Design Discharges: (l/s)

Well/casing: ID (inches)	Screen/casing material Slotted Plastics	Slotted Steel	Punched or Wire-wound	Hard rock open hole
4	15	15	15	10
6	30	30	25	20
8	60	55	50	40
10	100	90	80	80

Somewhat exaggerated claims are made as to the utility of wire-wound screens in reducing entry losses; most of this is simply manufacturers' propaganda. The main reason for using such screen is in cases of high-yielding but limited thickness aquifers or where the installation of an artifical gravel pack might be difficult, dangerous or expensive.

The step test, or any low-discharge test, can also be used to derive a good, approximate estimate of the transmissivity, T, of the piece of aquifer "seen" by the screened or producing section of the bore. The method of estimation is based on the so-called "Logan approximation" which is itself derived from the Thiem steady-state euquation:

$$s_w = \frac{2.303\,Q}{2\pi T} \cdot \log \frac{re}{rw} \quad \text{(Thiem)} \qquad (4)$$

where r_e and r_w are respectively the radius of influence and the radius of the bore.

Although r_e is an imaginary parameter, the effective value of the ratio of r_e to r_w, calculated by various workers by more rigorous means, has been found to lie commonly in the range 1,000 to 7,000. Logan, amongst others, noted that the calculation of T by equation (4) was remarkably insensitive to even large variations, depending as it does on a log of the ratio.

Taking re/rw as 1,000, equation (4) reduces to:

$$s_w = \frac{1.10Q}{T} \qquad (5)$$

or, as 7,000 :

$$s_w = \frac{1.41Q}{T} \qquad (6)$$

Logan proposed a numerical constant of 1.22, which would give:

$$T = \frac{1.22Q}{s_w} \quad \text{(Logan Approximation)} \qquad (7)$$

This expression can be applied to project bores until experience proves otherwise, provided that:

- well losses are small compared to aquifer losses;
- the bore is close to or at equilibrium;
- there are no recharge or barrier boundaries near the bore.

The step test gives a means of estimating the value of sw/Q if well losses were totally absent. It is simply the value of the intercept 'A' (Figure 4.8(a)) converted of course to m^3/day units from the l/s units in which it is derived. If the l/s numerical value of A is used in calculation, the Logan formula becomes:

$$T = \frac{1.22 \times 86.4}{A} \qquad (8)$$

Permeability is usually estimated by dividing the estimate of T by the length of the yielding screened section or that length plus a few metres (up to 15 m) to allow for partial penetration.

The value of the Logan approximation is that it is rarely far wrong and it therefore gives a useful yardstick against which to measure the accuracy of results obtained by supposedly more theoretically rigorous and complex analyses. If the difference is great, then the supposedly "rigorous" method has been wrongly applied or, more likely, the boundary conditions pertinent to its application have not been complied with.

4.3.3 Modified Step Tests

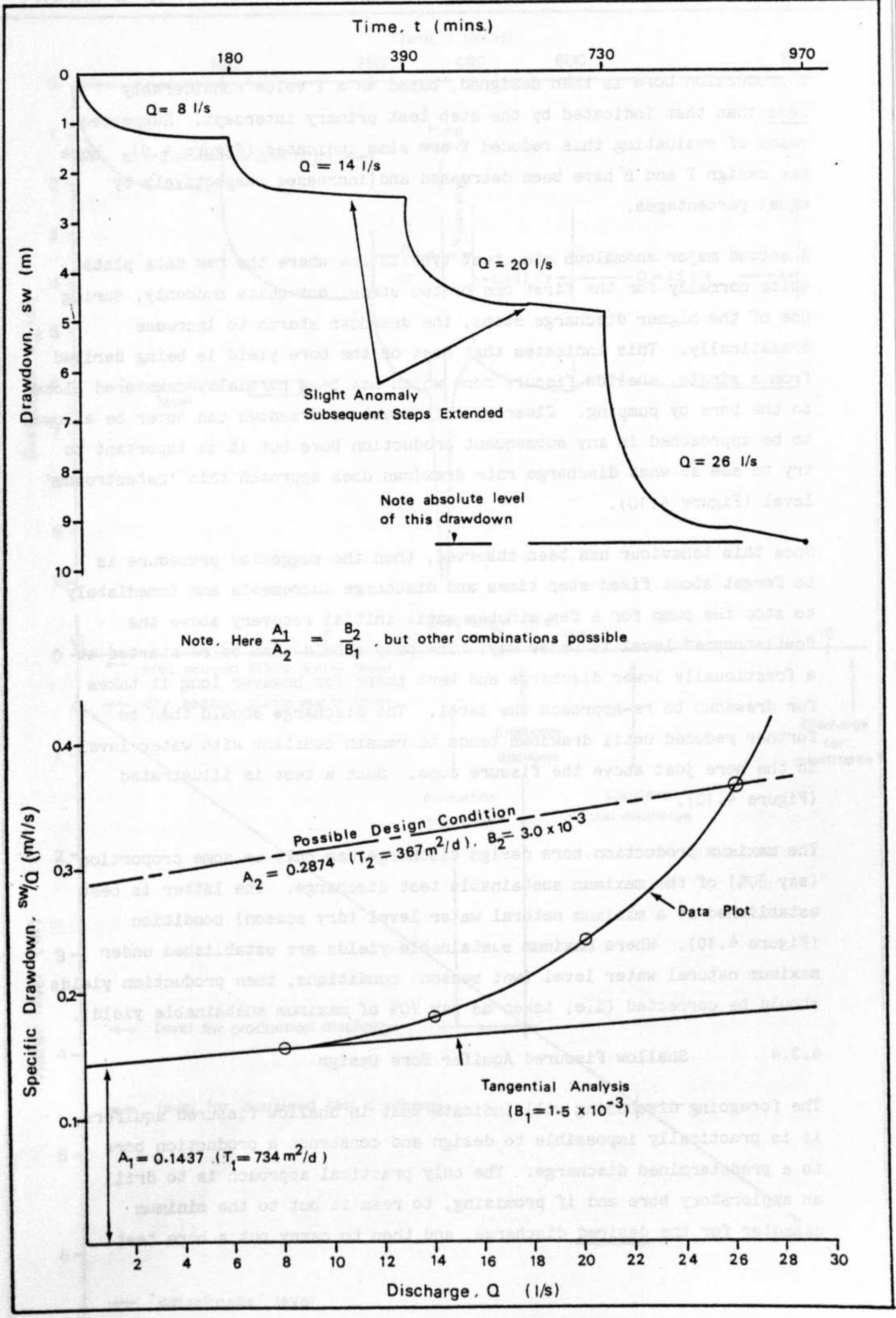
In an aquifer which is thought to yield from a few principal fissure zones, less weight should be put upon accurate theoretical determination of T and of the well-loss coefficient, B. This latter may anyway vary with discharge, either really, as the region of turbulent flow around the bore is increased, or apparently, as yielding zones are dewatered and T actually decreases.

For such aquifers, it is more important to have a test-pump capable of discharging considerably in excess of the bore's capacity, estimated from drilling yields. The requirement for equal time steps can be varied since the establishment of genuine steady-state drawdown conditions within a step is more important; any step showing time/drawdown anomalies in its plot (as detected by data plotting on site), should be continued until the anomaly is resolved; subsequent steps should also be lengthened. The principle of superimposition can be used to sort out the data for analysis.

Two likely conditions can occur with step-tests in a shallow-fissured aquifer. Firstly, the raw step-test data plot (Figure 4.9) can look quite normal at first sight, but the derived sw/Q plot, instead of a straight line with an intercept and a fixed slope, is a line with an intercept and a marked upward curve. This has been interpreted as indicating that the well-losses are increasing at a higher power than the square of discharge (Equation (3)).

However, a more likely interpretation is that the main transmissive zone is concentrated in the upper part of the saturated zone (i.e. that part being progressively dewatered by testing) and hence the effective transmissivity is actually declining. At the same time, apparent well losses may be increasing since a larger quantity of water is being forced to enter the bore through a decreasing inflow zone, thus increasing disproportionately the entry velocity, the parameter to which the squared term is strictly applicable. An indicative test is illustrated (Figure 4.9). In this case, the level at which the increase in apparent loss becomes intolerable, must be established. A drawdown no greater than the difference between this level and the lowest likely static water level generated by natural and imposed water table fluctuations is defined.

FIGURE 4.9 ANOMALOUS STEP TEST, TYPE 1



A production bore is then designed, based on a T value considerably less than that indicated by the step test primary intercept. Suggested means of evaluating this reduced T are also indicated (Figure 4.9). Here the design T and B have been decreased and increased respectively by equal percentages.

A second major anomalous step test type is one where the raw data plots quite normally for the first one or two steps, but quite suddenly, during one of the higher discharge steps, the drawdown starts to increase dramatically. This indicates that most of the bore yield is being derived from a single, shallow fissure zone which has been partially dewatered close to the bore by pumping. Clearly, this level of drawdown can never be allowed to be approached in any subsequent production bore but it is important to try to see at what discharge rate drawdown does approach this 'catastrophe' level (Figure 4.10).

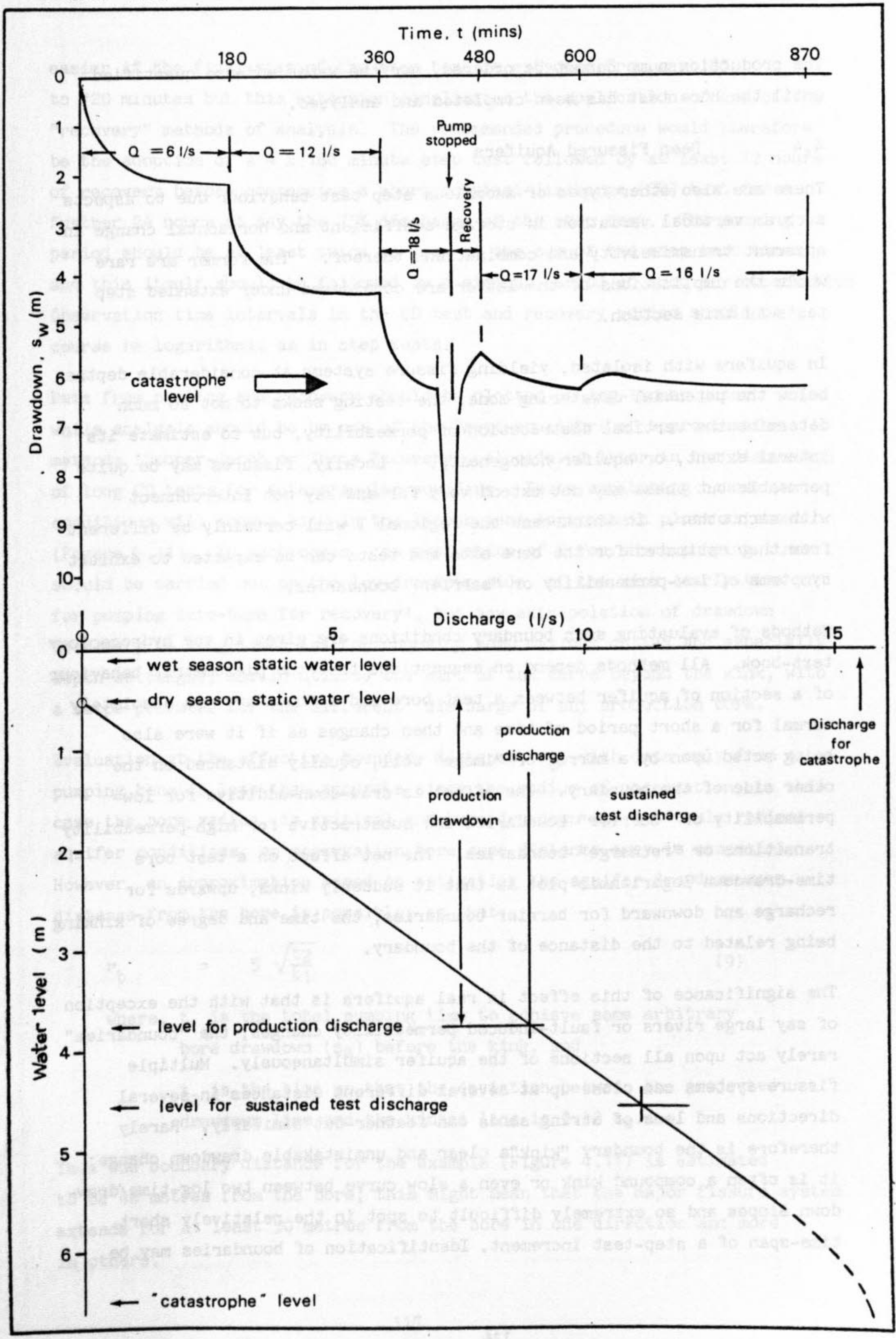
Once this behaviour has been observed, then the suggested procedure is to forget about fixed step times and discharge increments and immediately to stop the pump for a few minutes until initial recovery above the "catastrophe" level is under way. The pump should then be re-started at a fractionally lower discharge and kept there for however long it takes for drawdown to re-approach the level. The discharge should then be further reduced until drawdown tends to remain constant with water level in the bore just above the fissure zone. Such a test is illustrated (Figure 4.10).

The maximum production bore design discharge can only be some proportion (say 80%) of the maximum sustainable test discharge. The latter is best established at a minimum natural water level (dry season) condition (Figure 4.10). Where maximum sustainable yields are established under maximum natural water level (wet season) conditions, then production yields should be corrected (i.e. taken as say 70% of maximum sustainable yield).

4.3.4 Shallow Fissured Aquifer Bore Design

The foregoing discussing will indicate that in shallow fissured aquifers, it is practically impossible to design and construct a production bore to a predetermined discharge. The only practical approach is to drill an exploratory bore and if promising, to ream it out to the minimum diameter for the desired discharge, and then to carry out a bore test.

FIGURE 4.10 ANOMALOUS STEP TEST, TYPE 2



The production pump cannot be ordered, nor the water end-use quantified until the bore test has been completed and analysed.

4.4 Deep Fissured Aquifers

There are also other types of anomalous step test behaviour due to aspects such as vertical variation in storage coefficient and horizontal change in apparent transmissivity and combinations thereof. The former are rare while the implications of the latter are considered under extended step tests in this section.

In aquifers with isolated, yielding fissure systems at considerable depths below the potential dewatering zone, the testing seeks to not so much determine the vertical distribution of permeability, but to estimate its lateral extent, or aquifer homogeneity. Locally, fissures may be quite permeable but these may not extend very far and may not interconnect with each other. In which case the regional T will certainly be different from that estimated for the bore site and tests can be expected to exhibit symptoms of low-permeability or "barrier" boundaries.

Methods of evaluating such boundary conditions are given in any hydrogeology text-book. All methods depend on assumptions that the piezometric behaviour of a section of aquifer between a test bore and a "boundary" is absolutely normal for a short period of time and then changes as if it were also being acted upon by a mirror or "image" well, equally distanced on the other side of the boundary. The effect is draw-down-additive for low-permeability or "barrier" boundaries and substractive for high-permeability transitions or "recharge" boundaries. The net effect on a test bore time-drawdown logarithmic plot is that it suddenly kinks, upwards for recharge and downward for barrier boundaries, the time and degree of kinking being related to the distance of the boundary.

The significance of this effect in real aquifers is that with the exception of say large rivers or fault-induced permeability changes, the "boundaries" rarely act upon all sections of the aquifer simultaneously. Multiple fissure systems can close up at several different distances in several directions and lens of string sands can feather-out similarly. Rarely therefore is the boundary "kink"a clear and unmistakable drawdown change; it is often a compound kink or even a slow curve between two log-time/drawdown slopes and so extremely difficult to spot in the relatively short time-span of a step-test increment. Identification of boundaries may be easier if the final step of the step test is extended from say 180 to 720 minutes but this extension complicates the application of any of the "recovery" methods of analysis. The recommended procedure would therefore be the adoption of a 4 x 180 minute step test followed by at least 12 hours of recovery before commencing a short constant-discharge (CD) test for a further 24 hours at say the 75% discharge of the step test. The pumping period should be at least twice as long as the sum of the step test times and this itself should be followed by a similar period of recovery observation. Observation time intervals in the CD test and recovery period should of course be logarithmic as in step tests.

Data from pumping and recovery should be plotted on log-linear paper while analysis should be by one of the "semi-equilibrium" approximation methods (Cooper-Jacob or Theis Recovery), elucidated later in the discussion of long CD tests for intergranular aquifers. Tests exhibiting boundary conditions will show a kink in the linear semi-logarithmic plots as shown (Figure 4.11). In such cases, the evaluation of T for short pumping times should be carried out on the low-drawdown side of the kink (early-time for pumping late-time for recovery), but any extrapolation of drawdown necessary to long times (as for choosing pump maximum duties and especially depth settings), should utilise the part of the curve beyond the kink, with a slope prorated for the different discharge of any production bore.

Evaluation at the effective boundary distance, r_b, with data only from a pumping bore is less than accurate since the radius of observation, in this case the bore radius, is critical and bore drawdown may not truly reflect aquifer conditions; an observation bore some distance away is essential. However, an approximation based on estimating the aquifer drawdown some distance from the bore is possible, so that:

$$r_b = 5\sqrt{\frac{t2}{t_1}} \qquad (9)$$

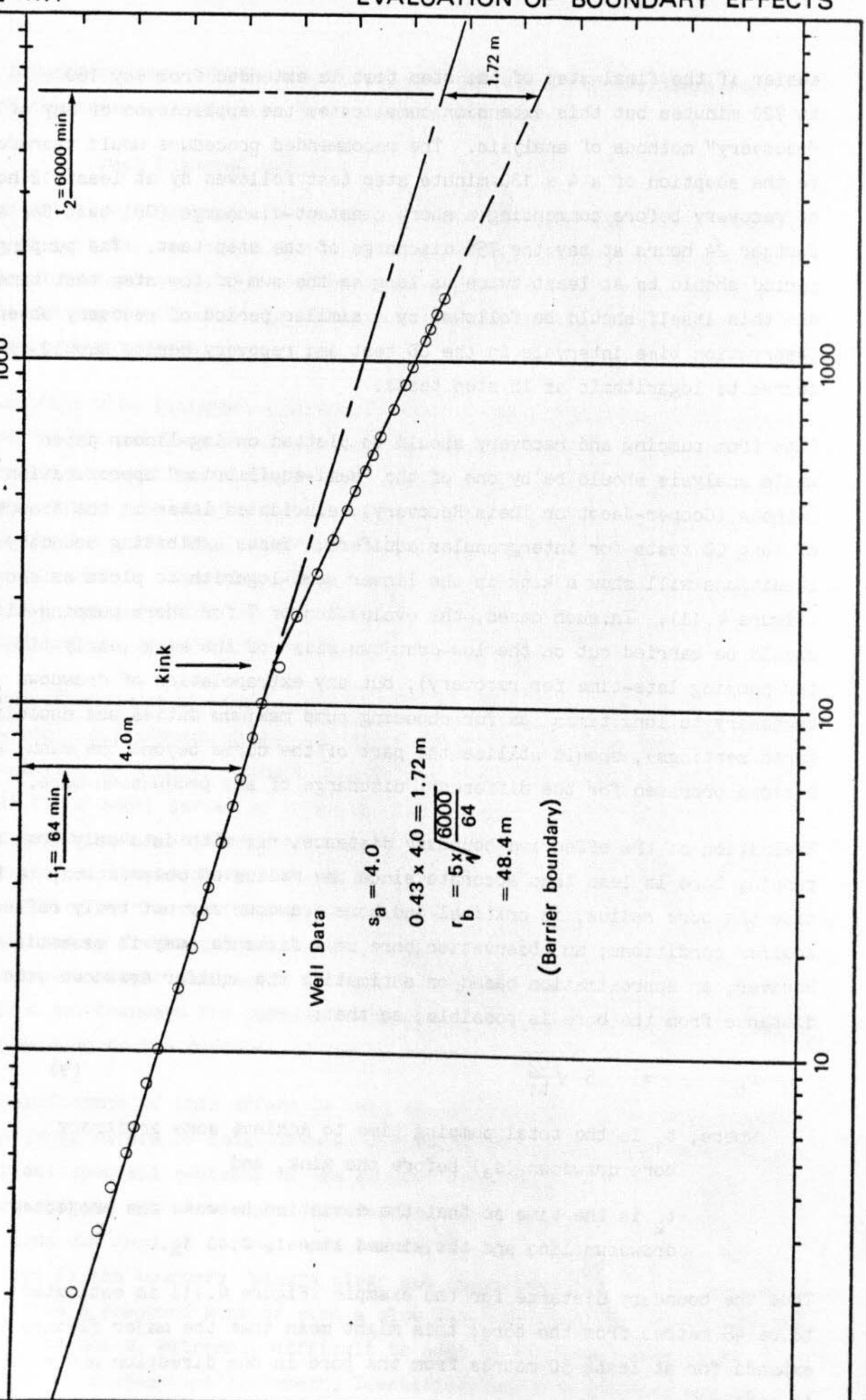
where, t_1 is the total pumping time to achieve some arbitrary bore drawdown (s_a) before the kink, and

t_2 is the time so that the deviation between the projected drawdown line and the kinked line is 0.43 s_a.

Thus the boundary distance for the example (Figure 4.11) is estimated to be 48 metres from the bore; this might mean that the major fissure system extends for at least 50 metres from the bore in one direction and more in others.

FIGURE 4.11

EVALUATION OF BOUNDARY EFFECTS



For piezometer data, the formula to be used would be:

$$r_b = \frac{r_o}{2}\sqrt{\frac{t_3}{t_1}} \qquad (10)$$

where r_o is the piezometer distance from the bore, in the boundary direction, and,

t_3 is the time for the drawdown deviation to equal 1.0 x s_a.

4.5 Intergranular Aquifers

4.5.1 Aquifer Test Set-up

An aquifer test comprises the pumping of a bore at a known discharge rate and observing the pressure changes, both in the pumped bore and in the aquifer around it, that this pumping induces. The data obtained from such a test can be analysed to give the characteristics of the bore and the aquifer; these are well efficiency and the transmission and storage properties of the aquifer. It is necessary to know these parameters for the design of bores and well fields.

An ideal test set up might comprise the test bore and up to nine piezometers set in two lines at right angles to each other, each terminating in screened sections set in the main aquifer, approximately opposite the mid-length of the pumped-bore screened section. Optimum distances for the piezometers from the bore are considered to be 5, 15, 50, 175 and 600 metres for the primary row, and for the row at right angles, 9, 30, 100 and 300 metres.

In extreme cases, where for example it might be desired to more fully study storage coefficients or upper layer vertical permeabilities, each piezometer would be replaced by a piezometer pair, one terminating at standard depth and the other just below the shallow water table. At the other extreme, where there were restrictions on piezometer numbers, a minimum of two would be recommended at say 5 and 20 metres from the bore.

In most cases the maximum information of value is obtained from a standard package of pumping and recovery tests carried out in a strictly controlled manner. There are numerous publications dealing with the theory of aquifer tests. This Manual aims at an introduction to the subject and discusses some of the simpler methods of test data interpretation with emphasis on practical application.

The test package would of course include the classic step test described earlier and piezometers would be observed during the steps, not so much for subsequent direct analysis but to give an indication of expected rate and absolute drawdowns in subsequent tests and to indicate the existence of any potential boundary conditions. The step test might also be analysed by say, the Brereton step test analysis method but the main component of the testing package would be a long constant discharge (CD) test. During a long CD test, a bore is pumped at a fixed discharge close to its maximum rated capacity for between 72 and 120 hours; the exact duration is decided on local experience but should be a compromise between the need to save on pumping costs and the requirement that the test be long enough to give the information desired. The latter can only be decided on the basis of a review of how a particular aquifer behaves when pumped. Drawdown is monitored in the bore and in observation piezometers at logarithmically increasing time intervals whilst discharge is maintained constant. Recovery is monitored at similar time intervals.

Details of pump test procedures and measurement are given in Section 4.6.

4.5.2 Basic Analytical Theories

The basis of analytical treatment fo flow of water in aquifers is Darcy's Law. In its simplest form Darcy's Law may be stated as follows:

$$v = K \frac{dh}{dL} \qquad (11)$$

where v = flow velocity

K = constant (coefficient of permeability)

h = hydraulic head

L = length

The term $\frac{dh}{dL}$ is the hydraulic gradient.

Darcy's Law states that the flow in a porous medium is proportional to hydraulic gradient, with the constant of proportionality being a property of the porous medium and known as the coefficient of permeability. Though Darcy's Law has limits of application (it does not apply to high flow velocities), it is the basis of all aquifer test theory.

The basic continuity equation describing the flow of water in an aquifer is derived from the principle of conservation of matter and, in its most general form, can be stated as the well known Laplace equation:

$$K_x \frac{\partial^2 h}{\partial x^2} + K_y \frac{\partial^2 h}{\partial y^2} + K_z \frac{\partial^2 h}{\partial z^2} = V \frac{\partial h}{\partial t} \qquad (12)$$

where h = potential function expressed as hydraulic head above a specified datum

V = a storage parameter

t = time

and K_x, K_y and K_z are the permeabilities in the principal directions x, y and z, of a cartesian co-ordinate system.

Assuming hydraulic isotropy in the horizontal plane it is convenient to express equation (12) in cylindrical co-ordinates in terms r, the radius vector, the angular co-ordinate and z the vertical co-ordinate. This gives:

$$K_r \left[\frac{\partial^2 h}{\partial r^2} + \frac{1}{r} \frac{\partial h}{\partial r} + \frac{1}{r^2} \frac{\partial^2 h}{\partial \Theta^2} \right] + K_z \frac{\partial^2 h}{\partial z^2} = V \frac{\partial h}{\partial t} \qquad (13)$$

For well systems with radial symmetry the term in Θ disappears. Further, in systems where all the flow is horizontal the term in z disappears. Thus equation (13) becomes:

$$K_r \left[\frac{\partial^2 h}{\partial r^2} + \frac{1}{r} \frac{\partial h}{\partial r} \right] = V \frac{\partial h}{\partial t} \qquad (14)$$

Multiplying equation (14) by aquifer thickness D, and substituting drawdown, s for h, we get:

$$\frac{\partial^2 s}{\partial r^2} + \frac{1}{r} \frac{\partial s}{\partial r} = \frac{S}{T} \frac{\partial s}{\partial t} \qquad (15)$$

where S = the storage coefficient

and T = coefficient of transmissivity

Equation (15) is the basis of all commonly used methods of aquifer test interpretation. Some of these are discussed below. There are however several assumptions inherent in the equation, some of which have been mentioned already:

- All the flow is horizontal;
- The flow system has a complete radial symmetry;
- The coefficients of transmissivity and storage of the aquifer are constant at all times and distances.

whilst to obtain usable solutions, further conditions have to be stipulated.

Equilibrium Methods

At equilibrium, as defined in Section 4.1, there are no significant changes in the flow system with time. The last term of equation (15) therefore vanishes and the continuity condition becomes:-

$$\frac{d^2s}{dr^2} + \frac{1}{r}\frac{ds}{dr} = 0 \qquad (16)$$

Solving equation (16) for the boundary conditions that $s = s_w$ for $r = r_w$ (bore drawdown is constant) and $s = 0$ for $r = r_e$ (drawdown is nil at some finite radial distance r_e from the bore), the following expression is obtained, where Q is the bore discharge rate:

$$s_w = \frac{Q}{4 \pi T} \ln \frac{r_e}{r_w} \qquad (17)$$

Converting to base - 10 logarithms, equation (17) becomes:

$$s_w = \frac{2.303\,Q}{4\,\pi\,T} \log \frac{r_e}{r_w} \qquad (4)^*$$

which is usually termed the equilibrium formula of Thiem and which is the same as equation (4), from which the Logan approximate solution was earlier derived in (Section 4.3.2). The Logan method is thus equally applicable to CD tests provided some allowance is made for unknown well losses. This is usually done simply by using a higher Logan coefficient, say 1.32 instead of 1.22.

Other analytical methods have been derived from equation (4), both for bore and for piezometer data; the same boundary restrictions applicable to the Logan method (given earlier, below equation (7)), are observed. In practice, even if a pumped bore has not reached steady-state, equilibrium drawdown can be extrapolated, while well losses can be estimated by step-test and boundary effects can be identified from drawdown curve discontinuities.

The equilibrium formula can be applied to drawdowns in piezometers around a discharging bore. Usually, several drawdowns say, S_1 and S_2, are measured at distances r_1, r_2, from the bore and the equation becomes:

$$s_1 - s_2 = \frac{2.303Q}{2 \pi T} \log \frac{r_2}{r_1} \qquad (18)$$

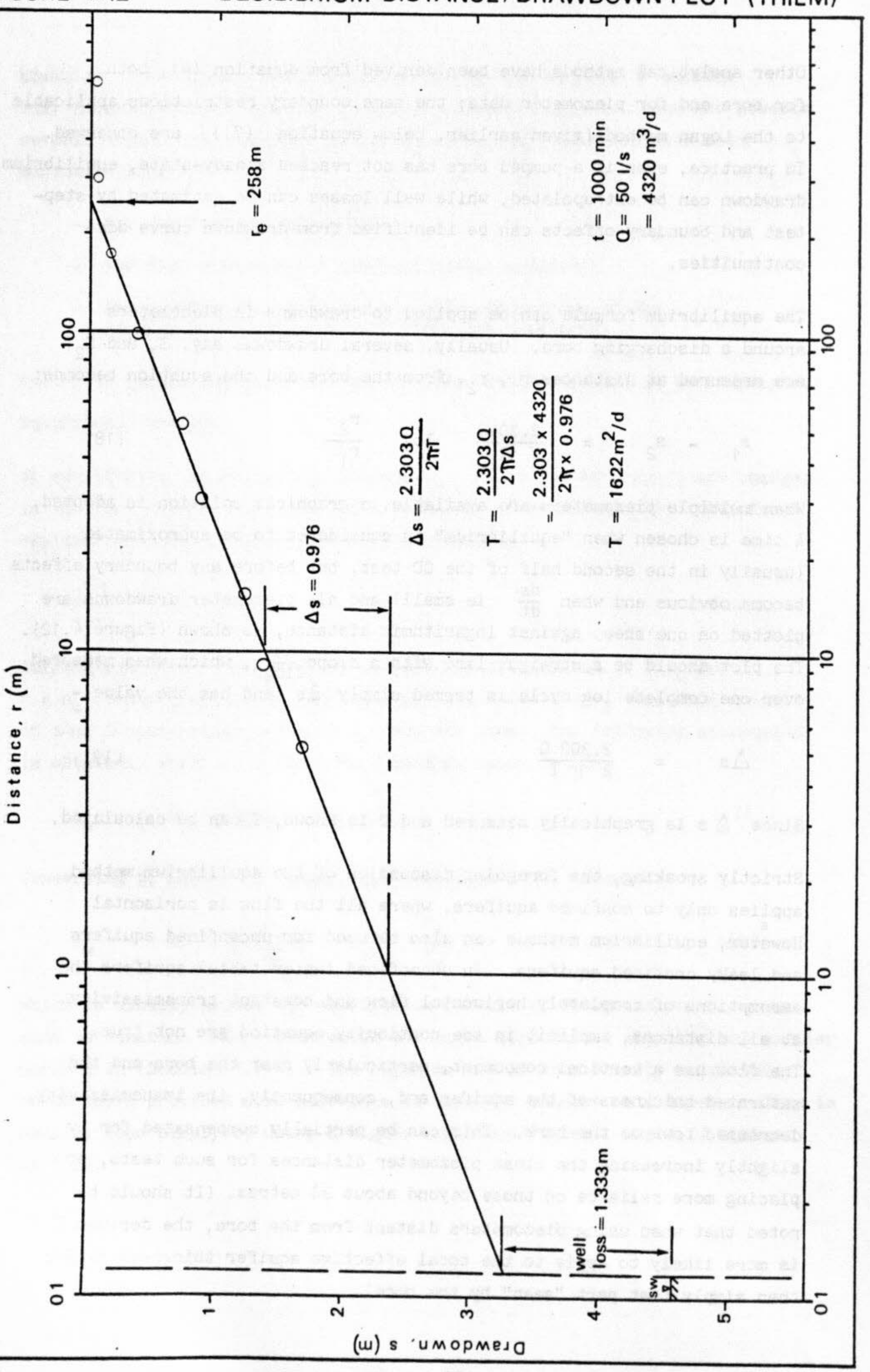
When multiple piezometers are available, a graphical solution is adopted. A time is chosen when "equilibrium" is considered to be approximated (usually in the second half of the CD test, but before any boundary effects become obvious and when $\frac{ds}{dt}$ is small) and all piezometer drawdowns are plotted on one sheet against logarithmic distance, as shown (Figure 4.12). The plot should be a straight line with a slope $\frac{ds}{d\log r}$, which when measured over one complete log cycle is termed simply Δs and has the value:-

$$\Delta s = \frac{2.303\ Q}{2\ \pi\ T} \qquad (19)$$

Since Δs is graphically measured and Q is known, T can be calculated.

Strictly speaking, the foregoing discussion of the equilibrium method applies only to confined aquifers, where all the flow is horizontal. However, equilibrium methods can also be used for unconfined aquifers and leaky confined aquifers. In unconfined (water table) aquifers the assumptions of completely horizontal flow and constant transmissivity at all distances, implicit in the continuity equation are not true. The flow has a vertical component, particularly near the bore and the saturated thickness of the aquifer and, consequently, its transmissivity, decreases towards the bore. This can be partially compensated for by slightly increasing the close piezometer distances for such tests, or placing more reliance on those beyond about 30 metres. (It should be noted that when using piezometers distant from the bore, the derived T is more likely to apply to the total effective aquifer thickness rather than simply that part "seen" by the bore).

FIGURE 4.12 EQUILIBRIUM DISTANCE/DRAWDOWN PLOT (THIEM)



A closer approximation for water table aquifers can be obtained by allowing the Dupuit-Forchheimer assumptions for confined flow and correcting for decreasing saturated thickness. The assumptions are:

- The flow everywhere is horizontal or more correctly vertical gradients are small compared to horizontal gradients and can be neglected;
- The hydraulic gradient to all depths is given by the slope of the water table.

The correction considers the thicknesses of saturation h_1 and h_2 above the impermeable aquifer base at two radii from the pumped bore, respectively r_1 and r_2, with r_2 being large (greater than 200 m). The expression is:

$$h_2^2 - h_1^2 = \frac{2.303Q}{\pi K} \log \frac{r_2}{r_1} \qquad (20)$$

The analysis of the behaviour of leaky aquifers is more complex. If equilibrium analyses fail, more specific variants, such as the Hantush or De Glee methods, should be applied.

Non-equilibrium Methods

All non-equilibrium methods of treatment of groundwater flow towards discharging wells are based on solutions of equation (15) or one of its variants. The simplest and best known method is that due to Theis, who obtained a solution for the following boundary conditions:

$$s = 0 \quad \text{for} \quad t = 0$$

s tends to 0 as r tends to ∞

and, $\lim_{r \to 0}$, $r. \frac{\partial s}{\partial r} = - \frac{Q}{2 \pi T}$ (well is a line sink)

$$\text{Hence, } s = \frac{Q}{4 \pi T} \text{ x } \int_u^\infty \frac{e^{-u}}{u} du = \frac{Q}{4 \pi T} \text{ x } W(u) \qquad (21)$$

Equation (21) is the well known Theis formula. The integral in the equation, which is often denoted as W (u), the well function is not directly integrable, but it can be expressed as the following infinite convergent series:

$$W(u) = - 0.5772 - \ln u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \ldots\ldots \qquad (22)$$

The argument of the function, u, is given by:

$$u = \frac{r^2 s}{4Tt} \qquad (23)$$

where s = storage coefficient of the aquifer in question

Equations (21) and (23) can be written in logarithmic form as follows:

$$\log s = \log \frac{Q}{4\pi T} + \log W(u) \qquad (24)$$

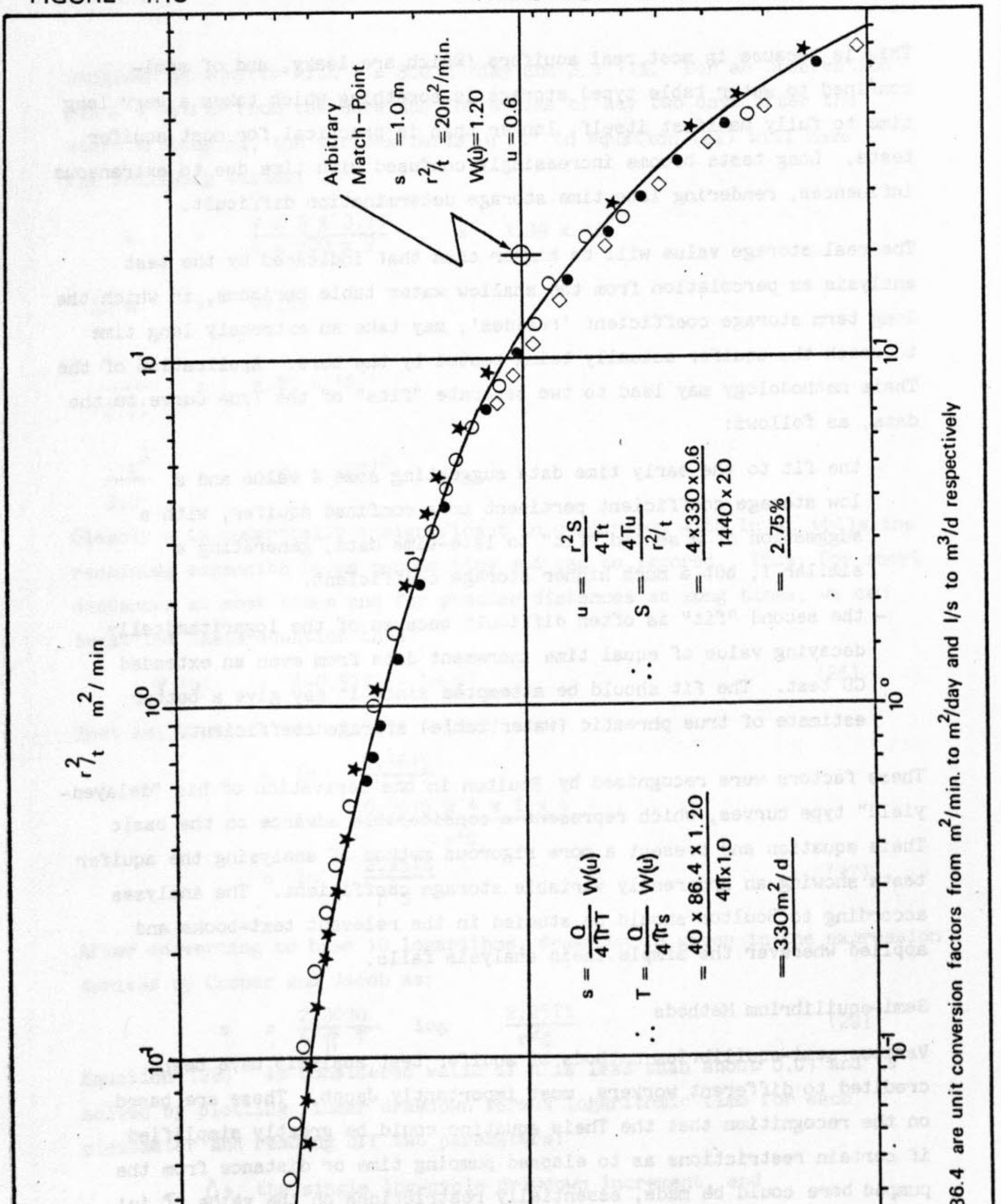
$$\log \frac{r^2}{t} = \log \frac{4T}{s} + \log u \qquad (25)$$

Thus, as can be seen, s and r^2/t are related in a similar manner to W(u) and u. In fact, if plotted on logarithmic paper the two curves have exactly the same shape.

A graphical solution of equations (24) and (25) is therefore possible. A Type Curve of u against W(u) is plotted on double logarithmic graph paper from tables. (Any hydrogeological book will give tabulated values of u and W(u)). A plot of s against r^2/t is then prepared on similar, transparent paper and superimposed on the Type Curve. The co-ordinate axes of the two graphs are held parallel and the data plot transferred to a position representing the best fit between the two curves (Figure 4.13). An arbitrary match point is then selected anywhere on the paper and values of u, W(u), s and r^2/t read off for it. These values are substituted into equations (21) and (23) and T and S calculated, as in the example given.

It should be noted that the Theis equation is the first analysis we have considered which evaluates storage coefficient. This is deliberate since, in spite of what various text-book treatises may infer to the contrary, storage coefficient is an exceedingly difficult parameter to evaluate accurately. Almost all evaluations from aquifer tests, except in the cases of low-storage, highly-confined aquifers, are probably too far low.

FIGURE 4.13 THEIS NON-EQUILIBRIUM ANALYSIS



This is because in most real aquifers (which are leaky, and of semi-confined to water table type) storage is something which takes a very long time to fully manifest itself, longer than is practical for most aquifer tests. Long tests become increasingly confused with time due to extraneous influences, rendering late time storage determination difficult.

The real storage value will be higher than that indicated by the test analysis as percolation from the shallow water table horizons, in which the long term storage coefficient 'resides', may take an extremely long time to reach the aquifer actually being tested by the bore. Application of the Theis methodology may lead to two separate "fits" of the Type Curve to the data, as follows:

- the fit to the early time data suggesting some T value and a low storage coefficient pertinent to a confined aquifer, with a suggestion of a second "fit" to late-time data, generating a similar T, but a much higher storage coefficient.
- the second "fit" is often difficult because of the logarithmically decaying value of equal time increment data from even an extended CD test. The fit should be attempted since it may give a better estimate of true phreatic (water table) storage coefficient.

These factors were recognised by Boulton in the derivation of his "delayed-yield" type curves, which represent a considerable advance on the basic Theis equation and present a more rigorous method of analysing the aquifer tests showing an apparently variable storage coefficient. The analyses according to Boulton should be studied in the relevant text-books and applied wherever the simple Theis analysis fails.

Semi-equilibrium Methods

Various semi-equilibrium methods of aquifer test analysis have been credited to different workers, most importantly Jacob. These are based on the recognition that the Theis equation could be greatly simplified if certain restrictions as to elapsed pumping time or distance from the pumped bore could be made, essentially restrictions on the value of 'u' in the Theis equation. The approximations, sometimes called semi-equilibrium methods since they postulate a constant value for the term $\frac{ds}{dslogt}$, rely on only treating data from very small values of 'u'; that is, from small radii (such as the bore itself) or from piezometers only after very long times.

Consider an aquifer with T = 800 m^2/day and S = 11%. For an observation point 9 metres from the bore and for a time of say two days after the start of pumping, the various terms in 'u' in equation (22) will have the following values:

$$u = \frac{9 \times 9 \times 0.11}{4 \times 800 \times 2} = 1.39 \times 10^{-3}$$

$$\ln u = -6.58$$

$$\frac{u^2}{2.2!} = 4.85 \times 10^{-7}$$

$$\frac{u^3}{3.3'} = 1.50 \times 10^{-10}$$

Clearly u is numerically insignificant in comparison with ln u, while the remaining expansion terms become tiny and can be ignored. Thus, for short distances at most times and for greater distances at long times, we can treat the Theis equation thus:

$$W(u) = (-0.5772 - \ln u) \qquad (26)$$

$$\begin{aligned} \text{That is, } W(u) &= (\ln 0.5615 - \ln u) \\ &= \ln \frac{0.5615}{u} \\ &= \ln \frac{0.5615 \times 4 \times T \times t}{r^2 S} \\ &= \ln \frac{2.25Tt}{r^2 S} \qquad (27) \end{aligned}$$

After converting to base 10 logarithms, drawdown is given in the expression derived by Cooper and Jacob as:

$$s = \frac{2.303Q}{4 \pi T} \log \frac{2.25Tt}{r^2 S} \qquad (28)$$

Equation (28) is considered valid if n is less than about 0.01 and is solved by plotting linear drawdown versus logarithmic time for each piezometer and reading off two parameters:

- Δs, the single log-cycle drawdown increment, and
- t_o, the time when the straight-line portion of the plot, extrapolated backwards, cuts the zero-drawdown axis.

Since $s = 0$ at $t = t_o$ and the $2.303Q/4\pi T$ term cannot be zero, then the log term must be zero and its argument must have a numerical value of 1.0 (log 1 = 0) . Thus:

$$\frac{2.25\ Tt_o}{r^2S} = 1$$

$$\text{or,} \qquad S = \frac{2.25Tt_o}{r^2} \qquad (29)$$

The value of T to be used in solving equation (29) is derived from the consideration that drawdown must be numerically equal to Δs at a time 10 x t_o, when the log term in equation (28) has to have a numerical value of 1.0. Thus:

$$s = \frac{2.303Q}{4\pi T} \times 1$$

$$\text{or,} \qquad T = \frac{2.303Q}{4\pi\Delta s} \qquad (30)$$

This type of solution is illustrated (Figure 4.14). The short, late-time section of the data to which the analysis is applicable (the approximation is valid) can clearly be seen.

Theis' own application of the approximation was to the recovery of groundwater levels around a discharging bore on the cessation of pumping. He stipulated that the recovery can be simulated by assuming that the bore continues discharging but superimposing a recharge bore of equal yield on top of it. Stated analytically this gives:

$$s^1 = \frac{Q}{4\pi T} W(u) - \frac{Q}{4\pi T} W(u^1) \qquad (31)$$

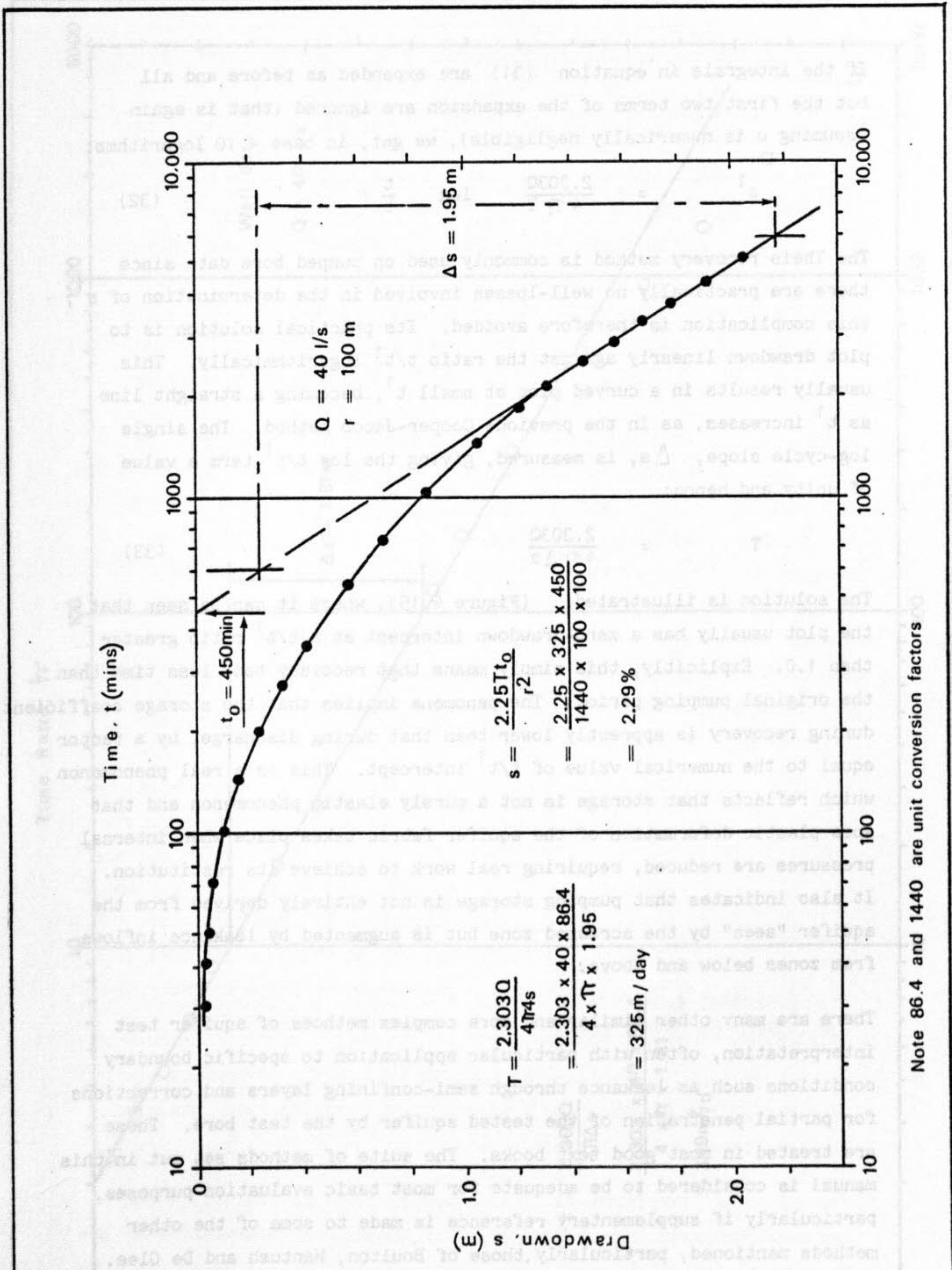
where s^1 = residual drawdown

$u = \frac{r^2S}{4Tt}$, $u^1 = \frac{r^2S}{4Tt'}$

t = time since pumping started

t^1 = time since pumping stopped.

FIGURE 4.14 SEMI-EQUILIBRIUM ANALYSIS (COOPER—JACOB)



$$T = \frac{2.303Q}{4\pi\Delta s} = \frac{2.303 \times 40 \times 86.4}{4 \times \pi \times 1.95} = 325\text{m/day}$$

$$s = \frac{2.25Tt_o}{r^2} = \frac{2.25 \times 325 \times 450}{1440 \times 100 \times 100} = 2.29\%$$

Note 86.4 and 1440 are unit conversion factors

If the integrals in equation (31) are expanded as before and all but the first two terms of the expansion are ignored (that is again assuming u is numerically negligible), we get, in base - 10 logarithms:

$$s^1 = \frac{2.303Q}{4 \pi T} \log \frac{t}{t} 1 \qquad (32)$$

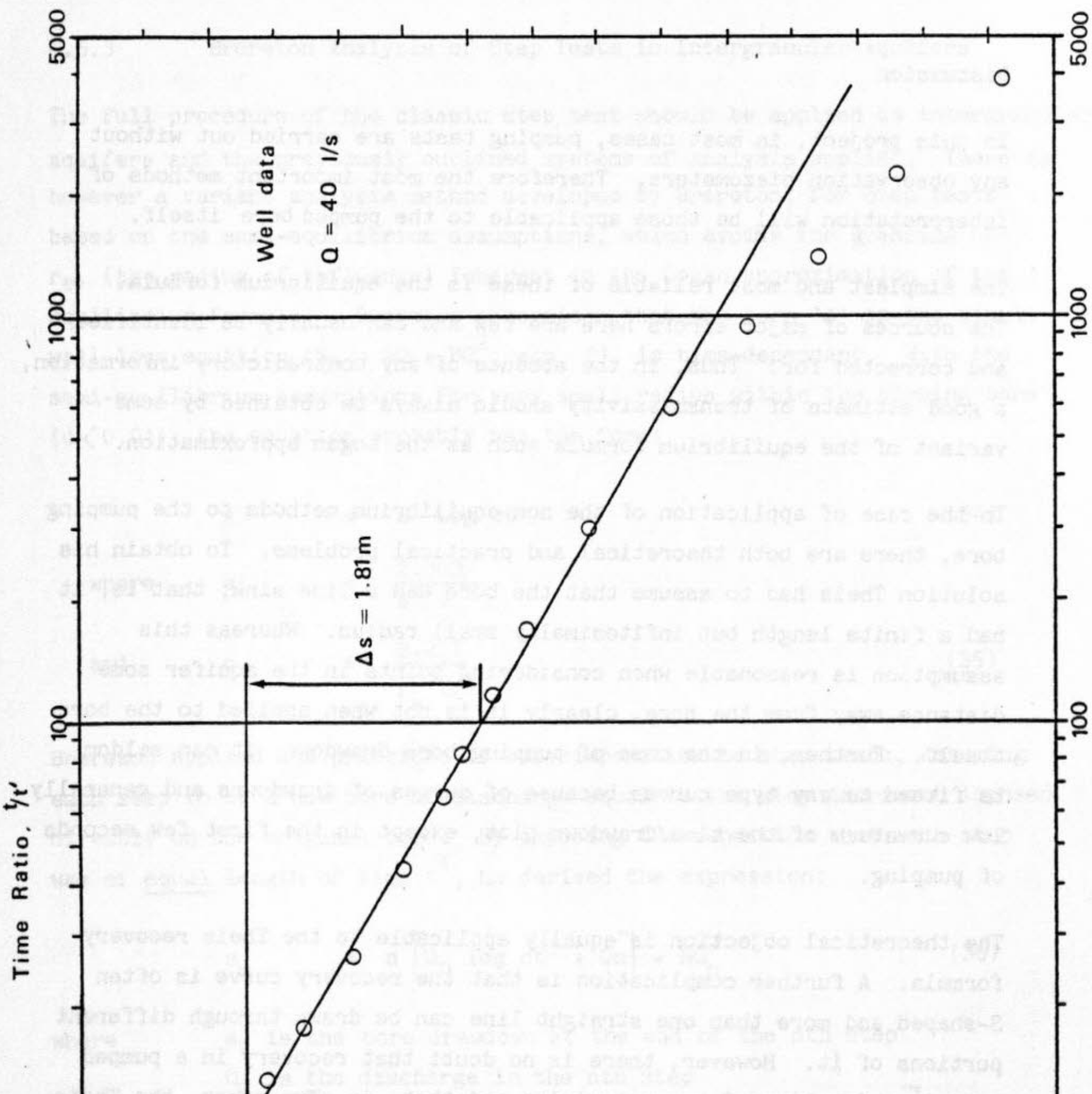
The Theis recovery method is commonly used on pumped bore data since there are practically no well-losses involved in the determination of s^1, this complication is therefore avoided. Its practical solution is to plot drawdown linearly against the ratio t/t^1 logarithmically. This usually results in a curved plot at small t^1, becoming a straight line as t^1 increases, as in the previous Cooper-Jacob method. The single log-cycle slope, Δs, is measured, giving the log t/t^1 term a value of unity and hence:

$$T = \frac{2.303Q}{4 \pi \Delta s} \qquad (33)$$

The solution is illustrated (Figure 4.15), where it can be seen that the plot usually has a zero-drawdown intercept at a t/t^1 ratio greater than 1.0. Explicitly, this simply means that recovery took less time than the original pumping period. The penomena implies that the storage coefficient during recovery is apprently lower than that during discharge, by a factor equal to the numerical value of t/t^1 intercept. This is a real phenomenon which reflects that storage is not a purely elastic phenomenon and that some plastic deformation of the aquifer fabric takes place when internal pressures are reduced, requiring real work to achieve its restitution. It also indicates that pumping storage is not entirely derived from the aquifer "seen" by the screened zone but is augmented by leakance inflows from zones below and above.

There are many other similar and more complex methods of aquifer test interpretation, often with particular application to specific boundary conditions such as leakance through semi-confining layers and corrections for partial penetration of the tested aquifer by the test bore. These are treated in most good text books. The suite of methods set out in this manual is considered to be adequate for most basic evaluation purposes, particularly if supplementary reference is made to some of the other methods mentioned, particularly those of Boulton, Hantush and De Glee.

FIGURE 4.15 THEIS RECOVERY ANALYSIS



Discussion

In this project, in most cases, pumping tests are carried out without any observation piezometers. Therefore the most important methods of interpretation will be those applicable to the pumped bore itself.

The simplest and most reliable of these is the equilibrium formula. The sources of major errors here are few and can usually be identified and corrected for. Thus, in the absence of any contradictory information, a good estimate of transmissivity should always be obtained by some variant of the equilibrium formula such as the Logan approximation.

In the case of application of the non-equilibrium methods to the pumping bore, there are both theoretical and practical problems. To obtain his solution Theis had to assume that the bore was a line sink; that is, it had a finite length but infitesimally small radius. Whereas this assumption is reasonable when considering points in the aquifer some distance away from the bore, clearly it is not when applied to the bore itself. Further, in the case of pumping bore drawdown, it can seldom be fitted to any type curves because of surges of drawdowns and generally low curvature of the time/drawdown plot, except in the first few seconds of pumping.

The theoretical objection is equally applicable to the Theis recovery formula. A further complication is that the recovery curve is often S-shaped and more than one straight line can be drawn through different portions of it. However, there is no doubt that recovery in a pumped bore is a function of transmissivity and that, in some cases, the Theis recovery formula works well. Nevertheless, in the case of conflict of results obtained by the equilibrium method and by Theis recovery method, the former (the Logan approximation) should be preferred.

Neither of these two methods allows the calculation of the storage coefficient. This parameter is difficult to estimate accurately and requires observation piezometer data.

The step test should always be analysed by plotting specific drawdown against discharge. The resulting graph should look something like the plot given earlier (Figure 4.8). Well loss can be calculated from the slope of the line. Further, a formula for the well loss can be derived and drawdowns can be corrected for it before using them for calculation of aquifer constants.

4.5.3 Brereton Analysis of Step Tests in Intergranular Aquifers

The full procedure of the classic step test should be applied to intergranular aquifers and the previously outlined systems of analysis applied. There is however a variant analysis method developed by Brereton, for step tests based on the semi-equilibrium assumptions, which avoids the guessing of r_e, (the radius of influence) inherent in the Logan approximation of the equilibrium formula. Brereton recognises that the term 'A' in the simple well-loss equation ($S_W = AQ + BQ^2$; eqn 2) is time-dependant. With the semi-equilibrium assumptions for very small radius within the pumping bore ($u < 0.01$), the equation probably has the form:

$$A = a \log ct \tag{34}$$

$$\text{where} \quad a = \frac{2.303}{4\pi T}$$

$$\text{and} \quad c = \frac{2.25T}{r_w^2 S} \tag{35}$$

Brereton applied the principle of superimposition to step tests, assuming each step to be a new bore of discharge equal to the step increment imposed directly on the original bore. By imposing the condition that each step was of equal length of time t^1, he derived the expression:

$$s_n = a\left[Q_n \log ct^1 + Dn\right] + BQ_n^2 \tag{36}$$

where s_n is the bore drawdown at the end of the nth step

Q_n is the discharge in the nth step

and $D_n = \left[Q_{n-1} \log 2 + Q_{n-2} \log \frac{3}{2} + \text{---} + Q_1 \log \frac{n}{n-1}\right]$ (37)

He also showed that for step test data plotted time-logarithimically from an origin at the beginning of each step, the log-cycle slope, Δs_n could be expressed, for a 4th step in a test as:

$$\Delta s_4 = a\left[Q_4 -0.76Q_3 - 0.104Q_2 - 0.041Q_1\right] \tag{38}$$

and for a 3rd step:

$$\Delta s_3 = a\left[Q_3 -0.76Q_2 - 0.104Q_1\right] \tag{39}$$

or generally:

$$\Delta s_n = a\,f(Q) \tag{40}$$

It is hence possible to evaluate 'a', and hence T, either individually from each step by using the pertinent form of equation (38) or (39) etc. or else overall, by plotting Δs_n against the f(Q) values for all steps; the slope of the plot should have the average a value, $\bar{a}$. These processes are illustrated (Figure 4.16). They are obviously of value in cases of varying T as permeable section is dewatered. The other parameters B, c (and hence A), can be calculated, but only overall and for constant T, from a manipulation of equation (36). This can be written as:

$$\frac{s_n - \bar{a}\, D_n}{Qn} = \bar{a} \ \log ct^1 + BQ_n \qquad (41)$$

Its application is clearly much simpler if the value of the first step Q_1 is numerically equal to the increments ΔQ between successive steps, so that:

$$D_n = \Delta Q \left[\log 2 + \log \frac{3}{2} + \text{---} \log \frac{n}{n-1} \right] \qquad (42)$$

With this restriction, the D term will have the following values:

$D_1 = 0$

$D_2 = 0.301\,\Delta Q = \Delta Q\ (\log 2)$

$D_3 = 0.477\,\Delta Q = \Delta Q\ (\log 2 + \log 3/2)$

$D_4 = 0.602\,\Delta Q$ etc.

$D_5 = 0.699\,\Delta Q$

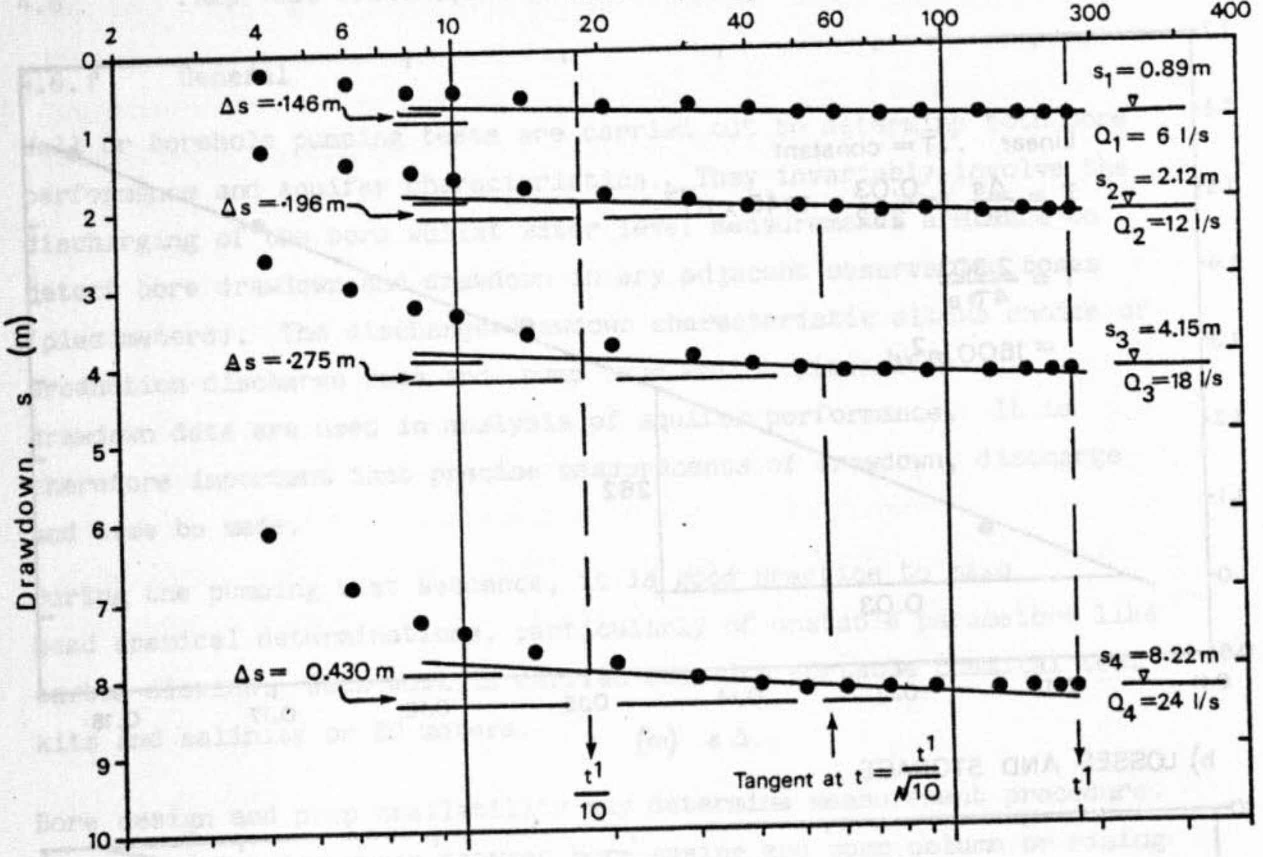
A linear plot of successive values of the left hand side of equation (41) against Q_n will give (for constant T), a straight line with an intercept of $\bar{a}$ log ct^1 (the well coefficient A) and a slope of B. Knowing $\bar{a}$, the value of c can be determined. This is illustrated (Figure 4.17).

If T does vary from step to step, trial and error solutions should be possible for an unknown and possibly varying B, since the variation in c must be directly proportional to the variation in T, provided storage coefficient remains constant, as it should for short times.

Comparison of Figure 4.16 with Figure 4.8 indicates how the rigorous solution compares with the approximate; data for Figure 4.16 is for a repeat test on the same bore slightly lower discharges.

FIGURE 4.16 BRERETON TRANSMISSIVITY ANALYSIS

a) INDIVIDUAL STEPS



$f(Q_1) = (6 \times 86.4) = 518.4 \therefore a_1 = \frac{\Delta s_1}{f(Q_1)} = \frac{0.146}{518.4} = 2.82 \times 10^{-4} \quad \therefore T_1 = 650 \ m^2/d$

$f(Q_2) = ([12 - 0.76 \times 6]\,86.4) = 642.8 \therefore a_2 = \frac{0.196}{642.8} = \quad \therefore T_2 = 600 \ m^2/d$

$f(Q_3) = ([18 - 0.76 \times 12 - 0.104 \times 6]\,86.4) = 713.3 \therefore a_3 = \frac{0.275}{713.3} \quad \therefore T_3 = 475 \ m^2/d$

$f(Q_4) = ([24 - 0.76 \times 18 - 0.104 \times 12 - 0.041 \times 6]\,86.4) = 762.6 \therefore a_4 = \frac{0.430}{762.6} \quad \therefore T_4 = 325 \ m^2/d$

b) COLLECTIVELY

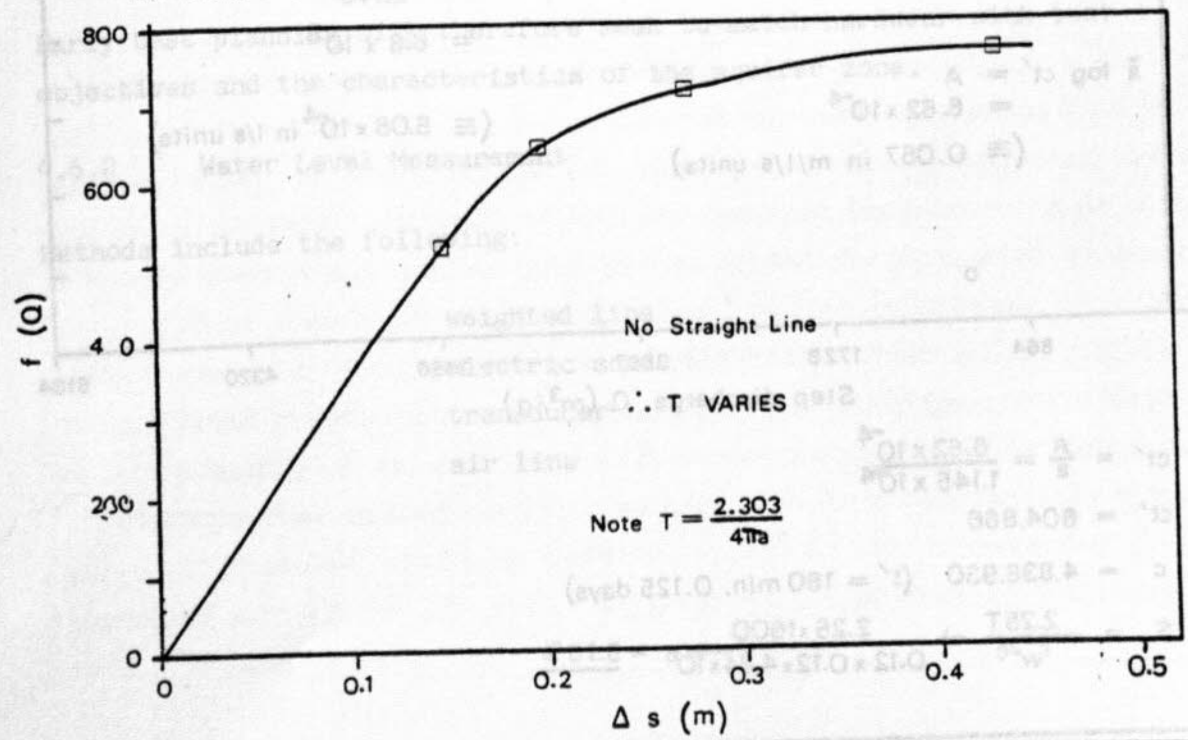
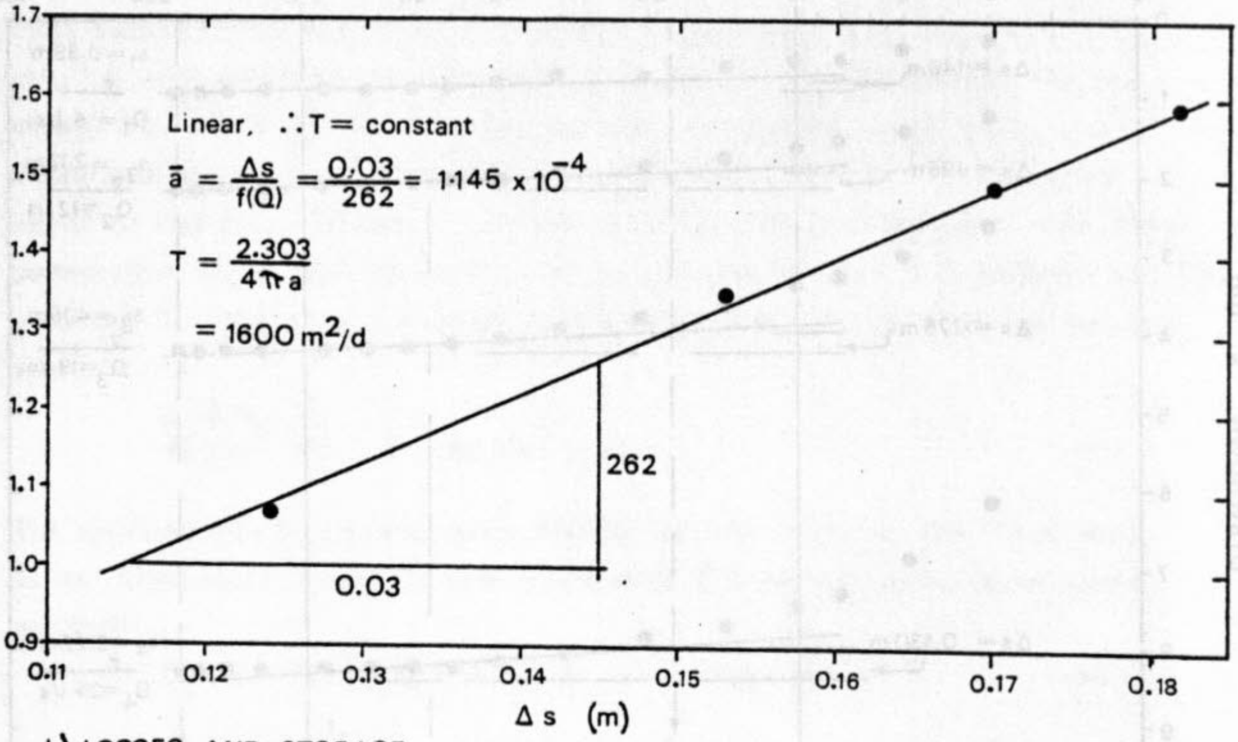


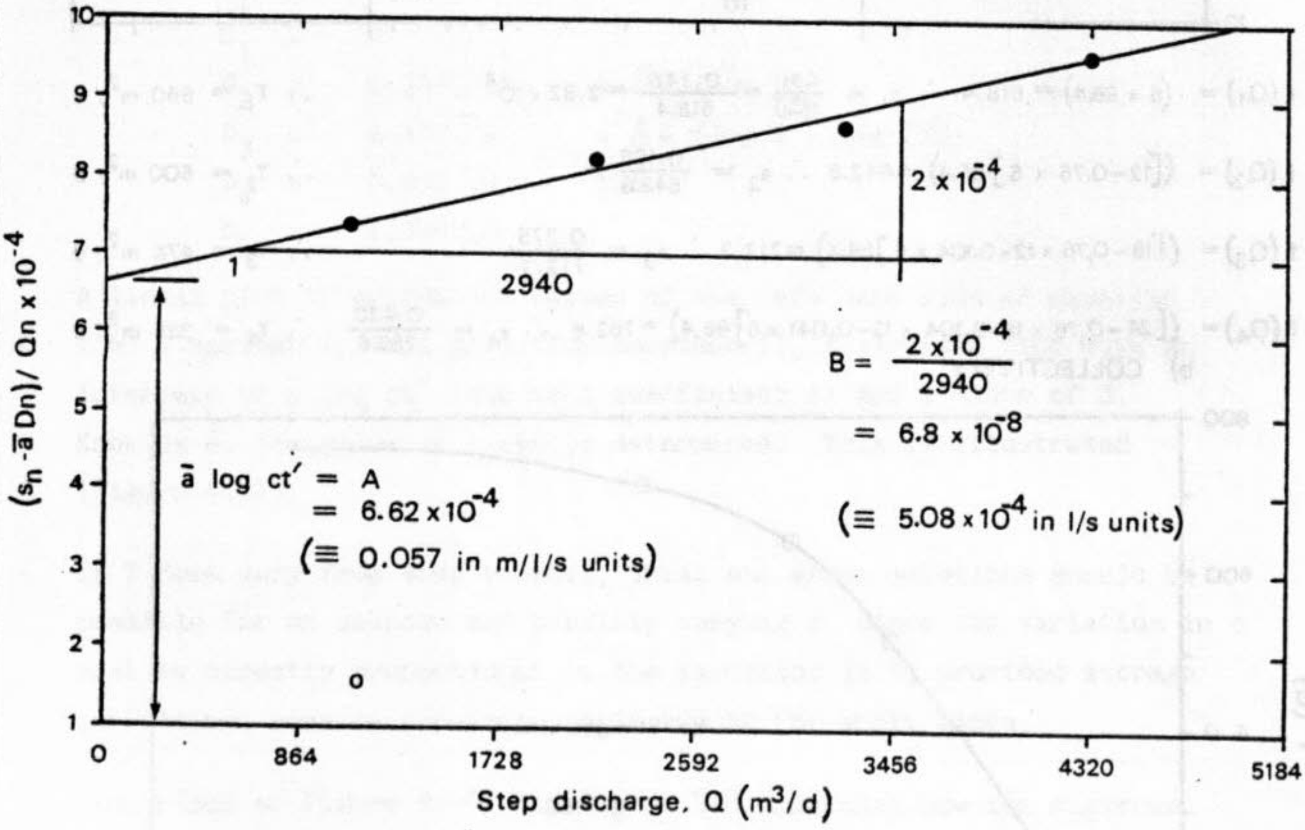
FIGURE 4.17 BRERETON WELL LOSS ANALYSIS

a) OVERALL TRANSMISSIVITY

Note: use consistent units, ie Q in m^3/day and t in days



b) LOSSES AND STORAGE



$\log\ ct' = \frac{A}{a} = \frac{6.62 \times 10^{-4}}{1.145 \times 10^{4}}$

$\therefore\ ct' = 604{,}866$

$\therefore\ c = 4{,}838{,}930$ (t' = 180 min, 0.125 days)

$S = \frac{2.25T}{r_w^2 c} = \frac{2.25 \times 1600}{0.12 \times 0.12 \times 4.84 \times 10^6} = \underline{5.16\%}$

4.6 Pump Test Procedure

4.6.1 General

Well or borehole pumping tests are carried out to determine both bore performance and aquifer characteristics. They invariably involve the discharging of the bore whilst water level measurements are made to detect bore drawdown and drawdown in any adjacent observation bores (piezometers). The discharge-drawdown characteristic allows choice of production discharge rate and pump type whilst piezometer drawdown data are used in analysis of aquifer performance. It is therefore important that precise measurements of drawdown, discharge and time be made.

During the pumping test sequence, it is good practice to make head chemical determinations, particularly of unstable parameters like carbon dioxide. Such work is carried out with portable chemical test kits and salinity or EC meters.

Bore design and pump availability may determine measurement procedure. Restricted annular space between bore casing and pump column or rising main will, for example, mean that water levels will have to be measured by an airline system. The volume of test discharge and the discharge variation may further determine the type of discharge measurement device to be used. Aquifer characteristics (e.g. permeability, confining beds) may determine choice of test duration and position of piezometers.

Early test planning will therefore seek to match hardwear with test objectives and the characteristics of the aquifer zone.

4.6.2 Water Level Measurement

Methods include the following:

- weighted line
- electric sonde
- transducer
- air line

The contact of a weighted line with a water surface can be seen for example, by illumination using the sun's reflection through a mirror. Otherwise the line can be chalked and the water level measured from the line wetting point. These simple methods are rather slow and hence unsuitable for pumping tests with rapidly changing water levels.

More positive and rapid indications can be obtained by use of the electric sonde. The sonde is attached to a graduated insulated plastic or fibreglass tape which contains the conductor wires. Sonde contact with the water surface makes a circuit, indicated either by a bulb or some audible device. Typical commercially available sondes have diameters of 10-15 mm. Electric sondes give reliable measurements to better that ±1 cm or ±0.4 inch except where water cascades into a pumping bore from upper evacuated zones or where leaks occur in the pump column. Measurements will also be difficult and imprecise where oil rests upon the bore water surface (e.g. following maintenance of an oil lubricated line shaft pump or after the use of the D.H.H. drilling method) or the sonde itself has become oily or dirty. Intermittent and spurious readings may also occur where conductors in the graduated tape are damaged. This commonly occurs where the tape is pulled across the often jagged top edge of the bore casing; in such cases the tape should be supported manually or run over a protective collar of plastic or cloth.

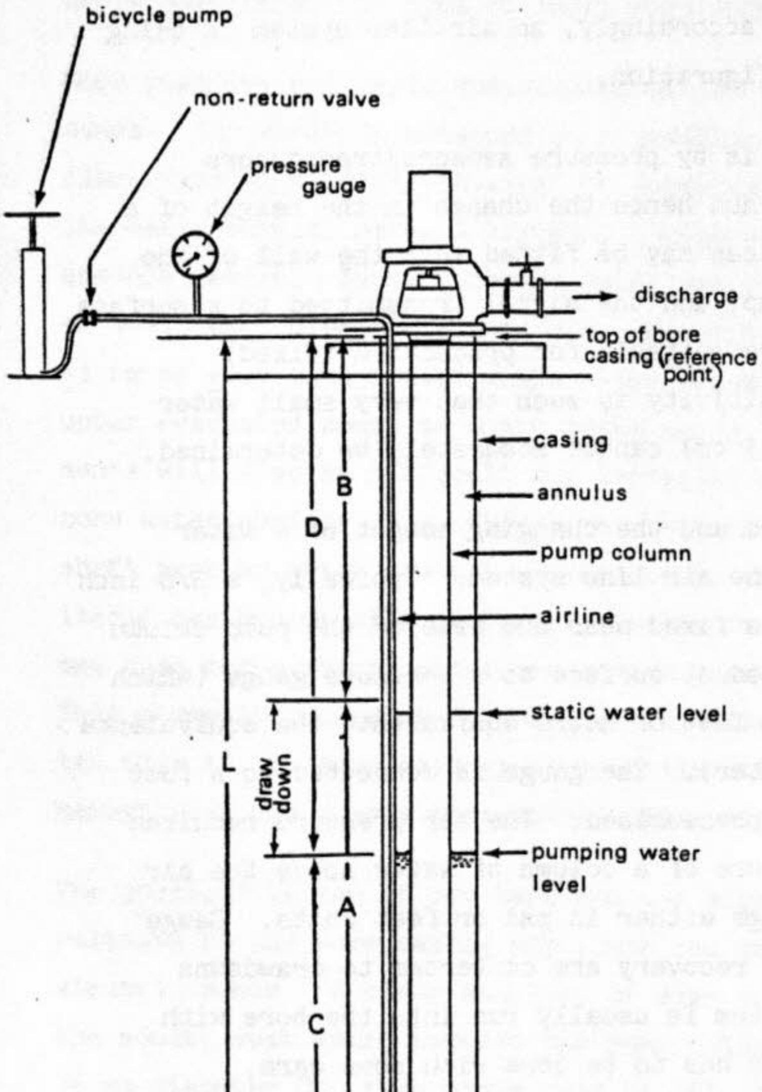
The physical design of the bore and the size of borehole pump column relative to the bore casing may limit the use of both weighted line and electric sonde. A clear annulus, of size sufficient to freely admit the sonde, must exist between the pump column and bore casing. For a 15 mm diameter Ott type sonde used by JPT, a 50 mm annulus is considered the minimum necessary after allowance for any power cable from an electric submersible pump (i.e. casing internal diameter to be 100 mm greater than the maximum pump column outside diameter). Evidently casing-pump column centricity need to be high and any power cable positively fixed to the column. Even under ideal conditions, sonde ingress may become difficult at large operating depths (>100 m say) and water levels will have to be measured with an air line. Tape stretch and hence measurement error will also occur at large depths although this can be corrected using manufacturer's stretch factors.

The small JPT test pump (BP 45-4 Grundfos line shaft) has a maximum diameter of 5.1 inch (129 mm) over the collars of the pump column and the pump is being operated within 6 inch i.d. (152 mm) diameter bore casings; the resultant 0.9 inch (23 mm) annulus is quite insufficient to allow free sonde access and accordingly, an air line system is being used with this pump-casing configuration.

A further means of measurement is by pressure sensors/transducers which measure pressure change (and hence the change in the height of a water column). Transducer devices may be fitted into the wall of the pump column, just above the pump, and the signal transmitted to a surface recorder. Such devices are more suitable for production (fixed) installations whilst their sensitivity is such that very small water level or drawdown changes (say 1 cm) cannot accurately be determined.

The equivalence between pressure and the changing height of a water column in the bore is used in the air line system. Typically, a 3/8 inch o.d. polythene or copper tube is fixed near the base of the pump column at a known position and connected at surface to a pressure gauge (which can in addition be graduated in feet or metre equivalent; the equivalence is psi x 2.31 = feet head of water). The gauge is connected to a foot pump via a valve, and the line pressurised. The air pressure required to balance the equivalent pressure of a column of water above the air line base, is read from the gauge either in psi or feet units. Gauge changes during bore pumping and recovery are converted to drawdowns (Figure 4.18). An air line system is usually run into the bore with the pump column assembly. This has to be done with some care, particularly where the annulus is small or otherwise restricted, to avoid puncture or crimping of the air line and leaks in the system. Errors in the use of the system are largely associated with the pressure gauge. Commercially available gauges contain either of two types of pressure sensitive elements, a diaphragm (for higher pressures) or a Bourdon tube (for lower pressure duty). The Bourdon gauge is of higher sensitivity and is to be preferred for pump test work typically involving rather small pressure changes. Even with a sensitive gauge with thin pointer, a large gauge diameter and fine calibrations, it is doubtful if gauge readings to better than about ±2 inch can be made. A pressure gauge (6" diameter, graduated 0-150 feet or 0-65 psi increments) currently used by the JPT pump test group cannot be read to better that about ±6" head of water.

FIGURE 4.18 AIRLINE SYSTEM



Note on use:

L is length of airline

Airline submergence before pumping (A) is gauge reading (p1) in psi or feet

Static water level = L - A

Airline submergence during pumping (C) is gauge reading (p2) in psi or feet

Pumping water level = L - C

Drawdown = A - C
or p1 - p2

Conversions:

psi x 2.31 = feet
or kPa x Q.102 = metres

Low reading accuracy makes the air line system inferior to the electric sonde for water level measurement in pump tests. It is used only where annular size precludes sonde access or in production bore installations where it acts as a low water level (or pump protection) warning system.

4.6.3 Discharge Measurement

The following methods are briefly discussed:

- bucket and stop watch
- flow meters
- weirs

Low and constant (non-pulsating flows) can readily be determined by timing the filling of a bucket or oil drum but the method gives erroneous results where pump discharges are high or pulsed. A typical violent pulsed flow is produced by air lift test pumping, used in approximate bore yield tests.

Direct reading flow meters, coupled in the discharge line, are mainly used in constant discharge production installations. They cannot easily be used where controlled and rapid flow variation and measurement is needed (i.e. step discharge tests).

Discharge is most commonly measured by means of weirs and for pump testing work, portable weirs are preferred. Useful types are:

- portable weir tanks with rectangular or V notch
- portable orifice weirs

In a weir tank, flows are derived by measuring head at a V notch or rectangular weir; measurements are typically made with a vertical steel rule at a point equal to at least one weir height upflow of the weir. Accuracy can only be obtained if flow at the wier face is smooth and non-pulsating. Turbulence in the weir tank must be minimised by use of baffles and by the choice of tank size appropriate to the discharge to be measured. A portable baffled weir tank currently used by JPT is about 880 litres in volume and can measure flows of about 6 l/s without excessive turbulence problems. At very high discharges, the excessive size required of weir tanks makes their use difficult. However, their main disadvantage for pump testing work is that, because of long response time, the rapid and known discharge changes used in step tests cannot be made. In addition, small discharge changes during a constant discharge test (perhaps because of increasing pumping heads or engine malfunction) cannot readily be noticed and rectified.

The orifice pipe weir is preferred for all pump testing work. Pressure drops (or head drop) across the circular weir are measured by means of a piezometer tube whose zero point is set at the orifice tube diameter. The piezometer measurements are then converted to flows using an equation for geometry of the particular weir (Figure 4.19). Provided that the indicated construction and set up conditions are observed, it has been shown that the orifice can measure discharges to within 2% of the true value.

Piezometer head, and hence discharge, can be closely controlled and rapidly varied, by means of a gate valve and pump engine speed; this makes the system ideal for step discharge testing where instantaneous discharge increases to a new step discharge are needed. The piezometer water level is highly visible and an operator can quickly detect unwanted discharge fluctuations. Although the meniscus in the piezometer tube will usually fluctuate over about 1 cm, it is considered that discharge differences of 0.15-0.25 l/s can be reliably measured with the JPT orifices (Table 4.2).

Table 4.2

Orifice Weirs

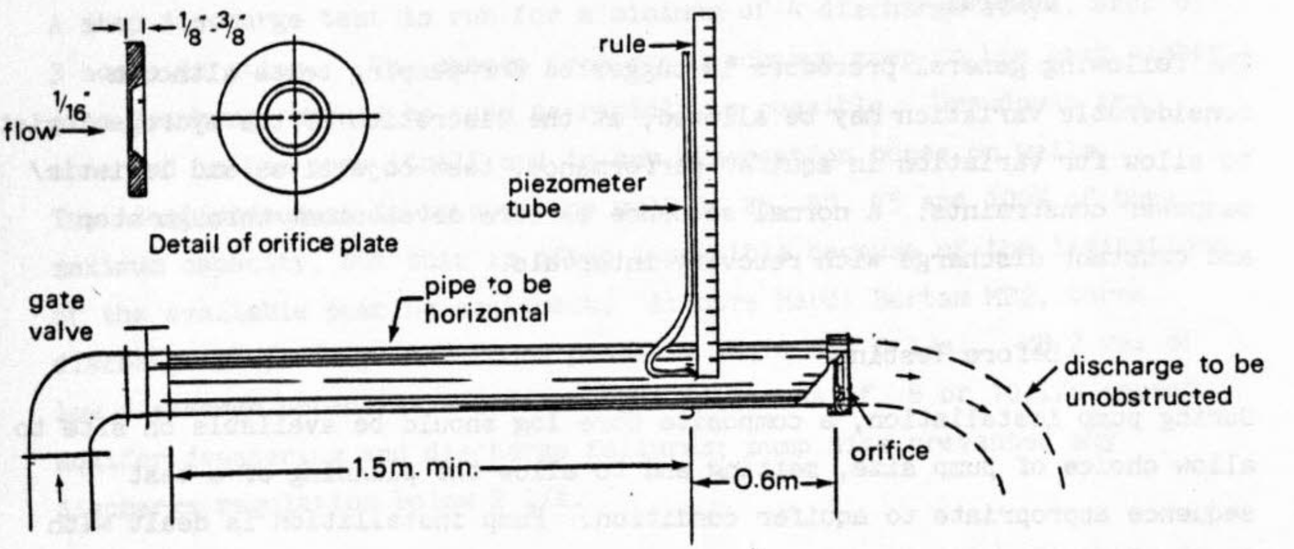
Orifice pipe internal diameter (inch)	Orifice diameter (inch)	Practical discharge range (l/s)
6	3	4 - 16
	4	8 - 31
	5	16 - 62
4	2.25	2.4 - 9
	2.5	3 - 12
	3	5 - 20

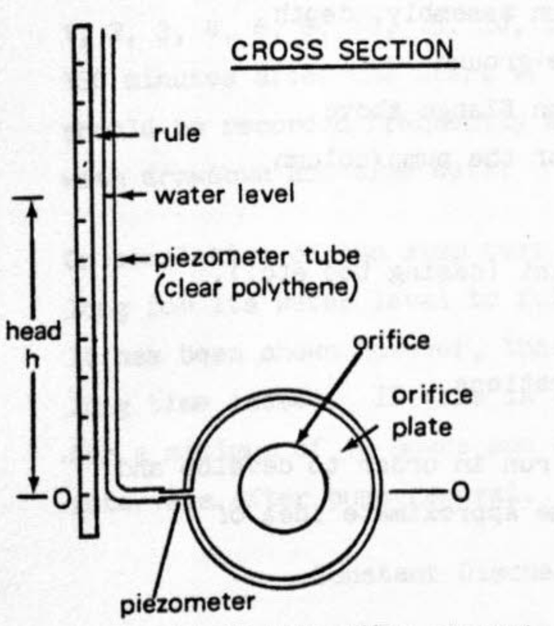
Note: - Maximum piezometer head 150 cm: heads in excess of this are difficult to measure

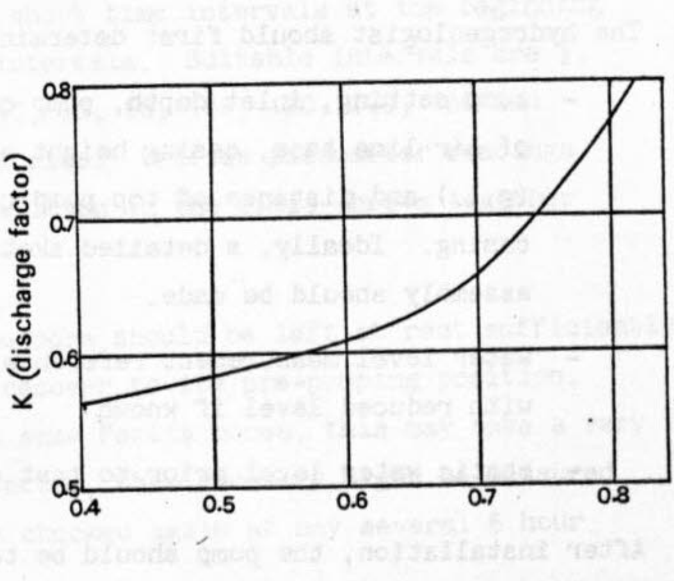
- Minimum piezometer head 10 cm: heads less than this unstable.

FIGURE 4.19

ORIFICE WEIR SYSTEM







Equation of flow in fps units is:

$$Q = 8.02\ KA\sqrt{h}$$

where Q is flow rate in US gpm

A is orifice area in sq.inches

h is head in piezometer. in inches

K is a discharge factor; derived from orifice geometry

Note on Construction/Operation

Orifice pipe to be horizontal; piezometer tapping to be on radius and internally flush. Critical distances between orifice, piezometer and any valves to be maintained. Orifice diameter should be less than 0.8 of the orifice pipe i.d. At low piezometer heads (h almost equal to orifice pipe radius) flow and head become unstable or pulsating. Further head reduction leads to partially filled orifice tube; measurement then impossible.

4.6.4 Standard Test Procedure

General

The following general procedure is suggested for pumping tests although considerable variation may be allowed, at the discretion of the hydrogeologist, to allow for variation in aquifer performance, test objectives and logistic/manpower constraints. A normal sequence is bore development through step and constant discharge with recovery intervals.

Before Testing

During pump installation, a composite bore log should be available on site to allow choice of pump size, setting and to allow the planning of a test sequence appropriate to aquifer condition. Pump installation is dealt with in the Drilling Manual.

The hydrogeologist should first determine and record the following data:

- pump setting, inlet depth, pump column assembly, depth of air line base, casing height above ground level (g.l.) and distance of top pump column flange above casing. Ideally, a detailed sketch of the pump/column assembly should be made.
- water level measurement reference point (casing top etc.), with reduced level if known
- static water level prior to test operations.

After installation, the pump should be test run in order to develop and clean the bore and subsequently to allow some approximate idea of discharge-drawdown behaviour.

Preferred procedure is to run the pump at increasing discharges up to the maximum, and to observe drawdowns. In some bores (e.g. Mata Ayer), the discharge water is initially silt laden and a prolonged period (up to several days) of bore development will be needed. Generally, development pumping should usually start at a low discharge with progressive small discharge increases as water clears at the previous discharge. If possible, the pump should <u>not</u> be stopped where turbid, silt laden water is being pumped. When clear bore water is produced at the maximum feasible discharge then development can stop. After development, water level recovery should be monitored.

Step Test

A step discharge test is run for a minimum of 4 discharge steps, each of 3 hours duration. The change from one discharge step to the next higher step discharge should be made as rapidly as possible. Drawdowns are observed in the bore itself and in any observation bores or wells. The ideal procedure is to pump the well at 25, 50, 75 and 100% of bore maximum capacity, but this is often impossible because of the limitations of the available pumping equipment. At bore Mardi Bertam MB2, three discharge steps only were found possible (2, 4 and 6 l/s). MB 2 was of low discharge potential and discharges in excess of 8 or 10 l/s caused aquifer dewatering and discharge failures; pump size prevented any discharge regulation below 2 l/s.

Drawdowns are measured in the bore during the step test at fixed time intervals, usually comprising very short time intervals at the beginning of each step and gradually longer intervals. Suitable intervals are ½, 1, 2, 3, 4, 6, 8, 10, 15, 20, 30, 40, 60, 80, 100, 120, 140, 160 and 180 minutes after the start of each step. Orifice piezometer readings should be recorded frequently and written in the field sheets together with drawdown and time data.

On completion of the step test, the bore should be left at rest sufficiently long for its water level to fully recover to its pre-pumping position. It has been shown however, that in some Perlis bores, this may take a very long time indeed. If this is suspected, then recovery might be measured for a maximum of 12 hours and then checked again at say several 6 hour intervals after pump removal.

Constant Discharge Test

Following recovery from step test, the bore should be pumped at a fixed, constant discharge which is usually about 75% of the maximum discharge used in the step test. However in the case of the crack rock aquifers at Bertam and Padang Melangit, it may only be possible to pump at 50% or less of the step test maximum since pumping levels induced by long (24-48 hr) pumping may decline below major transmissive cracks and cause discharge lowering and eventual failure.

The test is normally run for 48 to 72 hours but the exact duration is normally decided on the basis of local experience. Drawdown measurements in the pumped bore and any observation wells are made at logarithmically increasing time intervals starting from half a minute. The following would be suitable:

2 measurements at ½ minute intervals
5 " " 1 " "
5 " " 2 " "
3 " " 3 " "
5 " " 5 " "

7 measurements at 10 minute intervals
6 " " 20 " "
6 " " 30 " "
11 " " 1 hour "

and then at 3 hour intervals (although these may be varied for convenience during night pumping).

During all pumping tests, bore discharge should be closely monitored and adjusted if necessary. Such adjustments are normally done by means of a gate valve in the discharge pipe or by altering the speed of the motor driving the pump. All orifice piezometer readings should be recorded.

On completion of the test, recovery of the water levels is measured for about 12 hours or to full recovery although duration of measurements is at the discretion of the hydrogeologist. There is no point in continuing measurements where the rate of recovery becomes zero, i.e. full recovery will never occur. Measurements for the 12 hour period should be as shown for the constant discharge tests. Both step discharge and constant discharge pumping should be continuous. If the pump stops during a step test, the test should be repeated from the beginning; depending on pumping time before stoppage, the CD test may also have to be repeated.

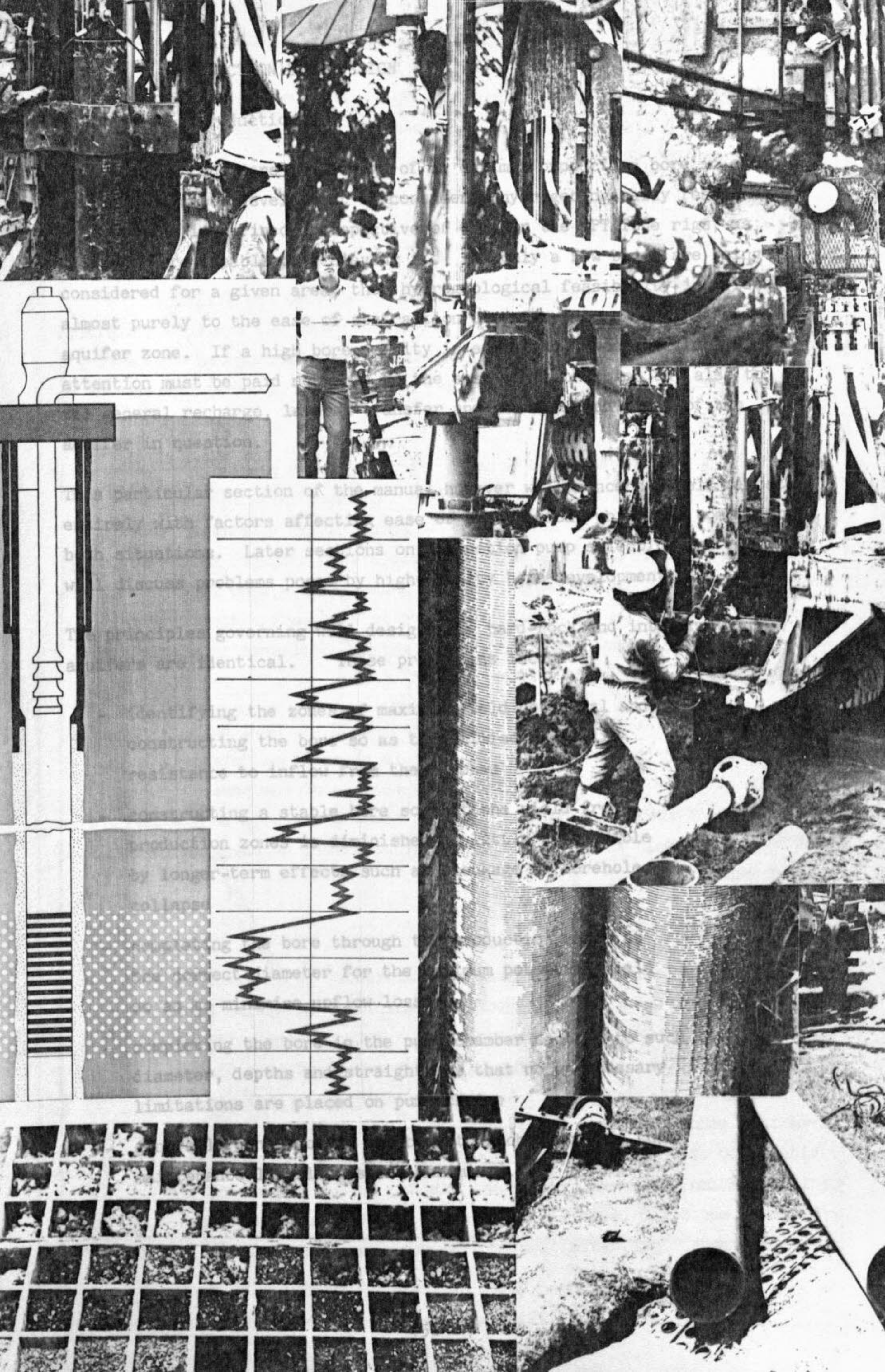
For all tests, arrangements should be made for water to be drained away from the site and special pipes or culvert may be necessary. This is to avoid recycling of discharged water back into the aquifer. There seems a particular danger of recycling in the unconfined karstic limestone aquifers in Kedah and Perlis.

Chemical Measurements

Unstable and volatile chemical constituents in the discharged groundwater, including pH, alkalinity, iron and gases, are best determined at the bore discharge point. Otherwise, the parameters mentioned will change markedly by the time that a water sample is analysed in a laboratory, e.g. gases lost, Ferrous iron in solution precipitated as ferric hydroxides. Bore head measurements are made with portable chemical test kits capable of titrametric and colorimetric tests (i.e. Hach type) and with electrical conductivity (EC) and pH meters.

During a step test, EC might be measured a minimum of 3 times per step to check for any EC variation with time. Chemical tests using the Hach are perhaps best run towards the end of the step, say at time 150 minutes. JPT groundwater group is currently testing chloride, sulphate, iron, pH and alkalinity/hardness. During a CD test pumping, EC should be measured at least every 3 hours and chemical tests run at say 6 hour intervals. At the end of any test, 2 x litre water samples should be taken, for standard chemical analysis. Changes in water turbidity, colour and sand content during testing should be noted, together with the chemical data. Such changes may be indicative of bore development or even of sudden vibrations caused by mechanical failure.

The analysis and presentation of chemical data is discussed in a later chapter.



considered for a given area than hydrogeological feasi
almost purely to the ease of
aquifer zone. If a high bore yield
attention must be paid
general recharge, la
aquifer in question.
This particular section of the manual
entirely with factors affecting ease of
both situations. Later sections on pump
will discuss problems posed by high
The principles governing bore design
aquifers are identical. These pr
1. Identifying the zones of maxi
constructing the bore so as t
resistance to inflow from the
constructing a stable bore so
production zones is diminishe
by longer-term effects such a
collapse.
the bore through t
correct diameter for the
so as to minimise inflow loss
the bore in the pump chamber
diameter, depths and straightness that no
limitations are placed on pu

5. PRODUCTION BORE DESIGN

5.1 Introduction

This chapter considers the design of an optimum production bore in areas where groundwater development is considered hydrogeologically feasible. This optimum is defined irrespective of whether the JPT Tone rigs, as equipped, are capable of achieving it. If only a few bores are being considered for a given area, then hydrogeological feasibility is related almost purely to the ease of abstraction of water from a particular aquifer zone. If a high bore density is envisaged, then specific attention must be paid not only to the ease of abstraction but also to the general recharge, lateral transfer and storage conditions of the aquifer in question.

This particular section of the manual however will concern itself almost entirely with factors affecting ease of abstraction, which are common to both situations. Later sections on production pump selection and recharge will discuss problems posed by high-density bore development.

The principles governing well designs for hard-rock and intergranular aquifers are identical. These principles include:

- identifying the zones of maximum yield potential and constructing the bore so as to minimise hydraulic resistance to inflow from these zones
- constructing a stable bore so that the yield from production zones is diminished as little as possible by longer-term effects such as blockage or borehole collapse
- completing the bore through the production zones at the correct diameter for the maximum potential yield so as to minimise upflow losses
- completing the bore in the pump-chamber section to such diameter, depths and straightness that no unnecessary limitations are placed on pump choice
- generally constructing the bore to such standards that maintenance liabilities are minimised.

It is only the application of these principles which differs between hard rock and intergranular aquifers. These applications are considered in the following Sections.

5.2 Hard Rock Bores

5.2.1 General

Hard rock bores can be considered according to whether the yielding section of the hard rock is stable or unstable. That is, can it be completed to an open-hole design or are some formation-support arrangements necessary. These differences are applicable whether the yielding fissured zone is shallow or at considerable depth.

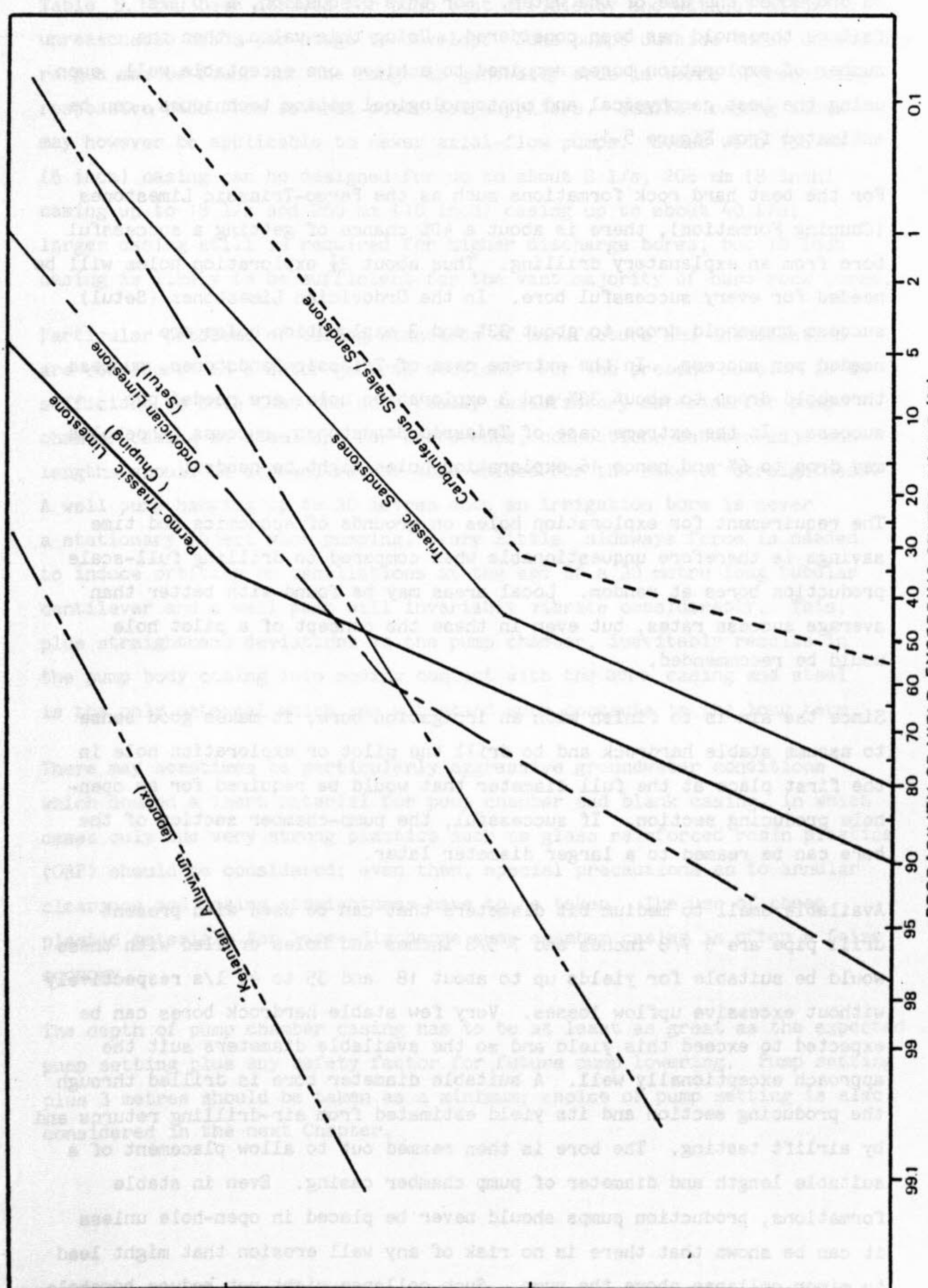
A bore with a few concentrated or major yielding zones has to be considered as unstable, since the concentration of yield is almost certainly due to preferential weathering or solution or to localization of stress fracture zones. These characteristics make it possible that collapse of the yielding zone will occur during the bore lifetime, necessitating some form of active formation support. On the other hand, hard rock bores yielding from a whole distributed range of minor fissures may be considered as stable and open-hole designs can be considered.

Open-hole in this context refers to the section of the bore below the pump setting. Even if this section of the bore is assessed as "stable", it is unwise to actually set a pump in open-hole because of the possibility of small blocks of formation coming loose and jamming the pump in the bore. Pump settings should always be protected by some form of blank or formation-support casing.

5.2.2 Stable Hard Rock Bores

Whether downhole conditions are stable or unstable, it is practically impossible to design a hard rock bore in Peninsular Malaysian conditions without first drilling an exploratory hole. This is simply illustrated by Figure 5.1 which shows the probability of estimated test yields (not production yields) for a 7.5 metre drawdown in various geological formations studied by this and previous projects. In most countries, test yields of 10 or 20 l/s would be classed as 'failures' in terms of economic padi irrigation. However, much lower yields, in the range 1-10 l/s, are common and could certainly be used for the irrigation of tobacco and vegetable crops. The definition of 'failure' therefore depends largely

FIGURE 5.1 FORMATION 'FAILURE' PROBABILITIES



on projected end use of the water. For this discussion, a 10 l/s failure threshold has been considered. Using this value, then the number of exploration bores required to achieve one acceptable well, even using the best geophysical and photogeological siting techniques, can be estimated from Figure 5.1.

For the best hard rock formations such as the Permo-Triassic Limestones (Chuping Formation), there is about a 40% chance of getting a successful bore from an explanatory drilling. Thus about 2½ exploration holes will be needed for every successful bore. In the Ordovician Limestones (Setul) success threshold drops to about 33% and 3 exploration holes are needed per success. In the extreme case of Triassic Sandstones, success threshold drops to about 33% and 3 exploration holes are needed per success. In the extreme case of Triassic Sandstones, success thresholds may drop to 6% and hence 16 exploration holes might be needed!

The requirement for exploration holes on grounds of economics and time savings is therefore unquestionable when compared to drilling full-scale production bores at random. Local areas may be found with better than average success rates, but even in these the concept of a pilot hole would be recommended.

Since the aim is to finish with an irrigation bore, it makes good sense to assume stable hardrock and to drill the pilot or exploration hole in the first place at the full diameter that would be required for an open-hole producing section. If successful, the pump-chamber section of the bore can be reamed to a larger diameter later.

Available small to medium bit diameters that can be used with present drill pipe are 5 7/8 inches and 7 5/8 inches and holes drilled with these would be suitable for yields up to about 18 and 35 to 40 l/s respectively without excessive upflow losses. Very few stable hardrock bores can be expected to exceed this yield and so the available diameters suit the approach exceptionally well. A suitable diameter bore is drilled through the producing section and its yield estimated from air-drilling returns and by airlift testing. The bore is then reamed out to allow placement of a suitable length and diameter of pump chamber casing. Even in stable formations, production pumps should never be placed in open-hole unless it can be shown that there is no risk of any wall erosion that might lead to minor collapse above the pump. Such collapse might not reduce borehole yield but could cause the pump to become jammed in the hole.

Table 5.1 has been derived for pump best efficiency and without allowing unreasonable heads-per-stage to develop. Some pumps outside these capability ranges may be found but the range is generally true in terms of receiving responsive bids from several potential suppliers. Smaller casing sizes may however be applicable to newer axial-flow pumps. Bores with 156 mm (6 inch) casing can be designed for up to about 8 l/s, 206 mm (8 inch) casing up to 18 l/s and 260 mm (10 inch) casing up to about 40 l/s; larger casing still is required for higher discharge bores, but 10 inch casing is likely to be sufficient for the vast majority of hard rock bores.

Particular problems of casing standards of manufacture and installation are considered in a later general section. For the present however it is sufficient to note that the only really satisfactory material for pump chamber casing is steel and for preference, connections between adjacent lengths should be screwed rather than welded for the sake of straightness. A well pump hanging up to 30 metres down an irrigation bore is never a stationary object when pumping. Very little sideways force is needed to induce orbiting or oscillations at the end of a 30 metre long tubular cantilever and a well pump will invariably vibrate considerably. This, plus straightness deviations in the pump chamber, inevitably results in the pump body coming into moving contact with the bore casing and steel is the only material which can withstand such contacts in the long term.

There may sometimes be particularly aggressive groundwater conditions which demand a inert material for pump chamber and blank casing, in which cases only the very strong plastics such as glass reinforced resin plastics (GRP) should be considered; even then, special precautions as to annular clearance and casing straightness have to be taken. The use of cheap plastic materials for large-discharge pump chamber casing is often a false economy.

The depth of pump chamber casing has to be at least as great as the expected pump setting plus any safety factor for future pump lowering. Pump setting plus 3 metres should be taken as a minimum; choice of pump setting is also considered in the next Chapter.

Table 5.1 Pump Casing Internal Diameter for Various Duties

		Electrical Submersible 2,900 rpm		Lineshaft 1,450 rpm	
Pump Diameter (mm)	Preferred* Minimum Clearance (mm)	Discharge** Range (l/s)	Preferred+ Casing Internal Diameter (mm)	Discharge** Range (l/s)	Preferred+ Casing Internal Diameter (mm)
95	10	1-3	156	n/a	
100	10	2-4	156	n/a	
125	15	4-8	156	n/a	
141				6-15[1]	206
150	20	6-18	156-206	2-9	156-206
175	25	9-30	206-256	5-15	206-256
200	25	13-40	256	10-30	256
241[2]				10-45[2]	256
250	25	25-75	311	20-60	311

Note

* Both sides of pump. Some manufacturers may allow smaller clearances, up to one half of values quoted.

** For mixed flow pumps.

\+ Next available size up.

n/a Not available.

1 JPT BP 45-4 lineshaft currently operating in 156 mm i.d. casing 2,000 rpm.

2 JPT 10 M50 lineshaft to be operated in 256 mm i.d. casing. Discharge range 15-60 l/s at 2,000 rpm.

256 Casing sizes currently available at JPT.

Casing internal diameters suitable for various pump discharges are considered to be roughly as shown (Table 5.1). Great care must be taken in specifying casing however to avoid confusion between internal, nominal and external dimensions since even drilling industry standards differ in their reference diameter. Confusion can also arise when, in the name of economy (false), tubular goods specifically designed for purposes other than well casing are used. Choice of pump type is considered in Chapter 6.

The diameter of hole that has to be reamed out for pump chamber casing is the next consideration and this should recognise both that the casing may be screw-threaded with collar-couplings and that the reamed hole may not be truly straight or vertical. Problems caused by misunderstanding of the importance of annular clearance at all stages of bore construction result in more physical difficulties than almost any other. The stiffness of even a steel pump chamber casing string when 30 or 40 metres long is slight. It will certainly curve to follow drilled hole deviations and when "landed" on bottom-hole or on hole diameter shoulders, will almost certainly buckle to magnify the drilled hole deviations. A bore is seldom a straight and vertical hole in the ground!

In genuine stable hard rock, the absolute minimum clearance between casing maximum OD and drilled diameter can be allowed. This is considered to be about 8 to 10 mm in annular terms, or 16 to 20 m in diametric terms. However the slimmest screwed-casing couplings are generally 22 to 32 mm greater in OD than the nominal or internal diameter and so a 194 mm (7 5/8 inch) hole is usually required for 156 mm (6 inch) pump casing, a 251 mm (9 7/8 inch) hole for 206 mm(8 inch) casing, a 311 mm (12¼ inch) hole for 260 mm (10 inch) casing, and so on, as shown on Figure 5.2. These drill bit sizes must of course be able to pass freely through any temporary or conductor casing used to reach hard rock through overburden.

The last major consideration in stable hard rock bore designs is whether or not a possible yielding zone is within the depths allocated to pump chamber casing. If so, then the length of casing in question has to be perforated to allow direct water entry. The only function of these perforations is to prevent collapse of large fragments of formation on and into the pump while still allowing direct water entry. Its technical name is simply "window casing" and the perforations in it can be quite rough; these are usually flame-cut slots 5 to 15 mm wide and 100 to 150 mm long, arranged to give 10 to 15% open area. Such casing is usually "landed" on the reamed hole shoulder to prevent any collapsed material falling further down the bore and,depending on the up-hole and temporary casing diameters, a "basket" or packer is sometimes fitted above the slotted section to prevent the ingress of loose overburden; this basket may be cemented if required.

Figure 5.2 shows typical completion details for stable hard rock bores and also shows discharges appropriate to various diameters.

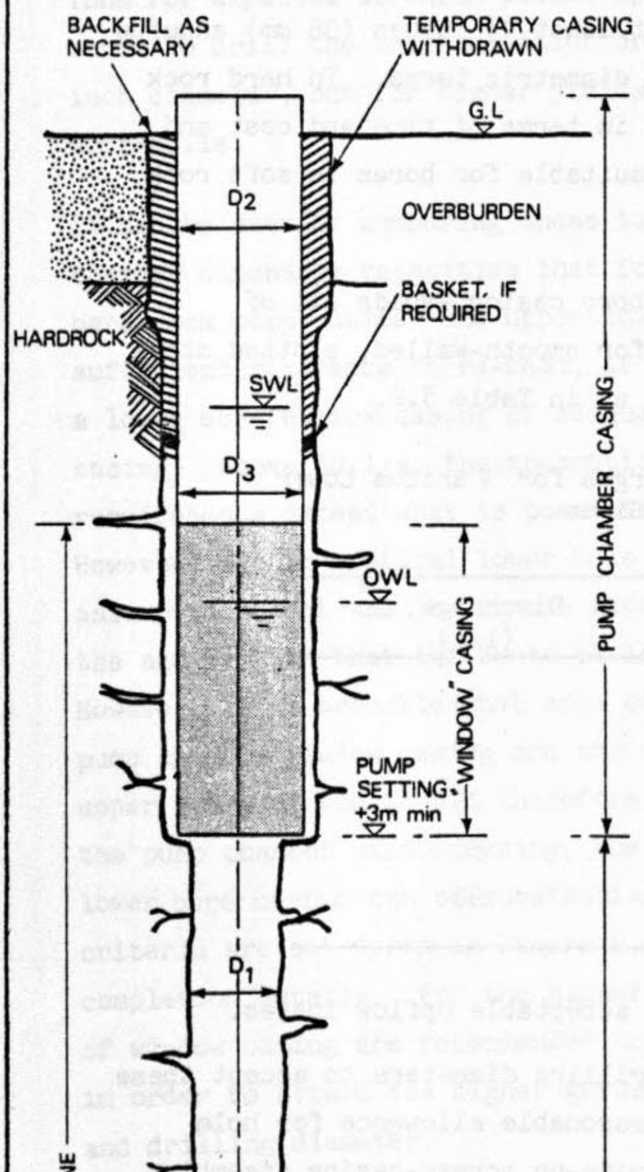
5.2.3 Unstable Hard Rock Bores

Where the following type of evidence is available, then the bore can be considered unstable and possibly subject to yield deterioration:

- evidence of significant overbreak taking place during drilling, or from geophysical logging,
- evidence from drilling and testing data that the yielding zone is concentrated in a few major and mature fissure systems,
- evidence from testing and development that granular material such as silt and sand, is continually being brought into the bore from the aquifer. This happens in Chuping and Setul Limestone where karst voids are filled with red silts.

The bore must therefore be completed to an unstable hard rock design. That is, window casing should be provided for most of the producing section, whether opposite or below the actual pump chamber. In the absence of evidence of extreme corrosion, screwed or weld-coupled steel casing would be the recommended material for window and blank casing below the pump casing (the lower bore casing). Plastics have been used for the same purpose but because the cheaper varieties (PVC and thermo-derivatives) tend to have very thick walls for suitable strengths and the more expensive varieties (GRP reinforced resin materials) tend to have very bulky joints, the larger drilling clearances needed may offset the original cost of any material savings. More importantly in hard rock bores and especially in terms of pump chamber casing, such plastics are not sufficiently strong in base pipe strength or coupling rigidity to withstand the diametric or straightness deformations that can result from asymmetrical borehole wall collapse. Plastics are therefore almost never used for such purposes unless a sufficiently large drilling clearance is provided between casing OD and bore diameter to ensure the satisfactory emplacement of a granular backfill or "formation support" material. This material prevents asymmetrical collapses developing, thus allowing the casings available strength to be developed optimally (a cylinder resisting radial pressures) and also limits straightness

FIGURE 5.2 STABLE HARDROCK BORE DESIGNS



Recommended Diameters

Discharge, Q (l/s)	D_1 Open Hole diam. (mm)	D_2 Pump Casing ID. (mm)	D_2 Pump Casing Nominal dia. (ins).	D_3 Minimum Reamed hole diam. (mm) plus Equivalent bit size (ins).
1-8	149	156	6	194 (7 5/8)
18	149	206	8	251 (9 7/8)
30	194	256	10	311 (12¼)
40	194	256	10	311 (12¼)
60	251	311	12	381 (15)

Note:

Open hole can always be re-reamed through pump chamber casing. Backfill may be cement just above basket.

156 Available JPT bit and casing sizes.

A stiff drill assembly including stabilisers must be used for these designs.

deviations. The necessary clearance to ensure satisfactory formation support material emplacement is however at least 1½ inches (38 mm) annulus over casing joints or 3 inches (75 mm) in diametric terms. In hard rock drilling these clearances are prohibitive in terms of time and cost and so the plastic materials are only really suitable for bores in soft rock or alluvium.

The use of welded steel casing for lower bore casing avoids all of these problems. The limiting discharges for smooth-walled, slotted steel casing are considered to be approximately as in Table 5.2.

Table 5.2 Maximum Desirable Discharges for Various Lower Bore Casing Sizes

Casing Nominal Diameter (ins).	I.D. (mm)	Discharge, Q* (l/s)
4	90	15
5	114	18
6	156	25
8	206	50
10	256	80

Note: *Maximum desirable for acceptable upflow losses.

Table 5.3 shows the minimum recommended drilling diameters to accept these nominal casing sizes in hard rock, with reasonable allowance for hole straightness and the straightness of the made-up screen-casing assembly.

Table 5.3 Drilling Diameters for Accepting Welded Casing

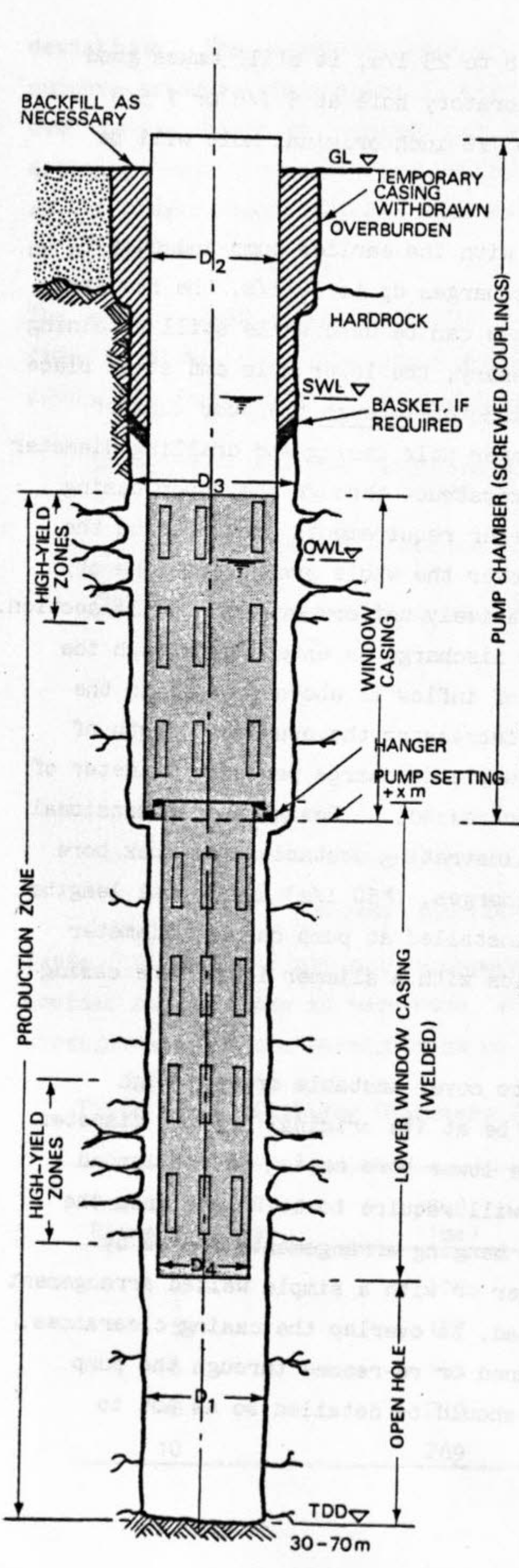
Casing Nominal Diameter (ins).	O.D. (mm)	Clearance on Annulus (mm)	Minimum Drilling Diameter (mm)	
			(mm)	(ins).
4	102	10	149	5 7/8
5	127	11	149	5 7/8
6	165	13	194	7 5/8
8	219	15	251	9 7/8
10	269	15	311	12¼

Thus for expected borehole yields up to 18 to 25 l/s, it still makes good sense to drill the original pilot or exploratory hole at 5 7/8 or 7 5/8 inch diameter, but for higher yields, a 9 7/8 inch original hole will be worthwhile.

It can be seen by comparing these tables with the earlier pump and pump chamber discharge capacities that for discharges up to 50 l/s, the standard hard rock pump chamber and upper hole sizes can be used while still retaining sufficient clearance to re-ream, if necessary, the lower hole and still place a lower bore window casing of adequate diameter through the pump chamber casing. Above 50 l/s, the theoretical lower hole casing and drilling diameter requirements exceed what is possible to construct through the upper casing. However, the theoretical lower hole diameter requirements are based on the assumptions that the lower hole extends over the whole production zone of the aquifer and that inflow to it is relatively uniform over the whole section. However, it is probable that some of the discharge is entering through the pump chamber window casing and the rate of inflow is above average in the upper zones of the bore. Therefore, by increasing the absolute length of the pump chamber window casing, the allowable discharge per unit diameter of lower bore casing can be substantially increased. Designs and dimensional criteria are set forth in Figure 5.3, illustrating unstable hard rock bore completion details. For the higher discharges, (>50 l/s) additional lengths of window casing are recommended to be installed at pump chamber diameter in order to attain the higher gross yields with a slimmer lower bore casing and drilling diameter.

The lower bore casing need only extend to cover unstable or very high yield zones. Below this, the bore may be at its original drilled diameter or may be reamed. If the latter and the lower bore casing is not landed on a reamed diameter shoulder, then it will require to be "hung" from the bottom of the pump chamber casing. The hanging arrangements may be by means of a proprietary bore casing hanger or with a simple welded arrangement at pump casing shoe and lower casing head, to overlap the casing clearances. Since the lower hole can always be cleaned or re-reamed through the pump chamber casing, the hanger arrangement should be detailed so as not to interfere with this facility.

FIGURE 5.3 UNSTABLE HARDROCK BORE DESIGNS



Recommended Diameters

Discharge, Q (l/s)	D_1 Open Hole diam. (mm)	D_2 Pump Casing ID (mm)	D_2 Pump Casing Nominal diam. (ins).	D_3 Minimum Reamed Hole Diameter (mm) plus equivalent bit size (ins).	D_4 Lower Casing Nominal diam. (ins).	X Pump Casing Setting (m).
1-8	149	156*	6	194 (7 5/8)	4	3
18	149	206	8	251 (9 7/8)	5	3
25	194	256	10	311 (12¼)	6	3
30	251	256	10	311 (12¼)	8	3
50	251	311	12	381 (15)	8	3
60	251	311	12	381 (15)	8	6

Note:

Lower bore can be re-reamed, cleaned or cased through pump chamber casing. Backfill may have to be cement just above basket if drilling clearances are large.

156 Available JPT bit and casing sizes.

*Preferred; lineshaft pumps with discharge 6- 16 l/s can fit into 156 mm casing.

5.2.4 Construction

It must be remembered that in hard rock bores, unlike alluvial bores, actual yield obtained bears little or no relationship to the depth or diameter to which a bore is constructed. Yielding zones are found randomly at discrete levels, and it is the quality of these that determines potential yield. Their number invariably decreases with depth drilled and experience has shown that drilling below about 60 m will seldom intersect yielding fissures and is therefore not worthwhile. Similarly, choosing a particular diameter for construction will not guarantee a particular yield. The diameters are simply chosen after potential yield has been quantified by initial testing and are chosen to minimise drawdown losses in abstracting it. Diameter is a consequence of yield, not vice-versa.

Thus for all hard rock bores, an exploratory or pilot hole would first be drilled through adequate temporary or conductor casing, at the exploratory diameter suitable for the desired well yield, 5 7/8, 7 5/8 or 9 7/8 inches (149, 194 or 251 mm). Air drilling yields would be noted to assess both the overall yield potential and vertical distribution of yielding zones; these would be confirmed by geophysical logging, particularly caliper logging and the rock assessed as stable or unstable . At least an airlift test and preferably a proper pumped step test would be carried out to provide numerical data to quantify the bore's ultimate production yield and pump setting (see Chapter 6 on pump selection).

If test yield is adequate, the eventual production bore completion diameter could be determined, as from Figures 5.2 and 5.3. The next operation would be to ream out the hole for the pump chamber casing and upper window casing to the necessary depth at the pertinent diameter.

If the existing pilot bore diameter is deemed adequate for eventual open-hole or lower window casing completion, this bore may be temporarily plugged (just below eventual pump casing shoe setting)during the reaming of the upper hole to prevent reaming cuttings filling the lower hole. If the lower hole has itself to be reamed out this can either be done through the pump chamber casing or directly after completion of reaming the upper hole.

On completion of reaming and cleaning out of the lower bore if necessary, the permanent pump chamber casing and window casing can be lowered into position. This should not be centralised in the hole, but adjusted within the annular clearances to optimise pump chamber straightness and verticality as measured by verticality surveys. If the annular clearance is significant, a cement basket may need to be fitted outside the casing, above the production zone, to prevent downward movement of collapsed weathered rock and overburden. For substantial annular clearances (above 25 mm) it may be necessary to emplace some cement grout just above the basket to give some permanence to the isolating seal. The remainder of the backfill (any material) can then be placed to the surface; its function is mainly to preserve the pump chamber casing in its straight and vertical condition. After placing pump chamber casing and allowing some time for cement to set, any required lower bore casing can be emplaced and seated on its hanger arrangement. The completed bore should then be developed by the most vigorous means available, usually some variant of overpumping combined with intermittent positive backwashing.

A full bore test should then be carried out to confirm the details of the production pump selection.

5.3 Casing and Straightness Standards

It is a prevalent misconception that almost any form of tubular goods can be adapted for use as bore casings. They can in special circumstances, but the particular precautions required and the risks involved must be recognised. It is therefore considered pertinent at this point to digress from the main design theme and consider particular bore casing requirements.

The stresses acting upon properly installed bore casing, once in place, are slight but what is generally not recognised is that the stresses generated during handling and installation and due to improper emplacement, can be several orders higher. For example, lower bore casing or screen needs only to be able to withstand an external uniform collapse pressure of some 0.08 kg/cm^2 for every metre of its depth setting. Most cheap plastics have such a collapse resistance, but they will usually be deformed, overstressed or broken during the handling and installation process. Further, the specified collapse resistance assumes a set of radial forces acting on a cylindrical pipe body. These conditions are never even approached for plastic casing in hard rock holes unless the space between the casing and the bore wall is filled with a sufficient thickness of formation-support backfill. In the absence of this, forces will be asymmetric and the necessary collapse resistance can increase by of factor of 10 at least. The provision of the necessary annular clearance and backfill can be expensive in terms of drilling time and effort.

Although collapse pressure may be a valid criteria for lower bore casing where yield is not affected by partial casing collapse, it is a different matter in pump chamber casing where the designer is working with very small clearances between pump OD and casing ID. These problems are greatly exacerbated by any lack of casing straightness (to which the cheaper materials are prone). A more pertinent criterion than collapse resistance for this application would be deformation resistance. When working at small pump-to-casing clearances, and bearing in mind handling stresses and thermal deformations that can easily occur, there is no known thermoplastic material that provides the absolute requirement in deformation resistance for irrigation bore pump casing duties. Some thermosetting plastics such as GRP (fibreglass), can provide the necessary deformation resistance as pump casings. However, they are not cheap and their joints are both bulky and flexible, necessitating extra drilling diameter and straightness precautions; they would only be used where aggressive groundwaters occur or in alluvial aquifers.

In normal groundwater conditions then, only steel casing should be considered for pump chamber use. Other materials can be considered for lower bore casing if some form of formation-support backfill is provided. Alternative materials for the pump chamber require additional clearance and drilling allowances and special straightness precautions which are generally not cost-effective.

Factors other the material used are relevant. Waterworks qualtity steel tubular goods often appear much cheaper than special bore casing for the following reasons:

- that the wall thickness of casing is usually greater than that of pipe, external pressures being more difficult to resist than internal pressures and rigidity being more important for casing.
- that the standards of coupling alignment, and hence coupled unit straightness are much higher for casing then for pipe.

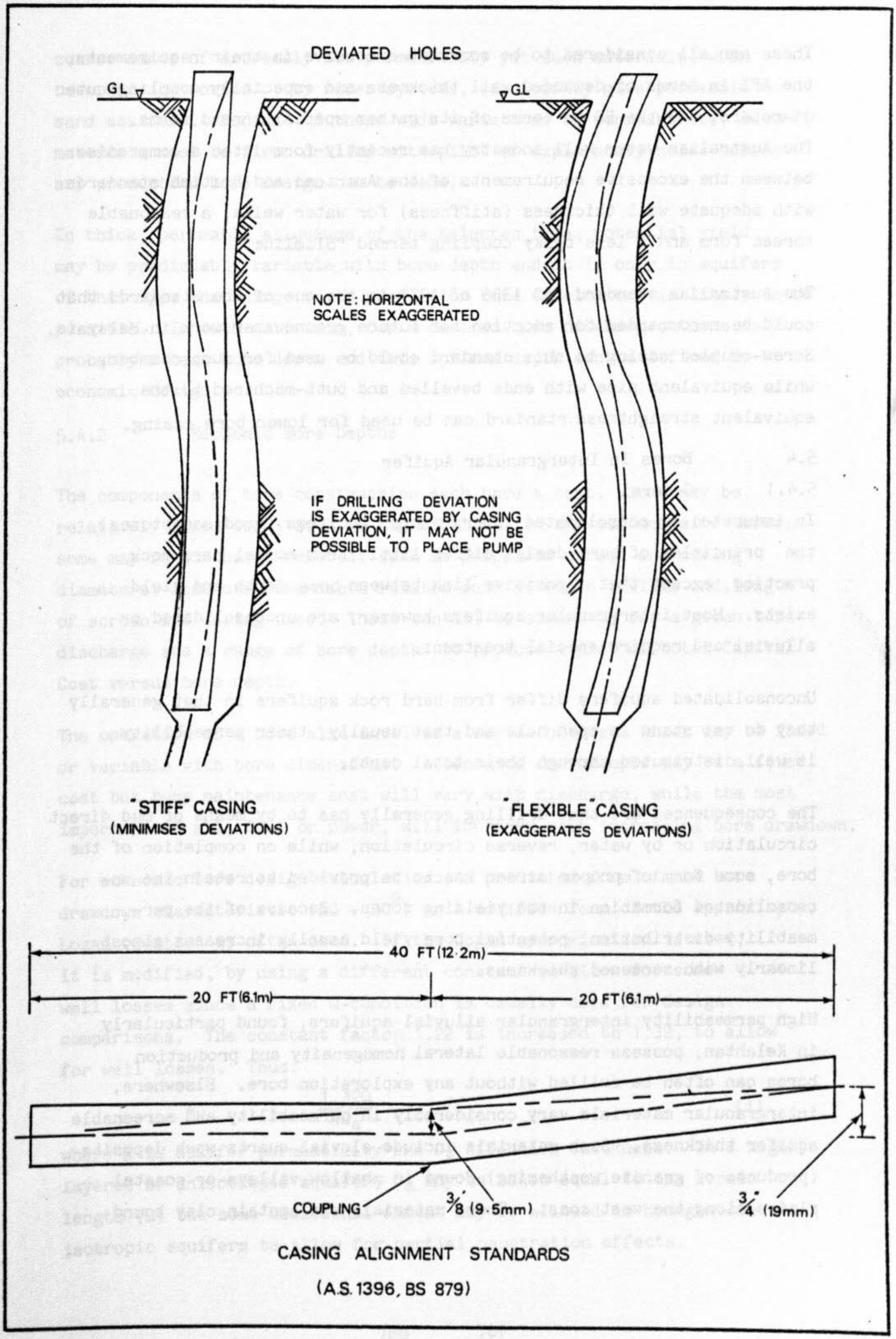
Since most casing standards are originally derived from the oil industry, and water-well pressures are much lower, the additional thickness requirement given above is partially irrelevant, except that it contributes to the stiffness of long casing strings. Stiffness stops them from buckling when landed and allows them to smooth-out rather than exacerbate any straightness deviations in the drilled hole, as illustrated in Figure 5.4.

Casing straightness standards are another matter, especially for pump casing. A lineshaft pump or column pipe must not touch casing at more than one point along its length otherwise an extra bending stress is imposed on the shaft during running, with consequent increased maintenance frequency. Casing straightness standards internationally recognise this fact and so call for a coupling alignment accuracy so that the maximum centreline deviation of 40 feet (12.2 m) of coupled casing is no more than 3/8 inch (9.5 mm) from a line joining the ends of the casing length (Figure 5.4). Pump recommended annular clearances are based on this standard and casing coupling threads are cut to it. No waterworks pipe thread comes close to these requirements.

Welded casing joint ends can be prepared to the same straightness standards by factory end-machining and weld-bevel cutting. Care must be taken with field welds of the casing in order to maintain straightness standards. Casing joints for pump casing should preferably be factory prepared, screw-threaded joints certified to a bore casing standard. Welded pipe joints can be used for pump chamber if carefully welded. They can also be used for lower bore casing and screen where straightness and verticality is much less important and only the ability to keep the casing or screen roughly centred in the drilled hole is significant.

Traditional steel casing standards are based on either the American Petroleum Institute's (API) oil well casing standards APISA (casing), APISB(couplings and threads), APISL (linepipe), or the British Standards Institution (BS) water-well casing standard BS 879. Casing dimensions for various standards are given in an appendix to the Drilling Manual.

FIGURE 5.4 CASING STIFFNESS AND STRAIGHTNESS



These are all considered to be somewhat excessive in their requirements, the API in terms of demanded wall thickness and especially coupling outer diameters, and the BS in terms of its rather special thread forms. The Australian water well industry has recently formulated a compromise between the excessive requirements of the American and British standards with adequate wall thickness (stiffness) for water wells, a reasonable thread form and a less bulky coupling termed "Slimline".

The Australian standard, AS 1396 of 1979 is the one of the standards that could be recommended for adoption for future groundwater work in Malaysia. Screw-coupled casing to this standard could be used for pump chamber, while equivalent pipe with ends bevelled and butt-machined to the equivalent straightness standard can be used for lower bore casing.

5.4 Bores In Intergranular Aquifer

5.4.1 General

In indurated or consolidated intergranular aquifers (good sandstones) the principles of bore design differ little from normal hard rock practice, except that a positive link between bore depth and yield exists. Most intergranular aquifers however, are unconsolidated or alluvial and require special treatment.

Unconsolidated aquifers differ from hard rock aquifers in that generally they do not stand in open hole and that usually, their permeability is well distributed through their total depth.

The consequences are that drilling generally has to by means of mud direct circulation or by water, reverse circulation, while on completion of the bore, some form of proper screen has to be provided to retain the unconsolidated formation in the yielding zones. Because of the permeability distribution, potential bore yield usually increases almost linearly with screened thickness.

High permeability intergranular alluvial aquifers, found particularly in Kelantan, possess reasonable lateral homogeneity and production bores can often be drilled without any exploration bore. Elsewhere, intergranular materials vary considerably in permeability and screenable aquifer thickness. Such materials include eluvial quartz wash deposits (products of granite weathering) found in shallow valleys or coastal plains along the west coast. These materials can contain clay bound coarse sands of extremely low permeability yet such materials, when reworked and sorted by river systems, can locally give permeable sand aquifers. Because of these wide variations in permeability, such materials must be first drilled with pilot or exploratory holes before any production bore design can be made.

In thick, permeable alluviums of the Kelantan type, potential yield may be predictably variable with bore depth and it is only in aquifers of this type that large scale production bore development is likely to be possible. In such materials, for a desired bore discharge, there is probably a best bore depth and this optimum depth can be defined by economic design.

5.4.2 Economic Bore Depths

The components of bore construction each have a cost. Some may be relatively fixed-cost, such as rig mobilisation and surface site works; some may be discharge-related, such as pumps and screen and casing diameters; some are bore-depth related such as depth drilled and length of screen and casing used. These can all be summed up for a given bore discharge and a range of bore depths to produce a plot of total Capital Cost versus bore depth.

The operation of a bore also involves a series of costs which may be fixed or variable with bore dimensions. Attendance for example may be a fixed cost but pump maintenance cost will vary with discharge, while the most important of all, fuel or power, will increase with increasing bore drawdown.

For economic bore design calculations, a predicted form of the bore drawdown characteristic ($AQ + BQ^2$) or a modified form of the approximate Logan equation is often used. If the Logan approximation is used, it is modified, by using a different constant to allow for some well losses since a fixed Q-condition is usually used for design comparisons. The constant factor 1.22 is increased to 1.32, to allow for well losses. Thus:

$$s_w = \frac{1.32Q}{KD_a} \qquad (1)$$

where K is aquifer permeability and D_a effective thickness. In a highly layered or anisotropic aquifer, D_a may be taken equal to the screen length (L) but some additional amount may be allowed in homogeneous isotropic aquifers to allow for partial penetration effects.

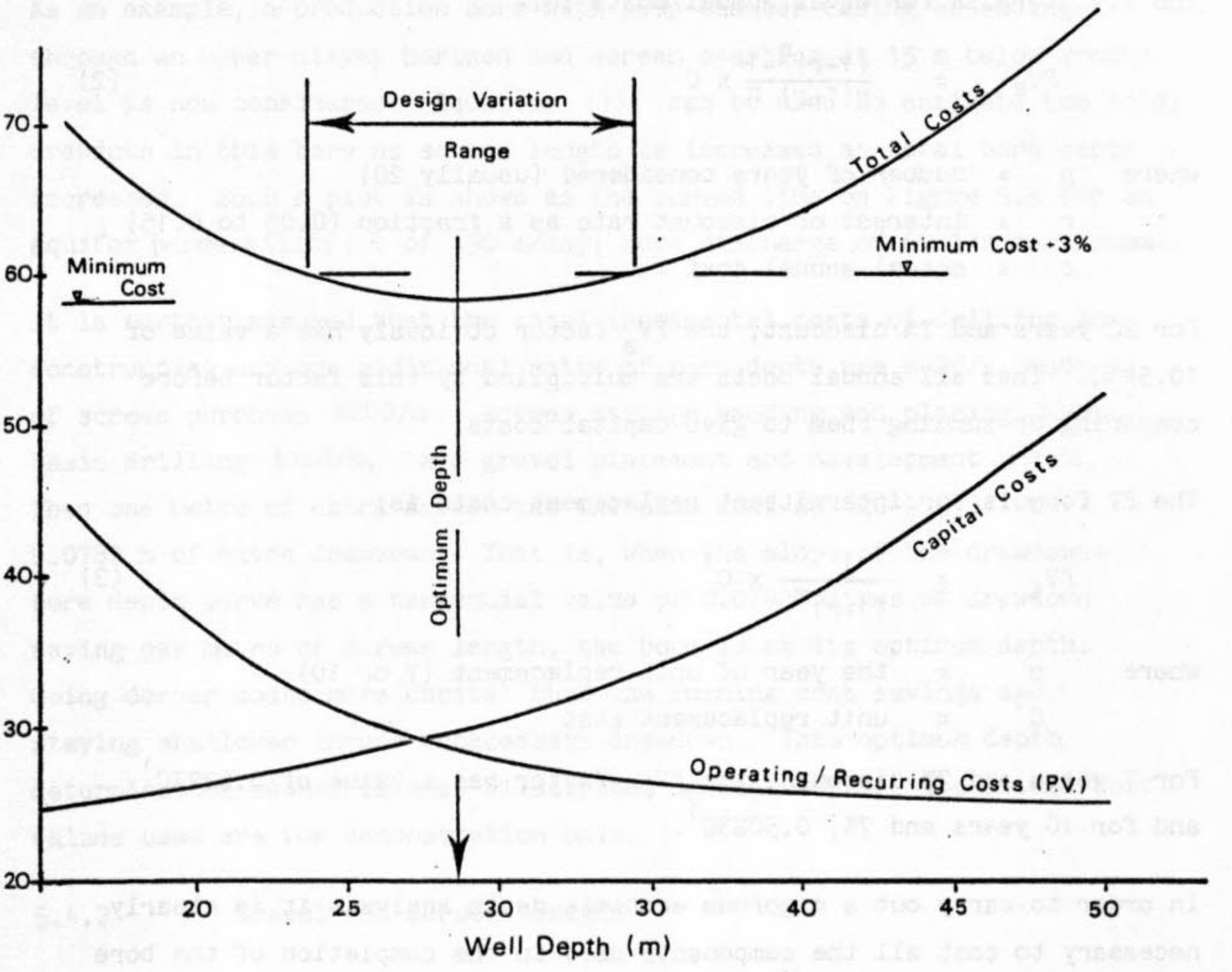
Equation (1) indicates that drawdown and hence pumping power costs, are inversely proportional to screen length. These costs therefore tend to reduce as bore depth increases, as in fact does the capital and maintenance cost of the pumping unit due to less onerous duties.

Thus, for a fixed design discharge and a range of bore depths, the operating or Total Annual Costs can be calculated and plotted against bore depth. This curve has a different slope to that for Capital Costs and when the two curves are summed together to give real Total Costs, it is found that the resulting curve has a minimum, as shown on Figure 5.5. This minimum represents the optimum cost for a bore of the discharge in question in the aquifer in question and the depth corresponding to it is termed the bore "optimum depth".

It can be seen that the Total Cost curve is very flat however in the region of optimum depth and a small depth change will not greatly affect overall costs. It may on the other hand considerably change the balance between Capital and Annual costs. If the sources of capital are different (i.e. capital costs met by Government, operating costs met by farmers), the designer then has a choice of effectively subsidising the farmers (deeper bores) or minimising the Government capital outlay for the some net benefit (shallower bore).

Before the optimisation can actually be carried out, it must be noted that capital and annual recurring costs cannot be directly compared; nor can capital be compared with the sum of running costs over say 20 years. Recurring costs have to be converted to an equivalent Present Value, PV . before the comparison is valid. This PV is roughly the sum which if invested or used to purchase an annuity now, would, over the economic comparison lifetime (usually 20 years), provide a series of annual and intermittent sums to exactly meet annual and intermittent recurring costs while becoming extinguished itself in the last year. Annual sums are needed for the obvious costs of attendance, power and regular maintenance, while intermittent sums may be needed for major pump or motor overhaul or replacement at the end of their service lines, usually taken as 7 or 10 years. The technique is also known as discounted cash flow.

FIGURE 5.5 OPTIMUM BORE DEPTH DEFINITION



OPTIMUM DEPTH = 28.7 m say 29 m

DESIGN VARIATION = 23.6 to 34.5 m, say ±5 m

WELL DEPTH (m)	CAPITAL COST ($)	RECURRING COST P.V. ($)	TOTAL COST P.V. ($)
23.6	27,600	32,250	59,850
34.5	33,500	26,350	59,850
28.7	30,000	28,100	58,100

∴ Shorter well depths to minimise capital investment

Greater well depths to minimise running costs

The P.V. formula for equal annual costs is:-

$$PV_a = \frac{(1+r)^n - 1}{r(1+r)\ n} \times C \qquad (2)$$

where n = number of years considered (usually 20)

r = interest or discount rate as a fraction (0.05 to 0.15)

c = actual annual cost

For 20 years and 7% discount, the PV_a factor obviously has a value of 10.594C. Thus all annual costs are multiplied by this factor before comparing or summing them to give capital costs.

The PV formula for intermittent replacement costs is:

$$PV_i = \frac{1}{(1+r)^m} \times C^1 \qquad (3)$$

where m = the year of unit replacement (7 or 10)

C^1 = unit replacement cost

For 7 years and 7% discount, the PV_i factor has a value of $0.6227C^1$ and for 10 years and 7%, $0.5083C^1$.

In order to carry out a rigorous economic depth analysis it is clearly necessary to cost all the components used in the completion of the bore as these vary with bore depth and hence drawdown; refinements such as decreasing pump chamber length and reduced pump capital costs with reduced drawdown might also be included. Such an exercise would certainly be done if it were decided to install a project-group of say 20+ bores. On the other hand, a very fair indication of bore optimum depth can often be obtained from a much simpler iterative process considering only fuel or power costs and incremental depth-related costs of bore components.

Say power costs, in real terms, \$0.35 per kWhr, then for a production bore operating 1,600 hours per year at 60 l/s with a pump and motor of 65% wire-to-water efficiency, one metre of drawdown in such a well has an annual power cost of:

$$C = \frac{1.0 \times 0.06 \times 9.81 \times 0.35 \times 1,600}{0.65} = \$507/-$$

The PV of this over 20 years is clearly 10.594 x 507 or \$5,371/-. Thus we can afford to spend as capital \$5,371/- to save one metre of drawdown.

As an example, a production bore with pump chamber casing extending through an upper clayey horizon and screen starting at 15 m below ground level is now considered. Equation (1) can be used to estimate the total drawdown in this bore as screen length is increased as total bore depth increases. Such a plot is shown as the curved line on Figure 5.6 for an aquifer permeability, K of 150 m/day; bore discharge of 60 l/s is assumed.

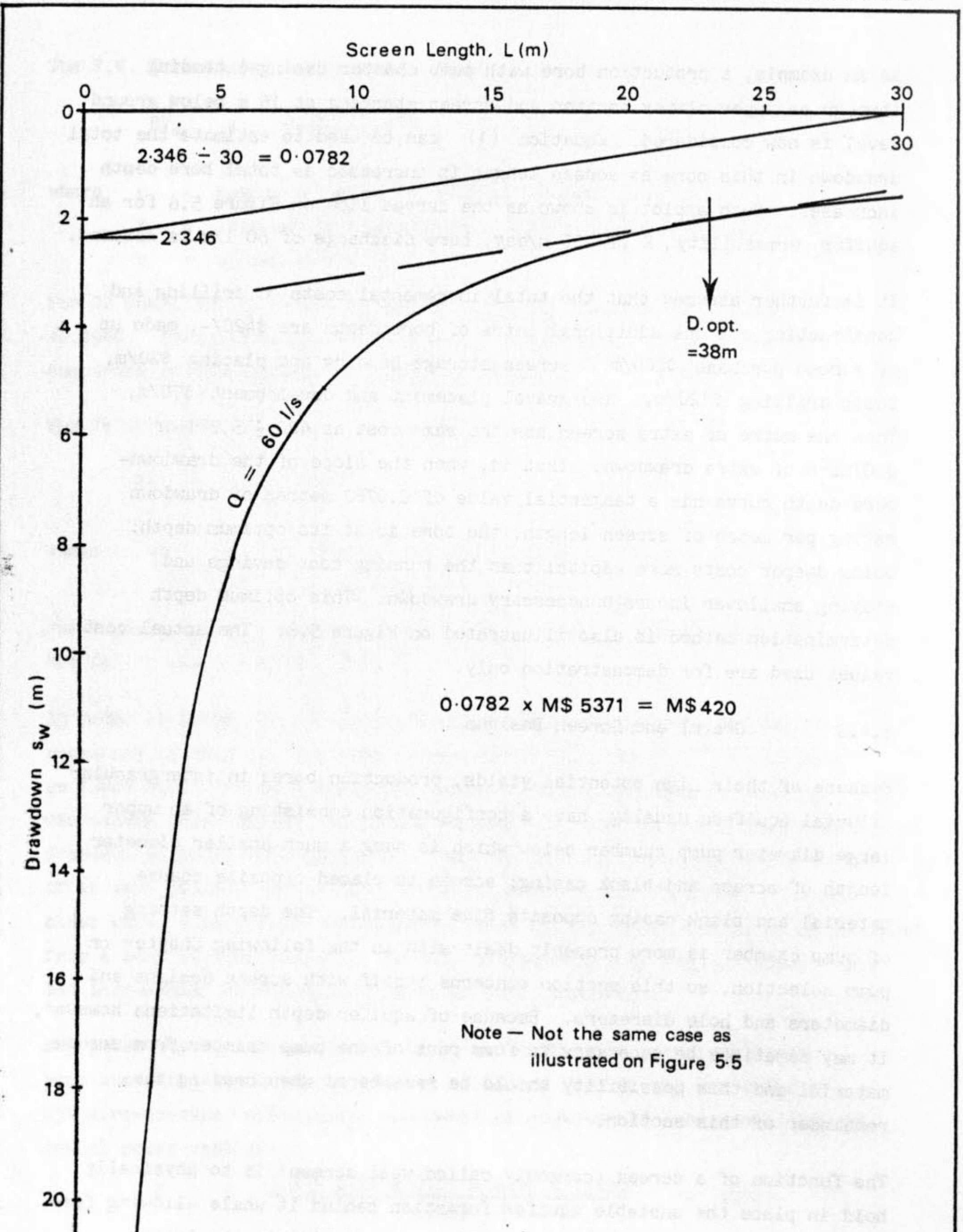
It is further assumed that the total incremental costs of drilling and constructing any one additional metre of bore depth are \$420/-, made up of screen purchase \$200/m screen storage handing and placing \$30/m, basic drilling \$120/m, and gravel placement and development \$70/m, Then one metre of extra screen has the same cost as 420 ÷ 5,371 or 0.0782 m of extra drawdown. That is, when the slope of the drawdown-bore depth curve has a tangential value of 0.0782 metres of drawdown saving per metre of screen length, the bore is at its optimum depth. Going deeper costs more capital than the running cost savings and staying shallower incurs unnecessary drawdown. This optimum depth determination method is also illustrated on Figure 5.6. The actual cost values used are for demonstration only.

5.4.3 Gravel and Screen Designs

Because of their high potential yields, production bores in intergranular alluvial aquifers usually have a configuration consisting of an upper large diameter pump chamber below which is hung a much smaller diameter length of screen and blank casing; screen is placed opposite coarse material and blank casing opposite fine material. The depth setting of pump chamber is more properly dealt with in the following Chapter on pump selection, so this section concerns itself with screen designs and diameters and hole diameters. Because of aquifer depth limitations however, it may sometimes be necessary to form part of the pump chamber from screen material and this possibility should be remembered when reading the remainder of this section.

The function of a screen (commonly called well screen) is to physically hold in place the unstable aquifer formation behind it while allowing free passage to water inflow from the aquifer. To facilitate the latter, screens are usually sized so as to allow some of the formation to be pulled into the bore during the "development" process when drilling detritus is also cleaned off the borehole wall. This re-grading of

FIGURE 5.6 APPROXIMATE OPTIMUM BORE DEPTH DEFINITION



formation material in the immediate vicinity of the borehole wall is assumed to increase its local permeability, so lessening its resistance to inflow.

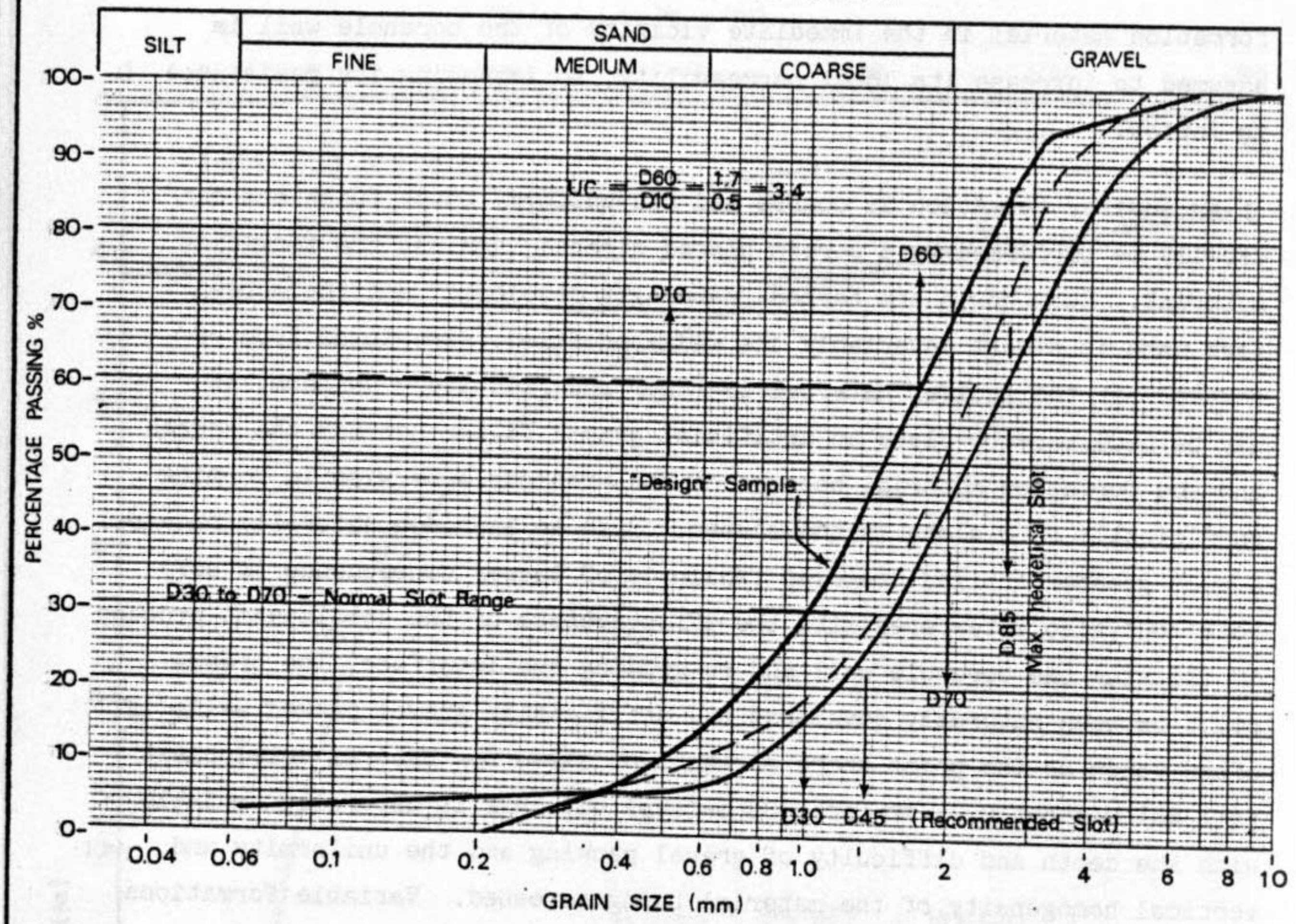
If an aquifer formation is coarse and non-uniform, these functions can usually be performed by a screen placed directly against the formation material. Such bores are termed "naturally developed" and the screen form used is almost invariably the wedge-profiled, wire-wound type of screen. If the aquifer material is finer and relatively uniform, it may be necessary to place an artificial gravel filter between the screen and the formation material because of the problems otherwise in forming sufficiently fine slots in the screen. Such an approach naturally requires a much greater drilled diameter (which is of lesser consequence in soft rock) but also interferes with the effectiveness of the development process in cleaning the borehole wall and re-grading the interface. The breakpoint between naturally developed and artificially gravel packed wells is often taken as the point where screen slot sizes for natural development fall below about 0.75 mm (or 0.03 inches) but this is naturally variable with the depth and difficulty of gravel packing and the uniformity and vertical homogeneity of the material being screened. Variable formations are treated with caution.

Formation uniformity is expressed in terms of the aquifer material's particle size distribution or sieve analysis of an aquifer material; several examples are shown on Figure 5.7a, for Kelantan river and coastal alluvium. Uniformity Coefficient is defined as the ratio of the D60 to the D10 passing formation sizes, Dx being size corresponding to x% of the sample being finer than that size. For Uniformity Coefficients (UC) greater than about 3.0, naturally developed bores are considered feasible provided the resulting screen slot size is not too small.

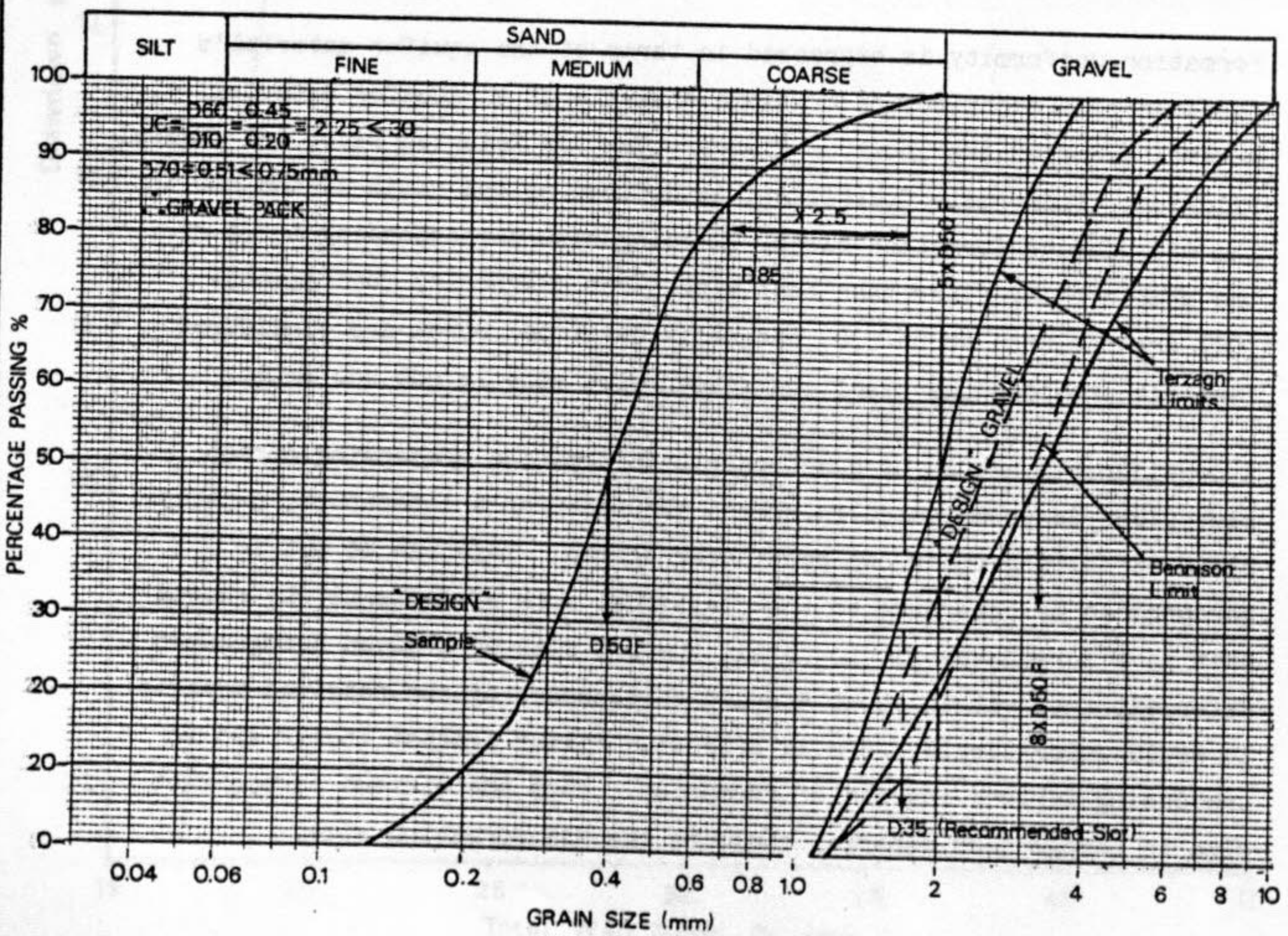
A considerable amount of questionable information concerning screen slot sizes is promulgated in screen manufacturer's literature, usually aimed at selling their own particular screen. This can be put in perspective by reference to the classifcal work of Terzaghi who showed that if a screen was fit to retain the D85 size of a given homogeneous material, then within about 5 grain thicknesses of the retained material, it would automatically form an effective filter against itself and so preventing further ingress of formation. Thus the extreme screen slot size in homogeneous material could be the D15 formation size.

FIGURE 5.7 SCREEN AND GRAVEL DESIGN

(a) KELANTAN ALLUVIUM AND NATURALLY DEVELOPED DESIGNS



(b) GRAVEL PACKED DESIGNS



Some factor of safety is obviously necessary in real wells to allow for sampling difficulties and formation vertical variability, and so a screen slot equal to half the D_{85} passing size, for drilling with adequate sample collection supervision by hydrogeologists, might be treated as an upper limit. If it is accepted that a log-linear plot describes the grain-size distribution of the bulk of an alluvially-deposited material and that the uniformity coefficient limit of UC = 3 is taken for naturally developed bores, then half the D_{85} size would imply on actual slot size of approximately the D_{45} passing formation size. Manufacturer's literature ranges between recommending slot sizes from D_{30} to D_{70} but the former is super-cautious and might be regarded as a means of selling extra-fine slots; the latter comes perilously close to relying on perfect sampling and no vertical variability. It is considered that the D_{45} as slot size would be a fair compromise for formations sampled to a reasonable standard. These slot sizes are also demonstrated on Figure 5.7a.

Where the formation UC consistently falls below 3.0 in the yielding zone or the required slot size falls below 0.75 mm (0.03 inches), an artificially gravel packed design is usually deemed necessary. There are several well proven empirical methods of arriving at gravel designs; two methods are suggested, along with recommended slot size (Figure 5.7b).

From experimental work, Terzaghi declared that any gravel with a grading between one slightly more uniform than the formation (lower UC) and having a D_{50} size 5 times that of the formation, and one slightly less uniform (higher UC) than the formation and a D_{50} ratio of 8, would satisfactorily retain "the "design" material. Bennison, relying on a series of otherwise derived empirical relationships, concluded that a gravel had to at least retain the D_{85} formation size (to satisfy Terzaghi's earlier observation). He also assumed that for a perfectly uniform gravel filter, its pore size would be 0.4 times the size of the particles forming it and that for a non-uniform gravel, the effective particle size was the D_{10} size. Thus 0.4 x the D_{10} gravel size has to be smaller than the D_{85} formation size (or $D_{10}G \leqslant 2.5 \times D_{85}F$). He further suggested that these relationships work best when the actual gravel had a UC of 2.0; therefore he could define the gravel D_{60} as $\leqslant 5 \times D_{85}F$.

In practice, a gravel has to be specified with small tolerances either side of its "design" grading, simply to allow for variations and inaccuracies in the grading plant process. It is therefore suggested that a "design" gravel be adopted roughly half way between the Terzaghi lower limit and the Bennison upper limit and that these two limits respectively form the tolerance allowances of the specification.

Choice of slot size for an artificial gravel pack must follow the same criteria as for natural development, but with reference to the gravel sizes rather than the basic formation. Thus D_{85} gravel is the maximum theoretically allowed slot but a factor of safety of 2 on this is still recommended to allow for field problems. Half the D_{85} gravel size would then be the suggested design slot; for materials with a Uniformity Coefficient of 2.0, this corresponds to the D_{35} gravel size, not the D_{45} size as for less uniform natural formations. The D_{35} gravel size is therefore the recommended slot size. Screen manufacturer's suggestions again range from the gravel D_{10} to the D_{60} size, presumably for the same reasons as before, but these can be taken as the allowable slot variation in extreme circumstances.

The method of emplacement of an artificial gravel pack really needs to be demonstrated practically but some criteria for successful placement are discussed here. Firstly, there has to be sufficient annular space between the borehole wall and all tubular components, over the joints to allow the gravel free passage downwards; the annular space must also allow a sufficient thickness of in situ gravel for the latter to form a filter. The theoretical filter thickness requirements are about one inch (25 mm), but to ensure this thickness at all points on the screen circumference, the actual annulus has to be much larger, up to 4 inches (100 mm) over screen nominal diameters, while centralising devices have to be used to centre the screen in the bore, irrespective of straightness or verticality. The line between sufficient and excessive annular space is on the other hand also quite fine; gravel thicknesses of over 5 inches (127 mm) probably partially prevent the desired removal of some formation material plus drilling detritus from the formation interface during bore development.

Gravel has to be placed continuously and at a steady rate to avoid segregation of graded material into single grain sizes on its passage down the hole; this leads to 'bridging' and partial blockage of the annulus. If bore water levels are shallow, the act of annular gravel emplacement can induce, by density effects, a downward flow in the annulus plus a compensating upflow inside the screen and casing. This helps to prevent bridging. Gravel emplacement has to be rapid and accomplished in one operation since it tends to scour any fines or mud-cake from the hole walls, reducing the effectiveness of stabilising hydrostatic forces within the hole and increasing the risk of hole collapse. "Spurts" in the counterflow system can be indicative of large formation collapses at depth since they rapidly alter the fluid density characteristics; all temporarily suspended material increases a fluid's density.

Bore development must follow as shortly as possible after gravel emplacement, both to consolidate the total placed volume and to clean out drilling detritus before it sets hard in the interface between gravel and formation.

5.4.4 Choice of Screen Types

There are three basic screen types for use in alluvial bores, as follows:

- those formed with apertures in some kind of mesh wrapped around a perforated base pipe
- those formed with slots cut, punched or otherwise fabricated into the base pipe itself
- those where the base screen is actually built up by wrapping and fixing some kind of profiled helical wire to a base former.

The mesh-aperture systems have little application in good aquifers where production bores are constructed with adequate hydrogeological supervision and after reasonable basic investigations. They may be used during such studies in a hitherto undeveloped area because of their "forgiving" nature.

The "slotted-pipe screens cover a wide range of variants, including sawn and milled slots, punched "louvre" and "bridge" slots and even drilled or perforated slots. The special variants have almost nothing to offer in quality or performance over simple sawn slots in basic casing and sometimes even have drawbacks, particularly with respect to corrosion.

Therefore only screen formed with simple sawn slots in plain casing need be considered and evaluated under the generic term of "slotted pipe".

Of the profiled helical-wrap forms, the commonest and most useful is that constructed by spot-welding a helical wedge-shaped wire (often stainless steel) to a series of rod bases in a cylindrical former. This type has therefore a continuous helical slot and usually a much higher open area than any other type. It is generically classified as "wire-wound" screen.

The obvious differences between wire-wound and slotted-pipe screens are in slot shape and relative slot area. Sawn slotted pipe slots are parallel sided and relatively few in number to preserve the pipe strength. Wire-wound screen slots are wedge-shaped, opening inwards and, being continuous have a proportionately larger open area. The slot shape is claimed both to minimise clogging risks and improve hydraulic entry conditions, so reducing losses; the greater open area further reduces entry velocities which also diminishes hydraulic entry losses.

These facts are undoubtedly true but manufacturer's literature invariably exaggerates their importance. Sone infer, for example, that the 12-20% open area possible with wire-wound screens (depending on slot size) is twice as "good" as the 8-13% open area possible with slotted-pipe screens. This is disputable; the improvement in overall bore performance may only be small, if measurable at all and in special circumstances can even be worse. This is because entry losses across a screen may only be a small component of total well losses as measured during testing. The total losses comprise:

- potential turbulent flow in the gravel approach to the screen
- true "entry" losses at the well screen itself
- losses in classical pipe-flow up the bore

Wire-wound screens do land themselves better to gravel development processes and have better approach-flow patterns, both of which minimise approach-flow losses. They also undisputably have much smaller entry losses. Conversely however their internal hydraulic "roughness" is high with consequently high upflow losses.

Thus in an excellent, high permeability aquifer, a short screen section (L_1) may provide the desired yield (Q). Because of high approach velocities per unit of screen length ($Q \div L$), entry and approach-flow losses will almost certainly predominate over relatively small upflow losses in the short screen. Here, a wire-wound screen type would generally be appropriate. However, if the same yield is to be obtained from a much poorer aquifer, a considerably longer screen length (L_2) might be required to reduce Darcian aquifer losses. The approach velocities would be much lower and since losses are proportional to V^2, the entry plus approach-flow losses would only have a quarter of their former importance, (if $L_2 = 2 \times L_1$). Conversely, upflow losses might double in importance due to greater length of pipe involved. Thus a smooth-bore slotted-pipe screen may actually be superior in overall performance to a wire-wound screen. A rough guide is that for aquifer permeabilities below 200 m/d and typical irrigation production bore yields, slotted pipe will be equal or superior to wire-wound. Above 400 m/d, a wire-wound screen will be superior. Between the two values, detailed checking of cost effectiveness is required.

One way to examine this cost-effectiveness is to quantify the bore upflow losses. An equation has been empirically derived for these, based on analysis of hundreds of bore pumping tests in Bangladesh, where the variables can be separately statistically analysed. This allows for some concentration of unit inflow towards the top of the bore because drawdown is effectively increased upwards by the increasing upflow losses. An equation for upflow loss is based on the well-known Manning equation and takes the form:

$$h^1 = \frac{n^2Q^2L}{0.525d^{16/3}} \qquad (4)$$

where h^1 = upflow loss in m

Q = discharge in m^3/sec.

L = lower bore casing and screen length, m

d = lower bore casing and screen diameter, also in m

n = metric equivalent of Manning's 'n' (= 1.4859 x n imp.)

The Bangladesh work suggested a value for Manning's 'n' in ft/sec units of 0.013 (0.0193 in metric units) for screen of mandrel-formed GRP slotted pipe whose basic 'n' as unslotted pipe might be 0.010 (0.0148 in metric units). That is, because of the extra turbulence induced by sideways entering flow and the slot cutting, the slotted pipe screen behaves as if it were approximately 30% greater in hydraulic roughness than the equivalent plain pipe.

The hydraulic rugosity of wire-wound screen is clearly enormously higher than that of the almost mirror-finish bore of mandrel-formed GRP. Observation has indicated that the approximate internal Mannings, 'n' values for different screens may be as follows (metric units):

Slotted GRP - $n = 0.0193$
Slotted PVC - $n = 0.0212$
Slotted steel - $n = 0.0251$
Punched steel - $n = 0.0328$ ("bridge" & "louvre")
Wire-wound - $n = 0.0406$

Table 5.4 is derived by substitution of the above values into equation (4) for diameters close to the optima and for screen lengths equivalent to a 5.0 metre aquifer drawdown; a 20% open screen area was assumed and use was made of the uncorrected Logan approximation. It can be seen that the wire-wound screen upflow losses are significantly higher than the slotted pipe losses in every case (Table 5.4; Columns 7 and 9).

It is considerably more difficult to derive a universal equation for entry losses, since these vary with aquifer permeability, gravel pack design and how effective bore development has been in removing the damaging effects of the drilling process on the aquifer (drill mud invasion etc). For the conditions in which equation (4) was derived (moderate aquifers with RC water flush drilling), the following estimate for entry losses has been made:

$$h_e = \frac{1000F\ Q^2}{L^2\ d^2} \qquad (5)$$

where, F is a factor and the other parameters have the same definitions and units as in equation (4). For plain slotted pipe, F can generally be reduced by development to a value of about 5.0. It is assumed that wire-wound screens, which have 1.5 times the open area of slotted screens (2.25 in area^2 terms), are actually five times better at reducing entry losses; they would therefore have an F-value of 1.0. Table 5.4 shows the calculated entry losses and sums entry and upflow losses to give total well losses. Those configurations where wire-wound screens are superior are marked with an asterisk.

Table 5.4 Calculated Well Entry, Upflow and Total Losses

K (m/d)	Q (l/s)	Screen		Well Losses (m)					
		Length (m)	Diam. (m)	Slotted PVC			Wire-Wound		
				Entry	Upflow	Total	Entry	Upflow	Total
500	80	16.86	0.20	2.81	0.49	3.30	0.56	1.81	2.37*
"	"	"	0.25	1.80	0.15	1.95	0.36	0.55	0.91*
"	60	12.65	0.20	2.81	0.21	3.02	0.56	0.76	1.32*
"	"	"	0.25	1.80	0.06	1.86	0.36	0.23	0.59*
300	80	28.11	0.20	1.01	0.82	1.83	0.20	3.02	3.22
"			0.25	0.65	0.25	0.90	0.65	0.92	1.57
"	60	21.08	0.20	1.01	0.35	1.36	0.20	1.27	1.47
"	"	"	0.25	0.65	0.11	0.76	0.65	0.39	1.04
150	80	56.22	0.20	0.25	1.65	1.90	0.05	6.03	6.08
"	"	"	0.25	0.16	0.50	0.66	0.03	1.84	1.87
"	60	42.16	0.20	0.25	0.69	0.94	0.05	2.55	2.60
"	"	"	0.25	0.16	0.21	0.37	0.03	0.77	0.80
"	40	28.11	0.15	0.45	0.95	1.40	0.09	3.50	3.59
"	"	"	0.20	0.25	0.21	0.46	0.05	0.75	0.80
75	60	84.32	0.20	0.06	1.39	1.45	0.01	5.09	5.10
"	"	"	0.25	0.04	0.42	0.46	0.01	1.55	1.56
"	40	56.22	0.15	0.06	1.91	1.97	0.01	7.00	7.01
"	"	"	0.20	0.04	0.42	0.46	0.01	1.51	1.52
40	60	158.11	0.20	0.01	2.60	2.61	Nil	9.54	9.54
"	"	"	0.25	0.005	0.79	0.80	"	2.90	2.90
"	40	105.41	0.15	0.01	3.58	3.59	"	13.12	13.12
"	"	"	0.20	0.005	0.77	0.78	"	2.83	2.83
"	20	52.70	0.10	0.01	3.88	3.89	"	14.25	14.25
"	"	"	0.15	0.005	0.45	0.46	"	1.64	1.64

Clearly a considerable number of tests have to be analysed to derive an equivalent to equation (5) for local conditions but the rough permeability guidelines given earlier are borne out, even for entry losses up to twice as high for each screen type. The foregoing calculations refer to gravel-packed production bores and for them, the conclusion is inescapable; that wire-wound screens are only justified at aquifer K values over 400 m/d.

For naturally developed bores on the other hand, the entry loss reduction achievable with the wire-wound system, as compared to slotted pipe, may be much greater than the factor of 5 previously used. That is, slotted pipe screen designs are not physically suited to the development methods necessary for this form of completion, namely high-pressure water jetting in conjunction with airlift discharging. With slotted pipe, the energy in this process simply cannot be transferred through the screen to liquidise the surrounding aquifer locally; the wedge-shape and continuous nature of wire-wound screens are essential for this process. Thus where natural development can be carried out, it is the preferred system because of reduced hole diameters and gravel cost savings; because of these considerable advantages wire-wound screens may be appropriate to medium aquifer permeabilities. At the lower permeabilities associated with medium and fine sands, the formation will usually anyway need an artificial gravel pack. A summary of production bore completion criteria is given (Table 5.5).

The last precautions on screen selection concern materials. Because of their very small dimensions and allowable tolerances, screen slots are particularly susceptible to corrosion damage, both by becoming blocked with corrosion products remaining in situ or by becoming enlarged if these products are carried away. Bi-metallic corrosion processes are usually predominant; bi-metallic differences as small as the difference between a weld and its parent material and between an exposed saw-cut section of metal and its adjacent mill-rolled surface can cause corrosion cells. For this reason, wire-wound screen should be at least hot-dip galvanised (not cold galvanised) to protect the wire-to-rod welded contacts, or better still, fabricated from stainless steel.

Table 5.5 Summary of Criteria for Bore Completion

Completion Type	When Indiated	Remarks, Advantages
Natural Development	Non-uniform, coarse, well graded, sediments; UC < 3	Generally the preferred method. Cost savings through reduced drill diameters, lack of gravel placement. Development easier. Slotted pipe unsuited to necessary development methods. Wire-wound screen preferred, particularly for high permeability aquifers. Because of other advantages, natural development may be used in medium K aquifers ($K \lesssim 100$ m/d).
Artificial Gravel Pack	Fire, uniform sediments; UC $\leqslant 3$ where indicated slot size < 0.75 mm. Poor sample control or running fine sands in aquifer zone.	Higher costs because of greater drilling diameters, and gravel placement. (Slotted pipe acceptable, particularly at lower permeabilities. Development more difficult particularly if slotted pipe used but, in terms of head losses, wire-wound screen only justified with high permeability aquifers of $K = 400$ m/day.

It has been found that slotted steel is exceedingly susceptible to even the most midly corrosive waters due to the cut-uncut metal bi-metallic differences. Slot sizes can change by one or two millimeters in the life time of a bore, both downwards by chemical incrustation triggered by corrosion products and upwards by corrosion product removal. Thus slotted steel pipe with slots less than about 3 to 4 mm is almost never used unless long-term local history supports its suitability.

For smaller slot sizes, the base pipe for slotted screens is usually an inert material such as GRP or thermoplastic. Epoxy resin GRP is considered the best fibreglass material because of its resistance to water-damage to the reinforcing fibres at the cut slot faces. Thermoplastics (PVC, ABS and polyolefin mixtures) can be used for gravel-packed bore applications but extreme care is neeeded in selecting them for strength, rigidity and dimensional stability during storage and handling.

Waterworks quality plastic pressure pipe is useless for well screen; proprietary systems specifically designed for screen duty should always be used.

Verticality and straightness are of little importance in screen below pump settings, but the screen must be capable of being centred in the drilled hole to allow satisfactory gravel pack emplacement. Therefore it must remain absolutely straight during storage and handling and not bend due to temperature effects. It will be certainly be violently pushed or pulled if there is bore partial collapse during construction. At least 0.04 Tonnes of joint-stripping load resistance is considered necessary to cope with this. It should also have threaded joints to bore casing straightness standards with the least possible outside diameter. It will also be subject to sudden increases in uniform external radial pressures at the start of well development when surrounding material moves instantaneously. International criteria are variable on this subject, but at least an external pressure collapse resistance requirement of 0.35 lb/in^2 per foot of hole total depth (0.08 kg/cm^2 per metre of hole depth).is generally agreed upon. External pressures are far more difficult for plastic pipe to resist than internal pressures, because of buckling effects; the internal/external pressure rating ratio is generally at least a factor of 4. Water supply pressure pipe cannot meet these standards.

5.4.5 Bore Configurations

In this section, configurations for natural and artificial gravel packed production bores are given. The designs are based on limiting discharges for given diameters proven by experience elsewhere in the world.

Table 5.6 shows the diameters of screen and pump casing components which are considered to be the minimum acceptable diameters for the stated desired discharges. Pump casings are considered to be standard steel with welded or screwed couplings to the usual straightness and verticality requirements. Lower bore casing and screen is stated to a nominal diameter (ND), on the assumption that actual ID's will be at least equal to the nominal. Wire-wound screen couplings can be obtained to normal industry standards, or welded, but plastic screen couplings vary greatly from manufacturer to manufacturer.

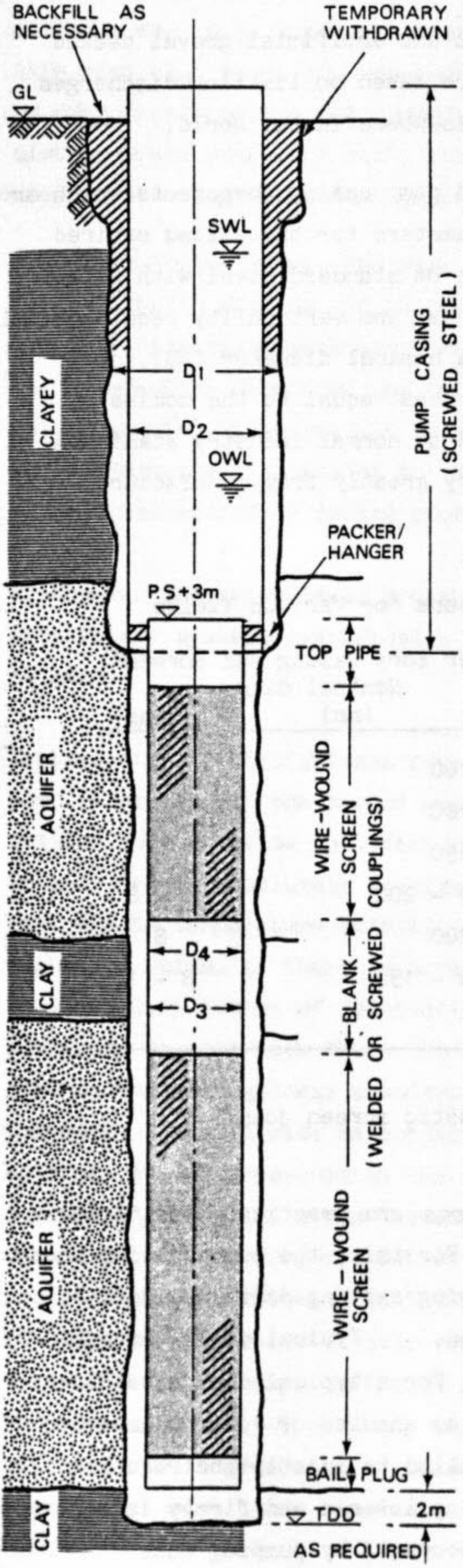
Table 5.6 Preferred Component Diameters for Various Yields

Discharge Q (l/s)	Pump Casing I.D. (mm)	Lower Bore Casing and Screen* Nominal diam. (mm)	(ins)
1 - 8	156	100	4
15	156 - 206	100	4
25	206 - 256	150	6
30	256	150 - 200	6 - 8
50	256 - 311	200	8
≥ 60	311	100 - 150	8 - 10

<u>Note:</u> *for Wire-wound screen; plastic screen joint sizes are variable.

For situations where naturally developed bores are practical, the "drill-through" process is strongly recommended. For this, the pump setting needs to be evaluated in advance and the pump casing setting depth decided. A standard two-string hole is then completed. Typical drill-through designs are illustrated on Figure 5.8. For a typical design, a hole with a very small annular clearance (say 8 mm annular or 16 mm diametric) over the pump casing couplings is first drilled to exactly the required pump casing setting depth and the pump casing lowered and firmly landed on the bottom; annular collapse is then encouraged by pumping out.

FIGURE 5.8 NATURALLY DEVELOPED ALLUVIAL BORE DESIGNS



Recommended Diameters

Discharge, Q (l/s)	D_1 Upper Hole diam. (mm) plus equivalent bit size (ins).	D_2 Pump Casing ID. (mm).	D_3 Lower Hole diam. (mm) plus equivalent bit size (ins).	D_4 Screen nominal diam. (mm).
1-8	194 (7 5/8)	156	149 (5 7/8)+	100
15	251 (9 7/8)	156-206	149 (5 7/8)	100
18	251 (9 7/8)	206	149 (5 7/8)	125
25	251 (9 7/8)	256	194 (7 5/8)	150
30	311 (12¼)	256	251 (9 7/8)	200
50	356 (14)	311	251 (9 7/8)	200
≥60	356 (14)	311	292 (11½)	250

Note:

Total screen lengths to be determined from desired discharge and economic design criteria.

Pump casing may need lower end plug-cementing to prevent washout when drilling on through.

156 Available JPT bit and casing sizes

+ Smallest Available.

The lower hole for the screen and lower casing string is then drilled through the pump casing, again with smallest feasible annular clearance over actual coupling diameters. The casing and screen string is quickly placed through the pump casing using a back-off tool, a fabricated hanger arrangement plus a tight seal between the two casing strings; this can be a push-fit rubber arrangement or a deformable soft metal (lead) ring which is expanded or "swedged" into place. The make-up of blank casing and wire-wound screen in the lower string is decided on the basis of the studies of formation samples, geophysical logs and sieve analyses which are interpreted to delineate screenable aquifer. Sometimes it may be necessary to have available screens of several slot sizes to match vertical changes in formation grading.

The bore should be immediately developed by a combination of jetting, air-lift surging and overpumping until its specific capacity shows no significant improvement and the sand-content criteria of the discharge are met. Development may take some considerable time with re-jetting producing sand each time after overpumping phases have stabilised it, but should be continued until no more sand show is obtained. The bore can then be step-tested for pump selection purposes.

For gravel packed designs, either a drill-through or a single-string configuration with a casing reducer could be adopted depending on required yields and diameters. However, if near the top of a discharge range, the drill-through technique would usually require the pump chamber casing to be oversize to accommodate the annulus necessary to adequately place the gravel pack. This should be at least 3 inches (75 mm) over screen couplings and since these can be 30 to 40 mm over ID in some plastic materials, a diameter difference of 180 to 190 mm between the nominal bores of the two components is required. This is rare and so drill-through gravel packs are infrequently used except when special pump conditions demand large pump casing or very slim-coupling screen is used. Drill-through configurations can easily be worked out however from the criteria already given, but using flush-jointed screens.

The normal gravel packed design is for the hole to be drilled in one or two passes (possibly with a slight diameter reduction just below pump casing for very deep bores of over 100 m) and for the pump casing and lower bore casing and screen to be placed in a single string connected by a concentric reducer. If the two-pass, two-diameter system is used, the upper hole should be drilled with a hole-opener or two-stage bit to ensure concentricity of the two holes.

The minimum required gravel-placing annulus is 3 inches (75 mm) over screen couplings or just less than 4 inches (100 mm) over screen internal or nominal diameters. Some 2 inches (50 mm) over pump casing couplings is also considered a minimum to allow the gravel to pass downwards, which translates to some 2.5 inches (65 mm) over casing ID's.

After the single-string casing has been lowered and hung from the surface with a 2 m end-float beneath it (to prevent screen string buckling), the gravel is immediately emplaced, observing the precautions noted earlier regarding general difficulties. The screen string is fitted with centralising devices to preserve its concentricity with the hole, but none are fitted to the pump chamber and as soon as gravel placement has been completed to above the reducer connection, the pump chamber is positively tensioned from the rig to preserve its verticality. Backfill in the pump chamber annulus may be any suitable or sealing material; clay fill in this context is almost impossible to place to an adequate density or compaction.

The bore is immediately developed by localised airlift surging and overpumping and acceptance/production pump selection tests carried out as before. Typical configurations for this form of completion are illustrated on Figure 5.9.

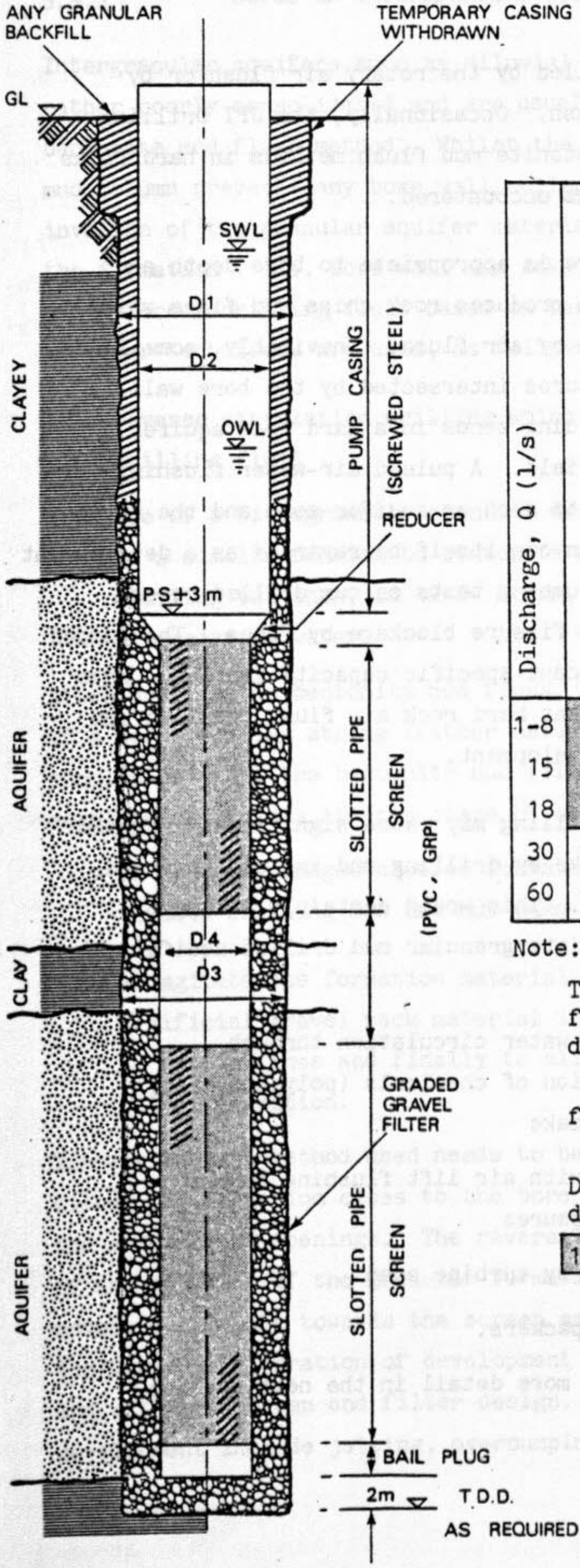
5.5 Bore Development

5.5.1 General

The purpose of development is to minimise the effects of the drilling process upon the hydraulic performance of the completed bore, whether this is an open hole completion in a hard rock fissure aquifer or a screened completion in an intergranular aquifer. In the second case, development has the second important function of removal of fines from the formation leading to production of a stable filter.

Bore development can often lead to some improvement in bore specific capacity. Short drawdown-discharge tests may be used to evaluate bore specific capacity during the development process and these provide an indicator of the success of the development process.

FIGURE 5.9 GRAVEL PACKED ALLUVIAL BORE DESIGNS



Recommended Diameters

Discharge, Q (l/s)	D_1 Upper Hole diam. (mm) with equivalent bit size (ins)	D_2 Pump Chamber ID. (mm).	D_3 Lower Hole diam. (mm) with equivalent bit size (ins).	D_4 Screen Nominal diam. (mm).
1-8	244 (9 5/8)	156	244 (9 5/8)	100
15	311 (12¼)	206	311 (12¼)	100
18	311 (12¼)	206	311 (12¼)	125
30	406 (16)	256	356 (14)	150
60	457 (18)	311	406 (16)	200

Note:

Total screen lengths to be determined from desired discharge and economic design criteria.

- 12¼ ins. is about smallest feasible R.C. Drilling diameter.

- Drag bits for larger R.C. Drilling can be made up to any desired diameter.

▨ 156 Available JPT bit and casing size.

5.5.2 Hard Rock Bores

Hard rock bores are commonly drilled by the rotary air flush or by down the hole hammer with air flush. Occasionally, the JPT Drilling Section has had to use rotary bentonite mud flush methods in hard rocks where extreme bore instability was encountered.

Provided air capacity and pressure is appropriate to bore depth and diameter, the air drilling action produces rock chips and fines which are largely cleared from the bore by air flush. Inevitably, some fines come to lodge in and clog up fissures intersected by the bore wall. Since these fissures are the yielding zones in a hard rock aquifer, their blockage will reduce bore yield. A pulsed air-water flushing invariably follows penetration into such an aquifer zone and the action of the drill air-water circulation can itself be regarded as a development process. Certainly, subsequent pumping tests on our drilled bores have not demonstrated significant fissure blockage by fines. The pumped discharge is clean and no significant specific capacity increases have been noted. It seems therefore that hard rock air flush drilling can produce bores requiring little development.

The use of bentonite mud flush drilling may cause significant invasion of the yielding fissures and cracks by drilling mud and cuttings and development work will be required. This would certainly include techniques normally employed for intergranular mud drilled aquifers namely:

- bentonite mud removal, clean water circulation through the bore with possible addition of chemicals (polyphosphate) to break down bentonite mud cake
- high velocity water jetting with air lift flushing across the main zone of yielding fissures
- overpumping and back washing by turbine pump
- localised development using packers.

These techniques are described in more detail in the next section.

5.5.3 Bores in Intergranular Aquifers

Intergranular aquifers such as alluvial sand and gravels are invariably rather poorly consolidated and are usually drilled by the rotary, bentonite mud flush method. Whilst the hydrostatic pressure of the mud column prevents any bore wall collapse, it also leads to some invasion of the granular aquifer material and consequent sealing of the formation (i.e. bore wall mud cake). Formation damage by invasion of a viscous drilling fluid based on bentonite and water can be avoided if other techniques are used, as follows:

- reverse circulation drilling which uses water alone as the drilling fluid
- use of a biodegradable chemical drilling mud which functions in a similar fashion to bentonite during normal circulation drilling but then, in 12-24 hours, reverts to a fluid of the viscosity of water.

After drilling by bentonite mud flush, the bore is completed by installation of a screen-casing string (either natural or artificial gravel pack completion) into the bentonite mud filled bore. The task of the development process at this stage of construction is as follows:

- to restore damaged aquifer formation, by removal of invaded drilling mud and mud cake
- to agitate the formation material or formation-artificial gravel pack material in the screen zone, to remove fines and finally to allow stable, permeable filter formation.

The development method used needs to be able to agitate violently the filter and formation close to the bore and cause reversal of flow through the screen slot openings. The reversal of flow causes the break down of any bridging of the granular formation during out-flow and the movement of fines towards the screen and into the well during inflow. The degree and duration of development depends on the drilling process and with the screen and filter design. Recognised methods of development include jetting, overpumping, backwashing and surging.

In the jetting method, the screens are jetted with air or water at high velocity, up to 60 m/s. The jetting tool consists of a number of nozzles pointing radially outwards and extending to within 25 mm of the well screen. The jetting tool is suspended from a drop pipe which carries the supply of water or air. While jetting takes place the tool is slowly turned and raised through the screened length. Although jetting with water is common,jetting with air has the advantage that the air causes the water to be lifted out of the bore and with it the fine material brought into the bore by the development process. With air jetting backwashing can also be carried out. Jetting is thought less effective when directed at slotted pipe screens with rather low open areas.

If a suitable pump is available at the time of bore completion, further development by a combination of overpumping and backwashing should be carried out. The procedure followed is generally to pump the bore at several different discharges up to 1.5 to 2 times its design discharge and to induce the reversal or backwashing intermittently at each stage. The flow reveral is achieved by stopping the pump and letting the water run back into the bore through the pump column pipe. The flow reversal is more positive if the air lift method of pumping is used and development using a modified air lift pumping arrangement is common.

Surging can also be achieved by operating a plunger up and down in the casing like a piston in a cylinder. The tool normally used is called a surge plunger or a surge block. The surge block must be used with care since the surging can produce high differential pressures which may collapse the screen if it is partly or wholly plugged by clay or mud. Following surging any material brought into the bore is removed by bailer.

Development is judged to be completed when no further increase in bore performance can be achieved and no sand or cuttings are pumped from the bore at the design discharge. Sand content should be no more than 30 parts per million (ppm) of solids in the discharge within the first 2 minutes of pumping at the maximum rate; sand content should fall rapidly to less than 5 ppm after 5 minutes.

5.6 Borehole Headworks

5.6.1 General

This section deals briefly with the civil engineering works required at the head of the production bore to protect and ease maintenance of the bore and associated pumping plant, and to feed pumped water into the irrigation distribution system at a controlled rate. The type of pumping plant used in the bore partly contols the complexity and cost of pump headworks required. Lineshaft surface driven pumps require considerable headworks (pumphouse, fuel storage etc.) whilst electric submersible headworks can be simple.

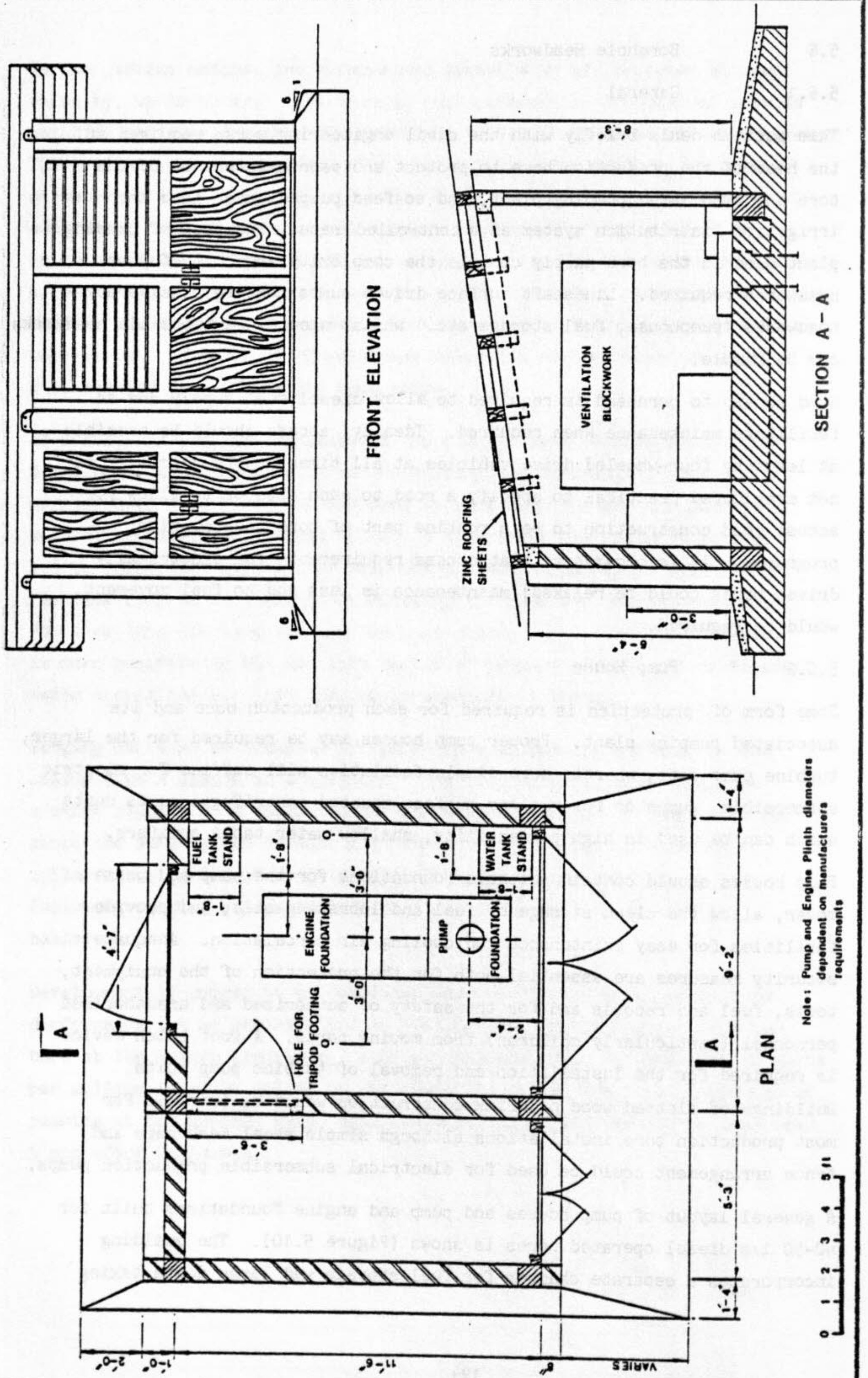
Good access to borehead is required to allow diesel fuel supply and to facilitate maintenance when required. Ideally, access should be possible at least by four-wheeled drive vehicles at all times of the year. It is not considered practical to specify a road to each site or to allow for access road construction to be a routine part of bore construction programme. It is considered that access requirements for electrically driven bores could be relaxed; maintenance is less and no fuel movement would be required.

5.6.2 Pump House

Some form of protection is required for each production bore and its associated pumping plant. Proper pump houses may be required for the larger, turbine pump sets, whereas more simple facilities will suffice for electric submersible pumps or for smaller surface-mounted centrifugal pumps units which can be used in high permeability, shallow water table aquifers.

Pump houses should contain suitable foundations for the pump and prime mover, allow the clean storage of fuel and lubrating oils, and provide facilities for easy maintenance and cooling air circulation. Adequate security measures are essential both for the protection of the equipment, tools, fuel and records and for the safety of authorised and unauthorised personnel (particularly children) from moving parts. A roof hatch device is required for the installation and removal of turbine pump units. Buildings of slotted wood or brick construction would be suitable for most production bore installations although simple steel mesh cage and fence arrangement could be used for electrical submersible production pumps.

A general layout of pump houses and pump and engine foundations built for 40-50 l/s diesel operated bores is shown (Figure 5.10). The building incorporates a separate chamber for fuel storage and large doors taking


FRONT ELEVATION
6
6
SECTION A–A
VENTILATION BLOCKWORK
ZINC ROOFING SHEETS
8'-3"
3'-0"
6'-4"
PLAN
FUEL TANK STAND
WATER TANK STAND
ENGINE FOUNDATION
PUMP
PUMP FOUNDATION
HOLE FOR TRIPOD FOOTING
4'-7"
1'-8"
1'-8"
3'-0"
3'-0"
1'-8"
1'-4"
1'-4"
6'-2"
5'-3"
1'-4"
3'-6"
2'-0"
1'-0"
2'-0"
11'-6"
6"
VARIES
A
A
Note: Pump and Engine Plinth diameters dependent on manufacturers requirements
0 1 2 3 4 5 ft

up a whole wall which are left open whilst pumping, thus increasing engine ventilation. The roof is constructed in three panels to allow removal at times of pump installation and inspection. Overall dimensions are based on experience and are generally limited by requirements for accessibility of tools (particularly chain tongs use for unscrewing the pump column), movement area for starting handle operation and space for any cooling water or pump and drive shaft lubration water.

Where electrically driven pumps are feasible, pump house dimensions and requirements are substantially reduced. All cabling should be buried or put in a conduit/duct and the switchgear/control box should be made weather-proof and lockable. A small steel mesh cage, with a top plated area to give shade to the switchgear can be used. It is worthwhile having a lockable on-off switch on the outside of the switchgear/control box so that the farmer-operator has no access to other controls; switchgear can be mounted on the incoming power pole.

5.6.3 Discharge Box

A discharge box is required to still a turbulent pump discharge prior to its entering the distribution system, and to measure changes in pump and bore performance both at different groundwater levels within the pumping season and to assess long term trends. If so designed, a discharge box also provides a means of dividing the pump flow accurately.

Flow measurement is determined from the head over a measuring weir installed at the end of the discharge box. Three types of weir can be considered:

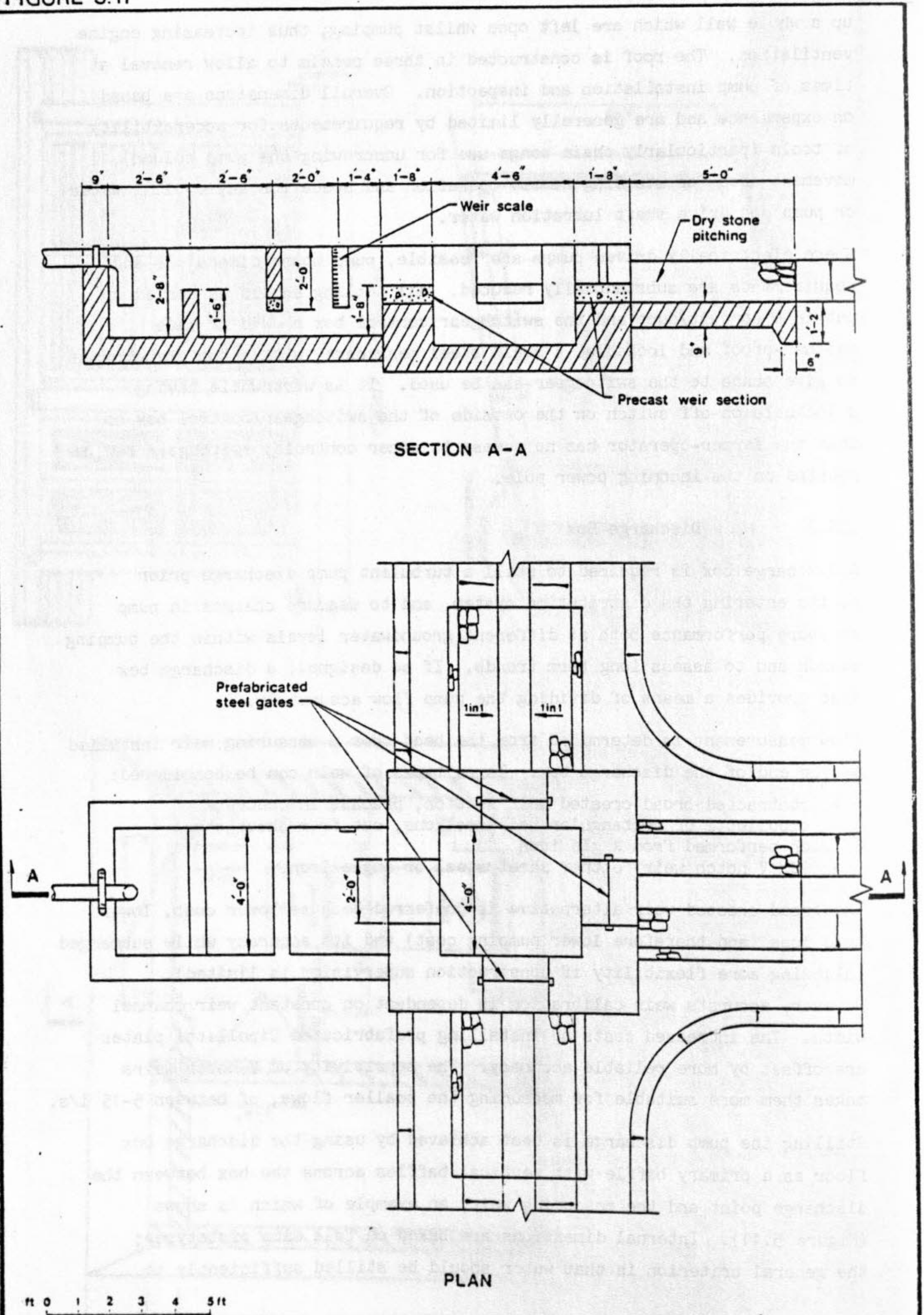
- contracted broad crested weir section, precast in concrete
- cipolletti or rectangular weir sections, cut from sheet steel or performed from angle iron
- 90° V notch weir, either sheet steel or angle iron.

The broad crested weir alternative is preferred because lower cost, lower head loss (and therefore lower pumping cost) and its accuracy while submerged (allowing more flexibility if construction supervision is limited). However, accurate weir calibration is dependent on constant weir channel width. The increased costs of installing prefabricated Cipolletti plates are offset by more reliable accuracy. The sensitivity of V notch weirs makes them more suitable for measuring the smaller flows, of between 5-15 l/s.

Stilling the pump discharge is best achieved by using the discharge box floor as a primary baffle with vertical baffles across the box between the discharge point and the measuring weir; an example of which is shown (Figure 5.11). Internal dimensions are based on full size prototypes; the general criterion is that water should be stilled sufficiently to

FIGURE 5.11

TYPICAL DISCHARGE BOX



allow accurate water level reading at the scale upstream of the measuring weir.

Discharge box levels are governed by the water level in the channel system downstream of the measuring weir. The general assumption is that the typical downstream water depth is 1 ft and the weir crest is set at this level. The overall dimensions will depend on discharge. Tentative dimensions are shown (Figure 5.11)for flows up to 55 l/s. Box and weir dimensions are based on 0.5 ft. (15 cm) freeboard and a suitable head over the weir, based on standard weir tables. If the bore is set at the head of two or more channels, the required control box can be added to the discharge box below the measuring weir. The secondary baffle shown requires a wooden or concrete lintel. An alternative is to build a 25 cm wall up from the floor of the box with honeycomb brickwork up to a certain height to allow water to pass through.

During construction provision must be made for the pump discharge pipe. This is sized to limit the velocity in the pipework to about 3 m/s; typically, a 20 cm pipe for a discharge greater than 60 l/s. The outlet of the discharge pipe should be submerged at least 10 cm below the operating water level in the discharge box and should be set at least 1.5 times its diameter above the floor of the box. The pipe should also be arranged to allow vertical discharge into the box and the radius of any bends in the pipework should be set at a minimum of twice the pipe diameter. A flexible joint is required in the pipework between the pump head and the discharge box to ease alignment and reduce any vibration in the system. This is achieved with a Viking Johnson type coupling fitted close to the pump outlet flange inside the pump house. The connection between the discharge box and the channel system is provided by a rectangular channel lined with brickwork. The feeder channels should be pitched to their natural section for distances of up to 3 m each direction as appropriate.

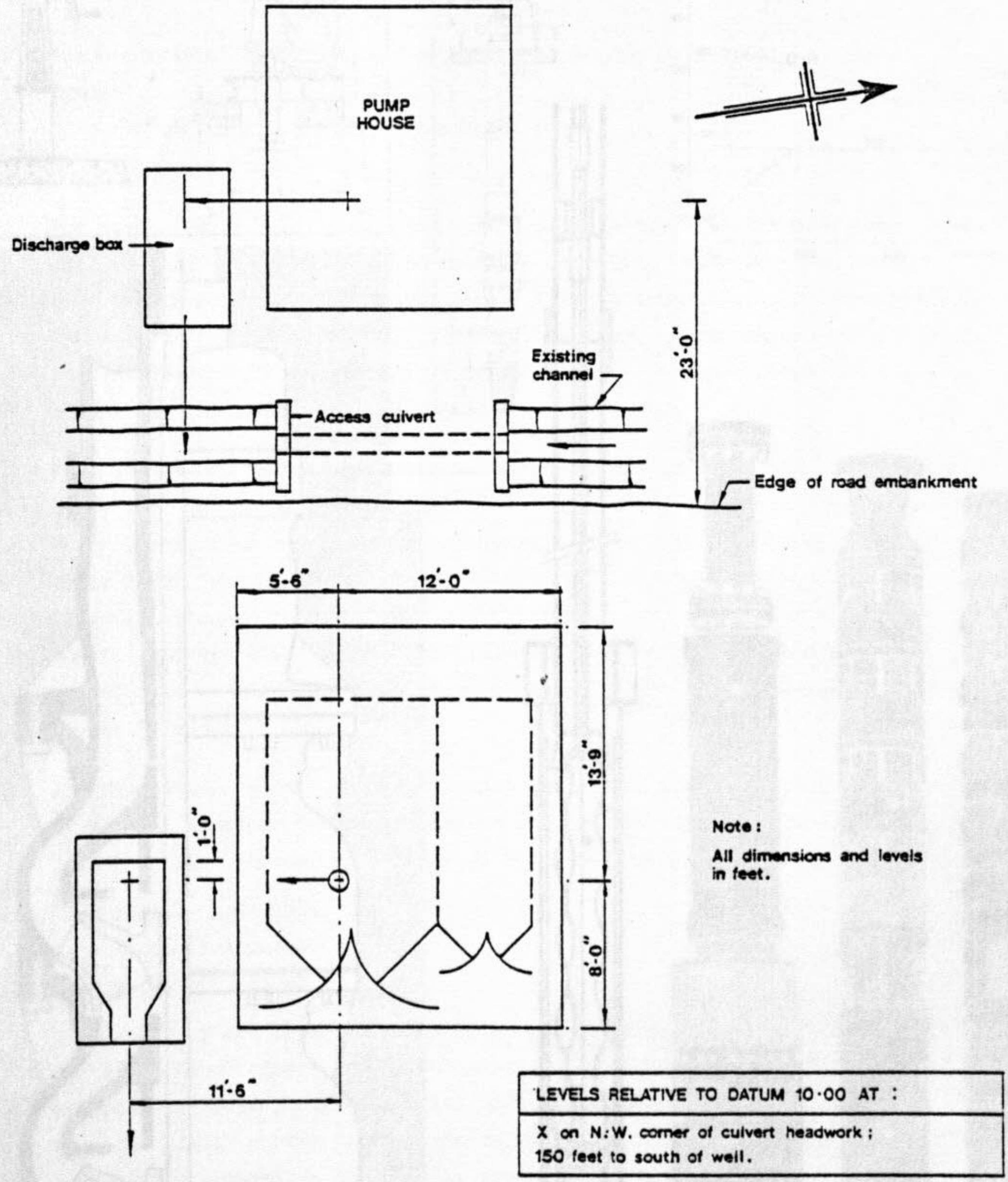
Discharge box facilities for small capacity pumps up to say 15 l/s, should be much simpler. If soils are heavy enough, it is possible that a small earth bunded pond with a V notch at the downstream end will suffice; some bed protection may be required depending on the pump discharge pipe arrangement.

Discharge boxes frequently become objects of great local interest, particularly for washing clothes and to playing children. Flow over a weir is frequently fast flowing and there is a potential drowning hazard. Consideration should be given to incorporating a washing place in addition to the discharge box.

5.6.4 General Layout

The orientation of a pump house and discharge box group will depend on the layout of roads and channels. Access should be adequate for maintenance vehicles, including mobile cranes, to reach the borehole. It is possible that a culvert will be required to cross a canal, flowing parallel to an existing road, into which the pump will discharge. The minimum access road width should be 3 m and generally pipe diameters of 30-45 cm are suitable depending on the channel to be crossed. Pump houses can be built as a mirror image of that shown on Figure 5.10. This generally allows sufficient versatility when setting out a bore group. Figure 5.12 shows a sample for bore layout design for a particular pump house discharge box configuration.

FIGURE 5. I2 SAMPLE BORE ARRANGEMENT



Note:
All dimensions and levels in feet.

LEVELS RELATIVE TO DATUM 10·00 AT :

X on N.W. corner of culvert headwork ;
150 feet to south of well.

TYPE OF PUMP	GH 100/2
TYPE OF DISCHARGE BOX	A (2 cusecs)
TYPE OF CULVERT	16'-6" 1'-4"Ø

PUMP HOUSE FLOOR LEVEL	7·87
DISCHARGE BOX WEIR LEVEL	7·54
OUTLET PIPE LEVEL (AXIS)	9·21
UPSTREAM CULVERT INVERT LEVEL	6·06

6. PUMP SELECTION

6.1 Basic Pump Types

Pumps suitable for irrigation wells and bores can be divided into two types:

- simple centrifugal surface mounted pumps
- submerged pumps

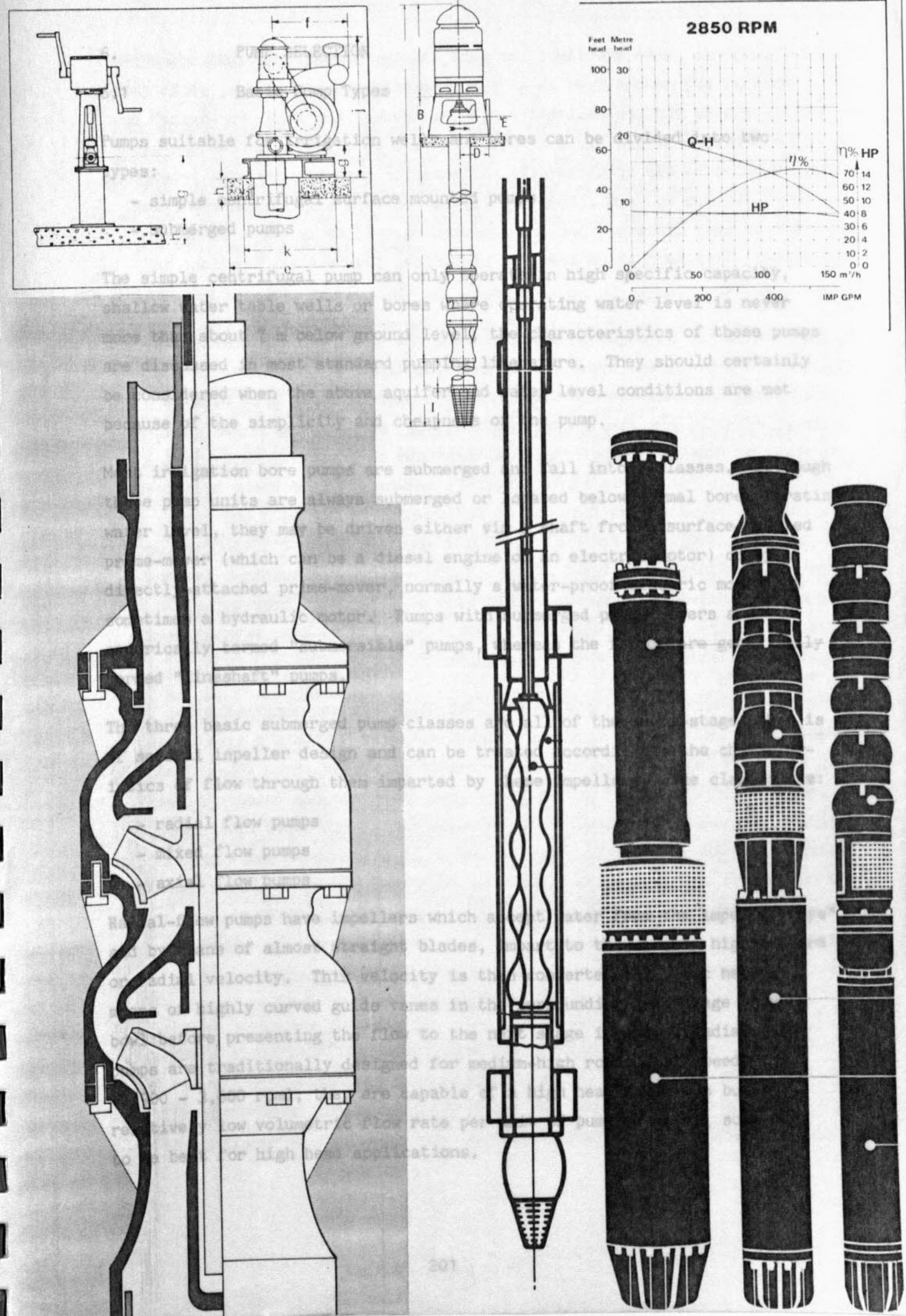
The simple centrifugal pump can only operate in high specific capacity, shallow water table wells or bores where operating water level is never more than about 7 m below ground level; the characteristics of these pumps are discussed in most standard pumping literature. They should certainly be considered when the above aquifer and water level conditions are met because of the simplicity and cheapness of the pump.

Most irrigation bore pumps are submerged and fall into [illegible] classes. Although these pump units are always submerged or located below normal bore operating water level, they may be driven either via a shaft from a surface mounted prime-mover (which can be a diesel engine or an electric motor) or [illegible] directly-attached prime-mover, normally a water-proof electric motor [illegible] sometimes a hydraulic motor. Pumps with submerged prime-movers are generically termed "submersible" pumps, whereas the [illegible] are generally termed "lineshaft" pumps.

The three basic submerged pump classes are all of the multi-stage [illegible] is [illegible] impeller design and can be treated according to the characteristics of flow through them imparted by these impellers. The classes are:

- radial flow pumps
- mixed flow pumps
- axial flow pumps

Radial-flow pumps have impellers which accept water [illegible] "eye" and by means of almost straight blades, impart to the water high [illegible] or radial velocity. This velocity is then converted [illegible] head [illegible] means of highly curved guide vanes in the surrounding stage bowl before presenting the flow to the next stage impeller. Radial pumps are traditionally designed for medium-high rotational speeds [illegible] - 3,500 r.p.m.; they are capable of a high head [illegible] but relatively low volumetric flow rate per [illegible] pump [illegible] to be best for high head applications.



6. PUMP SELECTION

6.1 Basic Pump Types

Pumps suitable for irrigation wells and bores can be divided into two types:

- simple centrifugal surface mounted pumps
- submerged pumps

The simple centrifugal pump can only operate in high specific capacity, shallow water table wells or bores where operating water level is never more than about 7 m below ground level; the characteristics of these pumps are discussed in most standard pumping literature. They should certainly be considered when the above aquifer and water level conditions are met because of the simplicity and cheapness of the pump.

Most irrigation bore pumps are submerged and fall into 3 classes. Although these pump units are always submerged or located below normal bore operating water level, they may be driven either via a shaft from a surface-mounted prime-mover (which can be a diesel engine or an electric motor) or by a directly-attached prime-mover, normally a water-proof electric motor but sometimes a hydraulic motor. Pumps with submerged prime movers are generically termed "submersible" pumps, whereas the former are generically termed "lineshaft" pumps.

The three basic submerged pump classes are all of the multi-stage, that is of several inpeller design and can be treated according to the characteristics of flow through them imparted by these impellers. The classes are:

- radial flow pumps
- mixed flow pumps
- axial flow pumps

Radial-flow pumps have impellers which accept water from the impeller "eye" and by means of almost straight blades, impart to the water a high outward or radial velocity. This velocity is then converted to static head by means of highly curved guide vanes in the surrounding pump-stage body or bowl before presenting the flow to the next stage in line. Radial flow pumps are traditionally designed for medium-high rotational speeds (2,900 - 3,600 rpm); they are capable of a high head per stage but a relatively low volumetric flow rate per unit of pump diameter, so tend to be best for high head applications.

Axial-flow pumps have impellers (or more nearly, propellers), which accept water axially and simply five it a "push" towards the next stage, while guide vanes are simple and are designed simply to prevent too much "vortexing" or rotational motion in the fluid stream. In slim bore pump applications at least, such pumps are traditionally designed for high to very high rotational speeds (2,500 - 5,000 rpm); they are capable of very high volumetric flows per unit of pump diameter, but are somewhat restricted in terms of head per stage. They tend to be best for high-flow, low-head applications in slim boreholes.

Mixed-flow pumps, as might be expected from the name, exhibit characteristics between the two extreme types. They have impellers which deliver both a radial and an axial component to the flow in each stage and hence also utilise guide vanes which can straighten the flow for the next stage. The great advantage of mixed flow type pumps is that they have traditionally been associated with relatively rigid lineshaft drives and have hence been designed to cover all speed ranges from low to medium-high (1,200 to 3,600 rpm). They are thus the ideal pump for driving with a standard 4-pole, 1,450 rpm electric motor or a medium-speed diesel engine (1,500 - 2,200 rpm) without complex gearboxes or drive arrangements. They also offer a happy compromise, at least for irrigation bore discharges in the range 15-90 l/s, between head per stage and flow per unit pump diameter.

For all pumps running at fixed speeds, the flow rate is essentially a function only of the pump type and overall diameter. Within a type class, only the total head is affected by the number of stages, each stage producing an equal incremental head; a four stage pump therefore has twice the head capacity of a two stage pump. However, especially with mixed flow pumps, detailed impeller design can alter the ratio of axial to radial flow and so provide a considerable range of discharges, each with a different head per stage, for the same pump diameter. Further, impeller design refinements can also squeeze more unit flow from a given pump diameter but this is generally detrimental to the pump efficiency characteristic curve which limits such pumps to a very narrow range of duties.

A considerably different story emerges when pump rotational speed changes are considered. Within fairly wide limits, pump discharge capacity (Q) is directly proportional to speed (N) and head capacity (H) is proportional to speed squared. These proportionalities are generally termed the

'Affinity Laws' and can be stated thus:

$$Q \propto N \quad ; \quad H \propto N^2 \tag{1}$$

Thus a pump running at 1,500 rpm and producing say 20 l/s against a total head of 10 m could, if operated at 2,200 rpm, produce 29.3 l/s against 21.5 m. However, in practice, the characteristics of the source from which the pump is discharging (the bore or well) must also be considered. This aspect will be considered later.

The main components of a mixed-flow borehole pump are shown in outline (Figure 6.1); various terms shown on the figure are discussed below:

Pump Shaft

- the short shaft which runs from the suction to the discharge end of the pump and on which the rotating components are mounted; this shaft is generally thicker than any lineshaft driving it and made of stainless steel to minimise any bearing damage due to corrosion.

Impeller

- the component, generally of bronze or stainless steel that rotates with the pump shaft and imparts some combination of radial and axial velocity to the flow. Impellers may be "closed" as shown with a shroud enclosing the outer blade extremities or "open" in which case the shroud is absent and the bowl below closely profiled to the blade shape. Closed impellers are more efficient and less susceptible to end-float maladjustment, but more susceptible to Sand-pumping damage; they generally have replaceable "wear-rings" set in the surrounding bowl structure to minimise potential back-flow water passages between the bowl and the shroud.

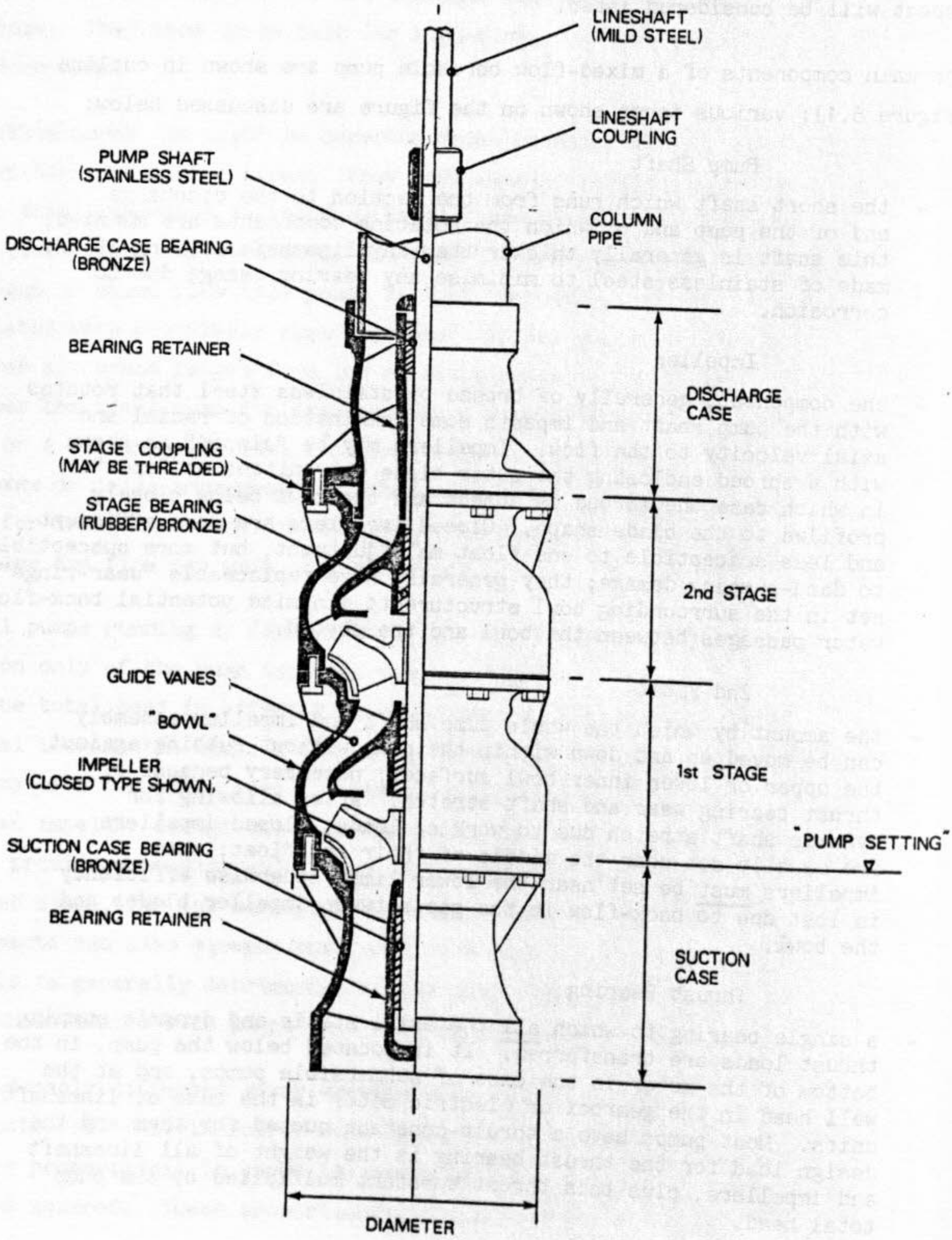
End Float

- the amount by which the whole pump shaft and impeller assembly can be moved up and down within the pump without rubbing against the upper or lower inner bowl surfaces; necessary because of thrust bearing wear and shaft stretch. After allowing for dynamic shaft stretch due to working loads, closed impellers are usually set near the middle of their end-float; open impellers must be set near the lower limit otherwise efficiency is lost due to back-flow in the gap between impeller blades and the bowl.

Thrust Bearing

- a single bearing to which all the axial static and dynamic pumping thrust loads are transferred. It is located below the pump, in the bottom of the motor in the case of submersible pumps, and at the well head in the gearbox or electric motor in the case of lineshaft units. Most pumps have a thrust-constant quoted for them and the design load for the thrust bearing is the weight of all lineshaft and impellers, plus this thrust constant multiplied by the pump total head.

FIGURE 6.1 MIXED FLOW PUMP—MAJOR COMPONENTS



Impeller Lock-Collet

- (not shown) the tapered cylindrical split wedge locating and locking the impeller in place on the pump shaft.

Pump Bowl

- the component, usually cast, in which the impeller and pump shaft is mounted and which contains the waterways and guide vanes used to deliver the flow to the next pump stage in the desired manner; connected by bolting, screwing or clamping. Pump bowls also carry the stage-bearings.

Stage-Bearings

- bearings mounted in each bowl to prevent shaft whip due to impeller imbalance; ususally bronze sleeves in rubber bushes to allow some resilience.

End-Bearings

- long sleeve bearings to impose shaft alignment, mounted in the Suction and discharge cases; generally made of bronze. The bottom bearing is sometimes lubricated before assembly.

Intermediate

- applied to impellers, bowls and bearings above the first pump stage but below the top stage if there are more than two of these.

6.2 Pump Characteristics

A pump characteristic is a graphical plot which primarily summarises the combinations of head and discharge which are possible with a single stage of a borehole pump at a particular speed, although curves for several stages are often drawn on the same graph. Other important information is often shown and this should include the variation of pump efficiency, power consumed and NPSHR with discharge.

NPSHR is the Net Positive Suction Head Requirement of a given pump before inlet cavitation sets in, reducing efficiency and causing erosion damage. It is usually rather confusingly stated as depth of water above absolute vacuum rather than relative to atmospheric so that the maximum theoretical suction of any pump in ideal conditions is in fact 10 m minus NPSH. This is relevant to pump setting since the actual setting should never be more than the suction capacity above the deepest ever expected bore operating water level.

"Pump setting" as defined (Figure 6.1) is usually taken to be the level of the inlet of the first-stage impeller, the others being pressurised by it. In practice there are other deductions to be made from theoretical static suction capacity. These are for any physical hydraulic losses in the pump suction pipe (if fitted) and suction case (He - say 0.5 m minimum) and for the saturation vapour-pressure of the water at the temperature being pumped (Hp - say 0.6 m for Malaysian conditions). If the total suction were allowed to reduce pressures towards this vapour pressure, the water would literally boil, causing premature cavitation. Some safety factor is also considered advisable and some 75 per cent of greatest allowable suction is recommended. For calculating pump settings therefore, the height of pump setting above deepest expected operating water level, or allowable suction (H_s), should never exceed:

$$H_s = 0.75 \left[(10\text{-NPSH}) - H_e - H_p \right] \text{ metres} \qquad (2)$$

Say, for short suction pipes in Malaysian conditions:

$$H_s \quad = \; 6.675 - 0.75 \times \text{NPSH metres} \qquad (3)$$

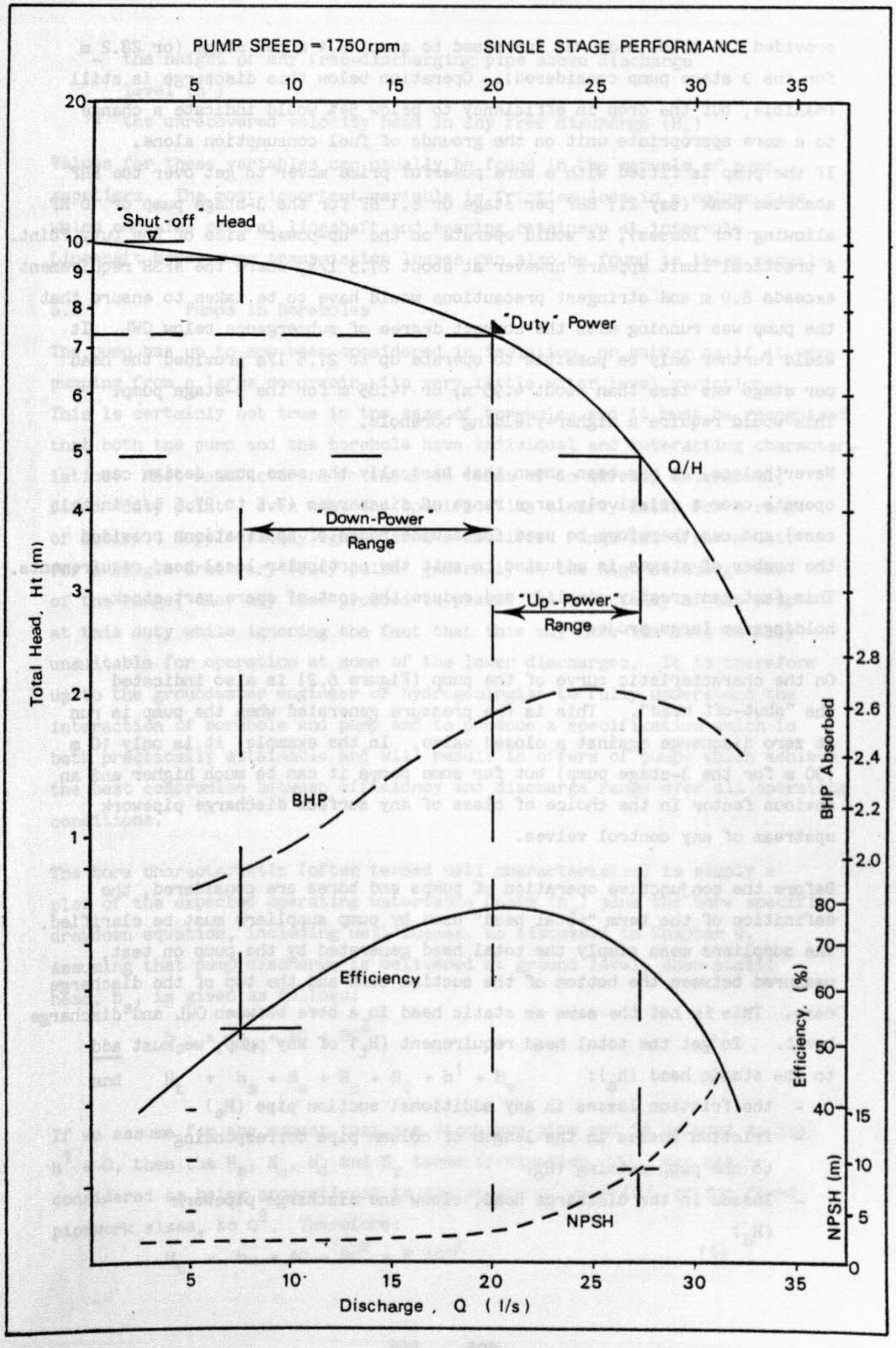
Even if this suction is allowed, a submergence of the end of any suction pipe below OWL is further necessary to prevent air being drawn into the pump in a vortex that may form near the inlet. This submergence is usually at least 1 m. If H_s is negative, the pump itself must be submerged by at least the numerical value of H_s.

A set of characteristics which may be considered typical for a medium-discharge irrigation pump is shown (Figure 6.2). The information contained can tell us a great deal. In general, at any discharge Q, the pump can only operate at the head H_t; this is defined by drawing a vertical line upwards from Q to intersect the Q/H characteristic and then projecting this intersection horizontally to find H_t. At this duty, the pump will have the power consumption, efficiency and NPSH requirement defined by the intersection of these curves and the vertical line from Q. For example, the pump maximum efficiency is about 79% and this occurs at about a discharge of 20 l/s. Therefore the optimum duty for this pump can be read off the Q/H characteristic and is thus 20 l/s at a total head of 7.25 m per stage. A 3-stage pump would therefore be required for a sum total head of 21.75 m and this would absorb about 2.57 BHP per stage so that a prime mover somewhat more powerful than 7.7 HP would be demanded; after allowing for transmission losses and any derating, a 9 HP prime mover would be suitable.

The characertistic of the BHP requirement having a peak value somewhat beyond the efficiency peak (Figure 6.2) is quite common, so that a pump equipped with the above prime mover could never operate at discharges higher than 20 l/s since the power requirement is greater. It could however operate quite happily down to discharges of about 7.5 l/s,

FIGURE 6.2

TYPICAL PUMP CHARACTERISTICS



provided the total head was increased to about 9.4 m per stage (or 28.2 m for the 3 stage pump considered). Operation below this discharge is still feasible, but the drop in efficiency to below 54% would indicate a change to a more appropriate unit on the grounds of fuel consumption alone. If the pump is fitted with a more powerful prime mover to get over the BHP absorbed peak (say 2.7 BHP per stage or 8.1 HP for the 3-stage pump or 10 HP allowing for losses), it would operate on the "up-power" side of the Duty Point. A practical limit appears however at about 27.5 l/s, where the NPSH requirement exceeds 8.9 m and stringent precautions would have to be taken to ensure that the pump was running with the correct degree of submergence below OWL. It would further only be possible to operate up to 27.5 l/s provided the head per stage was less than about 4.95 m, or 14.85 m for the 3-stage pump. This would require a higher-yielding borehole.

Nevertheless, it has been shown that basically the same pump design can operate over a relatively large range of discharges (7.5 to 27.5 l/s in this case) and can therefore be used for a wide range of applications provided the number of stages is adjusted to suit the particular local head requirements. This fact can greatly simplify and reduce the cost of spare part stockholdings on large projects.

On the characteristic curve of the pump (Figure 6.2) is also indicated the "shut-off head". This is the pressure generated when the pump is run at zero discharge against a closed valve. In the example, it is only 10 m (30 m for the 3-stage pump) but for some pumps it can be much higher and an obvious factor in the choice of class of any surface discharge pipework upstream of any control valves.

Before the conjunctive operation of pumps and bores are considered, the definition of the term "total head" used by pump suppliers must be clarified. The suppliers mean simply the total head generated by the pump on test, measured between the bottom of the suction case and the top of the discharge case. This is not the same as static head in a bore between OWL and discharge level. To get the total head requirement (H_t) of any pump, we must add to the static head (h_s):

- the friction losses in any additional suction pipe (H_e)
- friction losses in the length of column pipe corresponding to the pump setting (H_c)
- losses in the discharge head, elbow and discharge pipework (H_d)

- the height of any free-discharging pipe above discharge level (h^1)
- the unrecovered velocity head in any free discharge (H_v)

Values for these variables can usually be found in the manuals of pump suppliers. The most important variable is friction loss in a column pipe which contains central lineshaft and bearing retainers at intervals. Lineshaft horsepower transmission losses can also be found in these manuals.

6.3 Pumps in Boreholes

The pump has up to now been considered in isolation, or rather as if it were pumping from a large reservoir with very little water level variation. This is certainly not true in the case of boreholes and it must be recognised that both the pump and the borehole have individual and interacting characteristics. Most manufacturers do think in terms of an already determined, fixed "duty point", even when the specification clearly calls for a range of duties. Suppliers may ignore such a specified range and offer a unit for a single arbitrary "duty point" generally at the high discharge end of the range; they may then proceed to praise the efficiency of the pump at this duty while ignoring the fact that this may make the unit totally unsuitable for operation at some of the lower discharges. It is therefore up to the groundwater engineer or hydrogeologist to fully understand the interaction of borehole and pump and to produce a specification which is both practically attainable and will result in offers of pumps which achieve the best compromise between efficiency and discharge range over all operating conditions.

The bore characteristic (often termed well characteristic) is simply a plot of the expected operating watertable depth (h_w) plus the bore specific drawdown equation, including well losses, as discussed in Chapter 4. Assuming that pump discharge is delivered at ground level, then static head, h_s, is given as follows:

$$h_s = h_w + AQ + BQ^2 \qquad (4)$$

and

$$H_t = h_s + H_e + H_c + H_d + h^1 + H_v \qquad (5)$$

If we assume for the moment that the discharge pipe end is drowned so that $h^1 = 0$, then the H_e, H_c, H_d and H_v terms in equation (5) can all be considered as being proportional to the square of velocity, or for fixed pipework sizes, to Q^2. Therefore:

$$H_t = h_W + AQ + BQ^2 + F\ (Q)^2 \qquad (6)$$

where $F(Q)^2$ represents all the pipe friction loss terms. Consider a bore which is being pumped against a total head of say 21.75 m with the 3-stage pump (7.25 m per stage and hence delivering 20 l/s; Figure 6.2). If static head is reduced by 4.5 m (1.5 m per stage) due, say, to watertable level change, the new discharge would not be 25 l/s, as might be inferred from the pump characteristic alone. The new discharge can only be calculated by plotting both the pump characteristic and the effective bore characteristic (equation (6)) on the same graph and noting the point of intersection. This process is illustrated on (Figure 6.3(a)); the new discharge in this case would be some 22.9 l/s. The components of equation (5) are illustrated (Figure 6.3(b)). Another way of putting this is to consider that the head change may have generated additional discharge capacity from the pump, but in tending towards this discharge increment, the bore will have generated additional drawdown. The change in net discharge is always less than indicated by the pump characteristic alone; this is true for both upward and downward head changes.

The importance of realising that a pump and borehole combination can only operate at the intersection point of the respective characteristic curves cannot be overemphasised. Not only can the origin of the bore characteristic vary with water level changes, but its slope components can also vary both with age-deterioration in almost all bores and with vertical transmissivity variations is some hard-rock bores.

6.4 Production Pump Selection

6.4.1 General

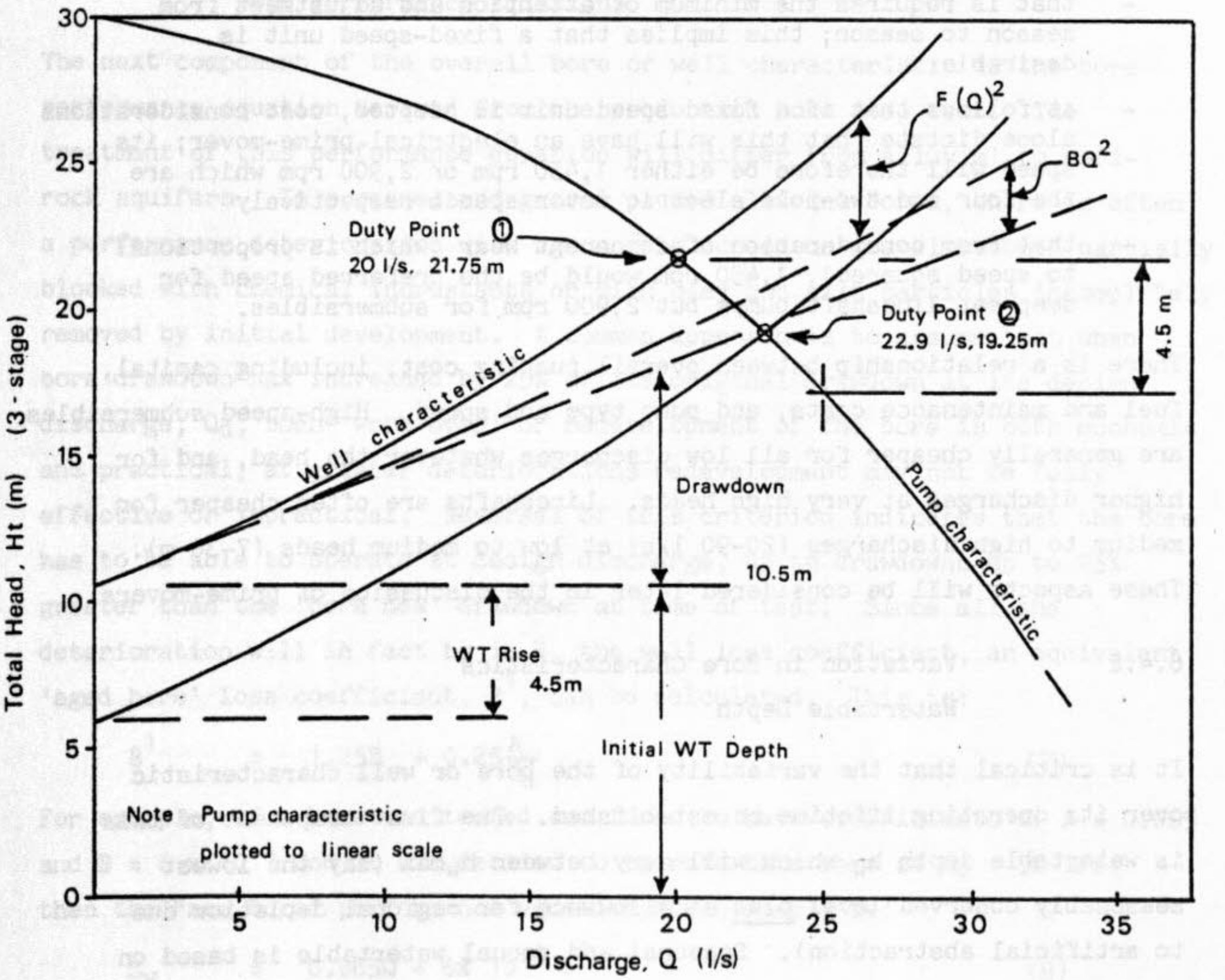
Definition of a suitable production bore pump is often considerably simpler than choosing an appropriate test pump since the range of possible discharges and heads involved in the production duty is invariably much smaller. This is particularly true for high-yielding bores such as those found in the east-coast alluvium. This section of the Manual will therefore address itself mainly to the slightly more difficult problem of production pump selection for hard-rock bores. The principles outlined also apply to high-yield bores but the constraints involved are generally much less severe.

The general requirements of a production pump include:

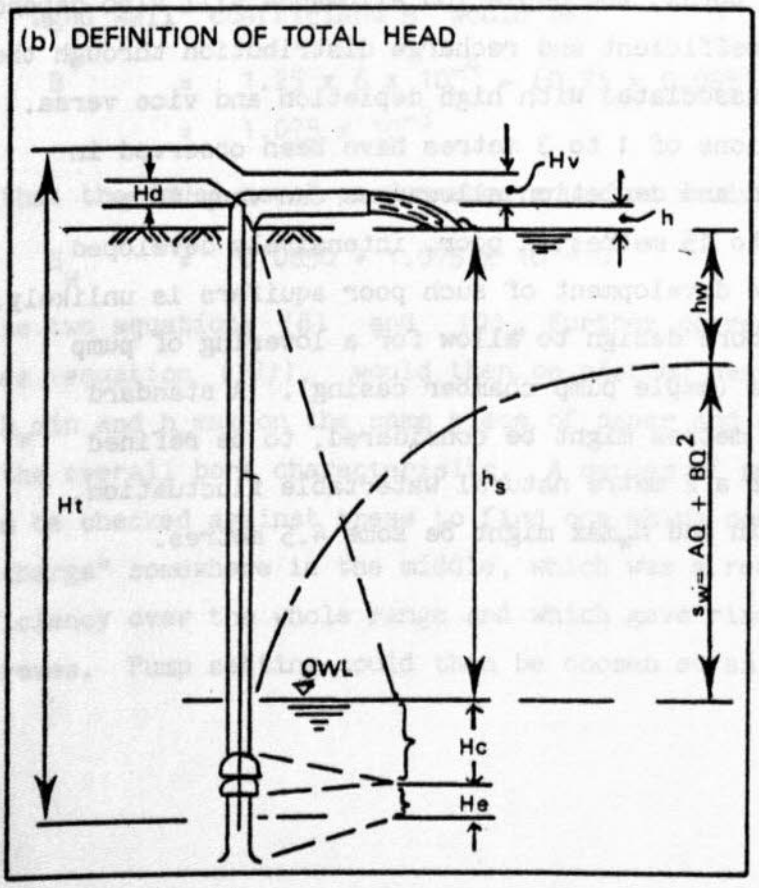
- that it shall be run as near optimum efficiency as possible, consistent with the standardization of units which is desirable in any groundwater pumping project
- that it never runs dry or exceeds its suction limit.

FIGURE 6.3 PUMP AND WELL INTERACTIONS

(a) MUTUAL CHARACTERISTICS



(b) DEFINITION OF TOTAL HEAD



He	Entry or suction pipe friction losses
Hc	Column pipe friction losses
Hd	Discharge fittings friction losses
Hv	Discharge velocity head
h'	Discharge pipe end level above GL (or WL)
h_s	Static head to GL
OWL	Operating Water Level
Ht	Pump total head

- that is requires the minimum of attention and adjustment from season to season; this implies that a fixed-speed unit is desirable
- it follows that if a fixed speed unit is adopted, cost considerations alone dictate that this will have an electrical prime-mover; its speed will therefore be either 1,450 rpm or 2,900 rpm which are the four and two-pole electric motor speeds respectively
- that from consideration of component wear (which is proportional to speed squared), 1,450 rpm would be the preferred speed for deep-set lineshaft pumps but 2,900 rpm for submersibles.

There is a relationship between overall pumping cost, including capital, fuel and maintenance costs, and pump type and speed. High-speed submersibles are generally cheaper for all low discharges whatever the head, and for higher discharges at very high heads. Lineshafts are often cheaper for medium to high discharges (20-90 l/s) at low to medium heads (7-30 m). These aspects will be considered later in the discussion of prime-movers.

6.4.2 Variation in Bore Characteristics

Watertable Depth

It is critical that the variability of the bore or well characteristic over its operating lifetime be established. The first component of this is watertable depth h_w which will vary between $h_w$max (say the lowest seasonably observed level <u>plus</u> an allowance for regional depletion due to artificial abstraction). Seasonal and annual watertable is based on measurement of observation bores; the depletion allowance will also depend on local aquifer storage coefficient and recharge distribution through the year; low storage will be associated with high depletion and vice versa. Typically, seasonal variations of 1 to 3 metres have been observed in hard rock aquifers in Kedah and depletion allowances can vary from 2 metres in good aquifers to 15 metres in poor, intensively developed aquifers although intensive development of such poor aquifers is unlikely. Provision must be made in bore design to allow for a lowering of pump settings at some later date (ample pump chamber casing). A standard depletion allowance of 2.5 metres might be considered, to be refined with experience. Thus, for a 2 metre natural watertable fluctuation, the difference between $h_w$min and $h_w$max might be some 4.5 metres.

Bore Performance Equations

The next component of the overall bore or well characteristic is the bore performance equation derived from the production bore step test. The treatment of this performance equation will differ from alluvial to hard-rock aquifers. In screened and gravel packed alluvial bores, there is often a performance deterioration with age as the screen or gravel becomes partially blocked with chemical incrustants or with clay and silt particles incompletely removed by initial development. A common approach is to assume that when bore drawdown has increased by 25% of its original drawdown at its design discharge, Q_d, then 'work over' or redevelopment of the bore is both economic and practical; at greater deteriorations redevelopment may not be fully effective or impractical. Reversal of this criterion indicates that the bore has to be able to operate at design discharge, up to drawdowns up to 25% greater than the 'bore new' drawdown at time of test. Since all the deterioration will in fact be in B, the well loss coefficient, an equivalent 'aged bore' loss coefficient, B^1, can be calculated. This is:

$$B^1 = 1.25B + 0.25\frac{A}{Q_d} \qquad (7)$$

For example, if a bore was tested and found to have coefficients of $A = 0.065$ and $B = 6 \times 10^{-4}$ and it was decided to operate this bore at $Q_d = 50$ l/s, then the "new bore" performance equation would simply be:

$$S_w = 0.065Q + 6x\ 10^{-4}\ Q^2 \qquad (8)$$

The "aged'well" coefficient B^1 would be:

$$B^1 = 1.25 \times 6 \times 10^{-4} + (0.25 \times 0.065)/50$$

$$= 1.075 \times 10^{-3}$$

so that the "aged bore" performance equation would be:

$$S_w = 0.065Q + 1.075 \times 10^{-3}\ Q^2 \qquad (9)$$

These two equations (8) and (9), further corrected for long pumping times (equation (12)), would then be plotted respectively from origins of $h_w$min and $h_w$max on the same piece of paper and would define the limits of the overall bore characterisitc. A series of pump characteristics could then be checked against these to find one which could produce the "design discharge" somewhere in the middle, which was a reasonable compromise of efficiency over the whole range and which gave rise to no problems at the extremes. Pump setting could then be chosen so as to avoid NPSH problems.

It should be obvious from the foregoing that "design discharge" is a somewhat nebulous concept and that in fact the actual discharge of any bore will vary considerably with watertable depth over the season and with bore age. In hard rock bores, there is unlikely to be much serious performance deterioration with age but the problems of transmissivity decreasing and well losses increasing with drawdown may occur. Examples were given in Chapter 4.

The test shown in Figure 4.16 was carried out with watertable levels half-way between their seasonal maximum and minimum values; the data can be used to re-construct equivalent bore performance equations for watertables at these extremes. This is done by first plotting and then extrapolating the test-derived transmissivity values, and hence A-coefficients (using Logan) against absolute depth to pumping water level. The test-derived and extrapolated values are summarised in Table 6.1.

Table 6.1 Test-Derived and Extrapolated A-Coefficients

Watertable Depth (m.bgl)	Drawdown (m)	O.W.L. (m.bgl)	T (m^2/d)	A (m/l/s)
5.6	0.89	6.49	650	0.162
5.6	2.12	7.72	600	0.176
5.6	4.15	9.75	475	0.222
5.6	8.22	13.82	325	0.324
Extrapolated values for		4.60	700	0.151
Interpolated values for		6.60	646	0.163
Extrapolated values for		15.50	293	0.360
Extrapolated values for		17.00	275	0.383

Note: OWL Operating Water Level
bgl below ground level

The equivalent B-coefficients can similarly be extrapolated and interpolated; the data shown in Table 6.2 is plotted and manipulated in conjunction with the previously derived values of A.

Table 6.2 Test-Derived and Extrapolated B-Coefficients

O.W.L. (m.bgl)	A (m/l/s)	Q (l/s)	Drawdown (m)	B (m.s^2/l^2)
6.49	.162	6	0.89	*
7.72	.176	12	2.12	5.5×10^{-5}
9.75	.222	18	4.15	4.75×10^{-4}
13.82	.324	24	8.22	7.71×10^{-4}
4.6	.151	*	*	0
6.6	.163	*	*	5.0×10^{-6}
15.5	.360	*	*	8.08×10^{-4}
17.0	.383	*	*	8.35×10^{-4}

Note: * no sensible values.

From the data in Tables 6.1 and 6.2, the bore performance curves of the actual bore when operating between the two static watertable extremities, can be constructed. These however would refer to the step-test pumping times of about 180 minutes and would require further correction for pumping times extended to long times.

Long-term pumping times in an irrigation season, between major recharge events of up to six or seven months are easily envisaged, but because the pump is rarely operating for 24 hours a day, pumping is of a cyclical on-off nature and the rapid recovery of water levels during the "off" cycle greatly offsets the long term effects. By means of the principle of superimposition, an approximation for the effects of such cyclical pumping can be made. It can be stated that a bore operating for 60 percent of the time has the same long-term effects on the aquifer as an equivalent bore operating all the time, but at 60 per cent of the real bore discharge. In fact, 60 per cent is a reasonable operating factor for an irrigation bore over a whole season and so we may estimate the long-term additional drawdown on this assumption.

The theoretical basis is the Cooper-Jacob semi-equilibrium approximation method (Figure 4.14). This gives the rate of single log-cycle time drawdown, Δs, as follows:

$$\Delta s = \frac{2.303Q}{4\pi T}$$

or, for the cyclical pumping case:

$$\Delta s = \frac{2.303 \times 0.6 \times Q}{4\pi T}$$

$$\text{Or} \quad \Delta s = \frac{0.11Q}{T} \qquad (10)$$

Therefore, for t = 6 months (262,800 min); the time log-cycles elapsed since the 180 minutes test basis are log (262,800 ÷ 180) or 3.164. The additional long-term well drawdown to be allowed for is therefore:

$$ds = \frac{3.164 \times 0.11Q}{T}$$

$$= \frac{0.348Q}{T} \qquad (11)$$

or, with Q expressed in l/s units and in terms of A:

$$ds = \frac{0.348AQ}{1.22}$$

$$ds = 0.285AQ \qquad (12)$$

We can thus describe two extreme-condition bore characteristics, an early-season, early-time characteristic:

$$h_s = h_w\text{min} + 1.0AQ + BQ^2 \qquad (13)$$

and, a late-season, late-time characteristic:

$$h_s = h_w\text{max} + 1.285\ AQ + BQ^2 \qquad (14)$$

A and B are themselves variable with the gross value of h_s or operating water level (OWL) and so the method of solution has to be by back-reference to tabulations equivalent to Tables 6.1 and 6.2 whilst Q is derived in terms of h_s by using the binomial method of mathematical solution. For the case in question, these solutions are given in Tables 6.3 and 6.4, using the binomial solutions respectively as follows:

$$Q = \frac{A^2 + 4Bs_w \quad - A}{2B} \qquad (15)$$

$$\text{and} \quad Q = \frac{1.65A^2 + 4Bs_w \quad - 1.285A}{2B} \qquad (16)$$

Table 6.3 Calculation of Early-Season, Early-Time Bore hole Performance (Equation (15))

OWL, h_s (m.bgl)	A (m/l/s)	B ($m.s^2/l^2$)	Drawdown S_w (m)	Q (l/s)
4.6	0.151	0.0	0.0	0.0
6.60	0.163	5.0×10^{-6}	1.89	11.59
7.72	0.176	5.5×10^{-5}	3.12	17.63
9.75	0.222	4.75×10^{-4}	5.15	22.15
13.82	0.324	7.71×10^{-4}	9.22	26.75
15.50	0.360	8.08×10^{-4}	10.90	28.46
17.00	0.383	8.35×10^{-4}	12.40	30.36

Note, in Table 6.3, the "plunging" drawdown for very little extra yield once a critical operating water level has been passed, even though a significant lower-hole Transmissivity still remains (about 250 m^2/d). This phenomenon is often observed on actual bore tests.

Table 6.4 Calculation of Late-Season, Late-Time Bore hole Performance (Equation (16))

OWL, h_s (m.bgl)	A (m/l/s)	B ($m.s^2/l^2$)	Drawdown S_w (m)	Q (l/s)
9.1	0.206	2.9×10^{-4}	0.0	0.0
9.75	0.222	4.75×10^{-4}	0.65	2.27
13.82	0.324	7.71×10^{-4}	4.72	11.11
15.50	0.360	8.08×10^{-4}	6.40	13.52
17.00	0.383	8.35×10^{-4}	7.90	15.64

It can be seen that while the bore on test appeared capable of more than 25 l/s, the predicted performance from a depleted watertable and after allowing for long-term pumping effects is only about half of the tested discharge. Clearly, the actual bore discharge will vary considerably throughout the season and hence the contradiction in using the term "design discharge".

6.4.3 Pump Selection

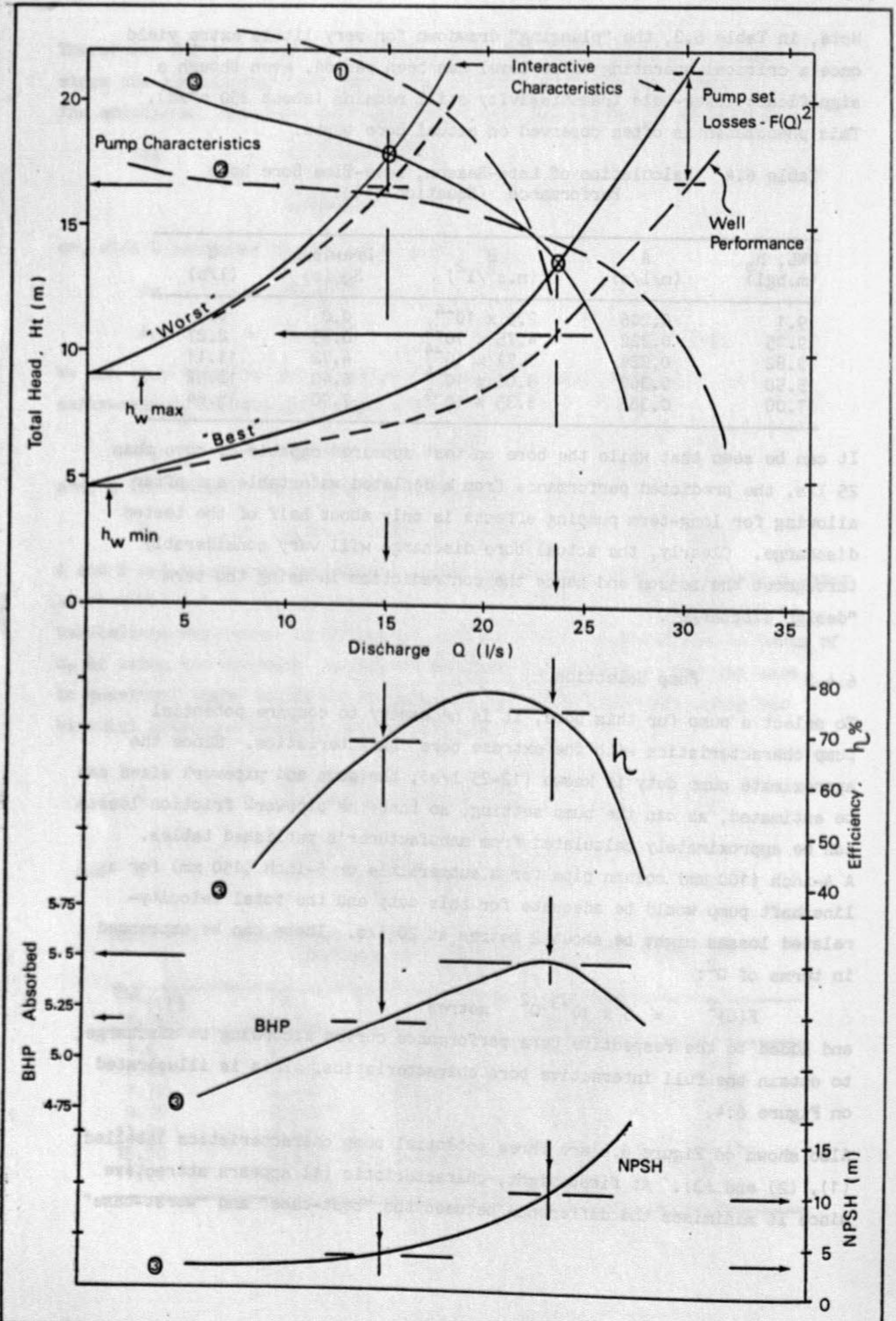
To select a pump for this bore, it is necessary to compare potential pump characteristics with the extreme bore characteristics. Since the approximate pump duty is known (12-25 l/s), the pump and pipework sizes can be estimated, as can the pump setting, so that the pipework friction losses can be approximately calculated from manufacturer's published tables. A 4-inch (100 mm) column pipe for a submersible or 6-inch (150 mm) for a lineshaft pump would be adequate for this duty and the total velocity-related losses might be about 2 metres at 20 l/s. These can be expressed in terms of Q^2:

$$F(Q)^2 = 5 \times 10^{-3} Q^2 \text{ metres} \qquad (17)$$

and added to the respective bore performance curves according to discharge, to obtain the full interactive bore characteristics. This is illustrated on Figure 6.4.

Also shown on Figure 6.4 are three potential pump characteristics labelled (1), (2) and (3). At first sight, characteristic (1) appears attractive since it minimises the difference between the "best-case" and "worst-case"

FIGURE 6.4 PRODUCTION PUMP SELECTION



interactive discharges - 23 and 17 l/s respectively - allowing the bore to be designated at a nominal 20 l/s for irrigation purposes. However, characteristic (1) is almost certainly unsuitable because the high-discharge inersection of characteristics is very near the end of the published pump curve. The following consequences are most likely:

- the efficiency at the high discharge will be extremely low, possibly only 40%, with consequent high fuel costs
- these will probably be NPSH problems which require detailed checking
- there may be a "superflow" problem which is manifest as cavitation in the guide vanes, due to flow separation in them caused by excessive velocities through the pump.

Characteristic (2) certainly would avoid the NPSH and "Superflow" problems, but it too is undesirable for several reasons, including:

- the wide range of discharges (24 to 13.5 l/s) between "best case" and "worst case" interactions
- the best efficiency is almost certainly at the high-discharge end making it unacceptably low at the "worst-case" end.

Characteristic (3) would be considered to be a fair compromise between these extremes and its interactive discharges would be 23.5 and 15 l/s respectively. Thus any irrigation system based on this bore would have to be designed so that its end-season peak water requirements could be met by pumping 15 l/s for say 22 hours per day. Early-season water requirements would then be met by pumping a discharge up to 23.5 l/s for much shorter times. From the bore performance curves below the interactive characteristics, the respective operating water levels in the bore can be read off; these will be about 10.7 and 16.6 m below ground in the "best" and "worst" cases. These give a reference level from which to calculate pump setting.

The efficiency, power consumption and NPSH curves for characteristic (3) are also shown in the lower half of Figure 6.4. It can be seen that the NPSH requirement at 23.5 l/s is 10 m and hence the allowable suction head, $H_s = 6.675 - 0.75 \times 10$ or $- 0.825$ m (Equation (3)). The pump must therefore run at least 0.825 m submerged and the setting must be at least 10.7 + 0.825 or 11.725 m below ground. At 15 l/s, the NPSH requirement is 3.4 m and so $H_s = 6.675 - 0.75 \times 3.4$ or $+ 4.125$ m. From a pump setting of 11.725 m, this would accommodate an OWL down to 15.85 m only.

Since the actual OWL is expected to be 16.6 m below ground, a lineshaft pump setting must be increased to 16.6 - 4.125 or 12.475 m. An actual setting with some factor of safety of at least 2.0 m would be politic and the recommended pump setting would be 14.5 m. Pump chamber length must be sufficient to accommodate this plus 3.1 m of suction pipe to put the pump inlet about 1 m below 16.6 m.Since electrical submersible pumps cannot, because of their construction, run with any suction lift, such pumps would have to have settings of at least 17.6 m thus requiring slightly deeper pump chamber casings.

The peak power absorbtion of the pump is at the highest discharge and is some 5.5BHP. A 7.5HP prime mover would be appropriate, allowing for temperature de-rating and transmission losses.

One last observation concerning pump component and bore casing diameters should be made. Bores are almost never straight or vertical in spite of the most careful drilling and construction methods. Long pump casings of 20 m or more can easily be curved quite considerably as they are inserted in a bore even if constructed in steel. The importance therefore of adequate diametric clearance between pumps and casings cannot be over-emphasised; this factor is often totally and willfully ignored in pump manufacturer's literature. Manufacturer's claims of what is possible on these grounds must be treated with some scepticism. An annular (all-round) clearance between pump maximum OD and casing ID of 1 inch (50 mm) is recommended for lineshaft pumps and 0.75 inch (38 mm) for submersibles. The column pipe OD over couplings should be at least 0.75 inch (38 mm) less than the pump maximum diameter. Even these clearances are often inadequate to insert essential measuring equipment such as electric contact. sondes and air lines into the annulus.

6.5 Test Pump Selection

The problem of test pump selection is rather different. We are not concerned with seasonal, long-term or age-induced differences in bore performance, but simply with the very large discharge range necessary to carry out an adequate step test over the range of heads met with in a whole variety of boreholes. On the other hand, considerations of pump efficiency are also important from the viewpoint of accuracy. With test pumps, it is therefore necessary to employ pump characteristic-modification techniques which are inappropriate to production bore pumps.

These techniques are those of discharge throttling by means of a valve in the discharge line and of pump speed variation. Extreme conditions have even called for a by-pass technique whereby some of the discharge is diverted back down the bore-casing annulus to reduce the net discharge below a pump's theoretical lower capability.

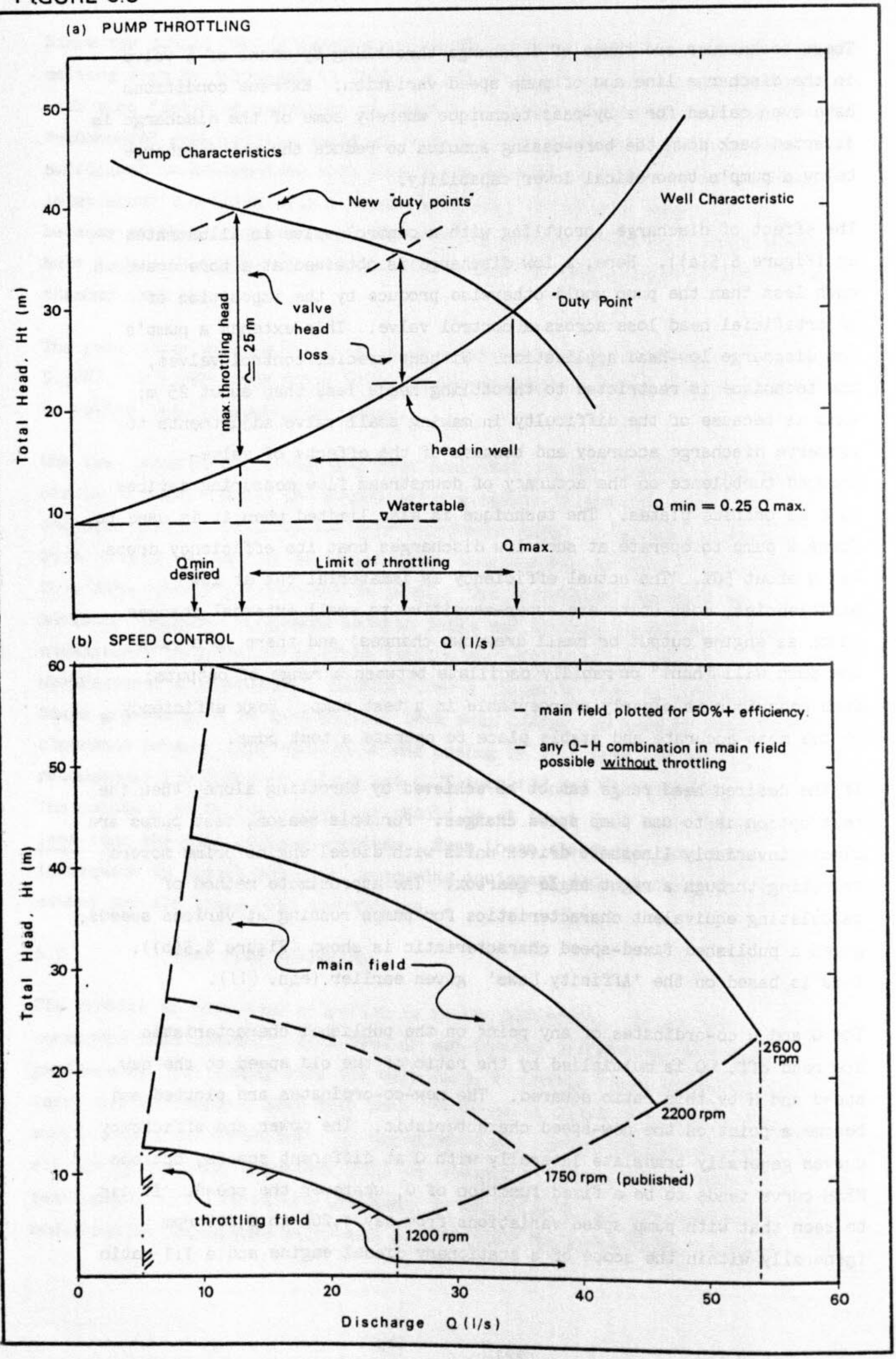
The effect of discharge throttling with a control valve is illustrated in (Figure 6.5(a)). Here, a low discharge is obtained at a bore drawdown much less than the pump would otherwise produce by the imposition of an artificial head loss across a control valve. This extends a pump's low discharge low-head application. Without special control valves, the technique is restricted to throttling heads less than about 25 m; this is because of the difficulty in making small valve adjustments to preserve discharge accuracy and because of the effects of valve-induced turbulence on the accuracy of downstream flow measuring devices such as orifice plates. The technique is also limited when it is used to force a pump to operate at such low discharges that its efficiency drops below about 50%. The actual efficiency is immaterial but at low efficiencies, most units are super-sensitive to small external changes (such as engine output or small drawdown changes) and there is a risk the pump will "hunt" or rapidly oscillate between a range of outputs; such behaviour is clearly unacceptable in a test pump. Peak efficiency is the most accurate and stable place to operate a test pump.

If the desired head range cannot be achieved by throtting alone, then the next option is to use pump speed changes. For this reason, test pumps are almost invariably lineshaft driven units with diesel engine prime movers operating through a right angle gearbox. The approximate method of calculating equivalent characteristics for pumps running at various speeds, given a published fixed-speed characteristic is shown (Figure 6.5(b)). This is based on the 'Affinity Laws' given earlier (eqn. (1)).

The Q and H co-ordinates of any point on the published characteristic are read off. Q is multiplied by the ratio of the old speed to the new speed and H by this ratio squared. The new-co-ordinates are plotted and become a point on the new-speed characteristic. The power and efficiency curves generally translate laterally with Q at different speeds, but the NPSH curve tends to be a fixed function of Q, whatever the speed. It can be seen that with pump speed variations from say 1,200 to 2,600 rpm (generally within the scope of a stationary diesel engine and a 1:1 ratio

FIGURE 6.5

TEST PUMP DISCHARGE CONTROL

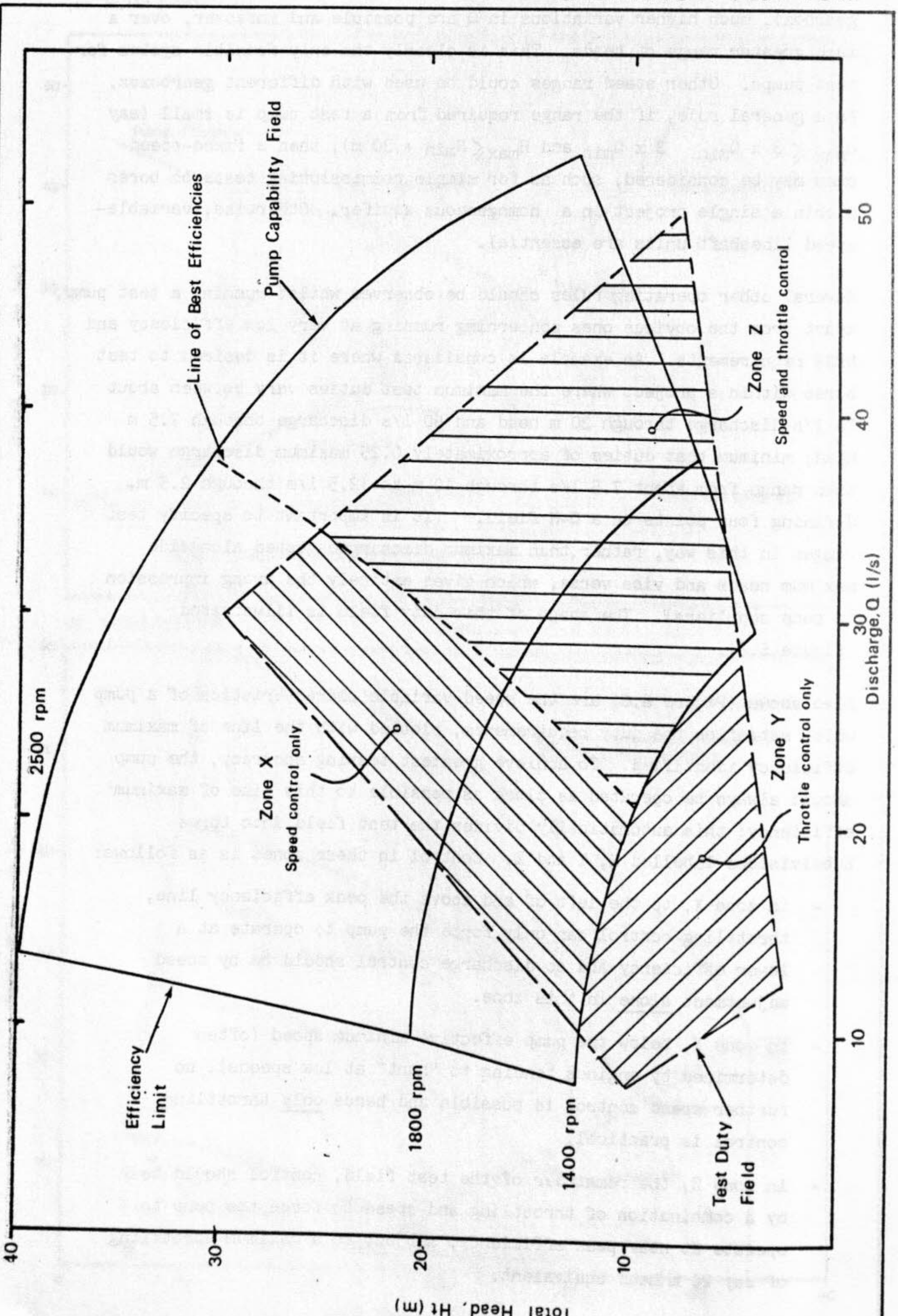


gearbox), much higher variations in Q are possible and moreover, over a much greater range of heads. This is clearly the only feasible system for test pumps. Other speed ranges could be used with different gearboxes. As a general rule, if the range required from a test pump is small (say $Q_{max} \leqslant 3 \times Q_{min}$ $3 \times Q_{min}$ and $H_{max} \leqslant H_{min} + 30$ m), then a fixed-speed pump may be considered, such as for simple commissioning tests on bores within a single project in a homogeneous aquifer. Otherwise, variable-speed lineshaft units are essential.

Several other operating rules should be observed whilst running a test pump, apart from the obvious ones concerning running at very low efficiency and NPSH requirements. An example is considered where it is desired to test bores within a project where the maximum test duties vary between about 30 l/s discharge through 30 m head and 50 l/s discharge through 7.5 m head; minimum test duties of approximately 0.25 maximum discharge would then range from about 7.5 l/s through 10 m to 12.5 l/s through 2.5 m, defining four points on a Q-H field. (It is important to specify test ranges in this way, rather than maximum discharges quoted alongside maximum heads and vice versa, which gives entirely the wrong impression to pump suppliers). The shape of this duty field is illustrated (Figure 6.6).

Also shown (Figure 6.6) are the speed-variable characteristics of a pump which satisfies the duty requirements, plotted with the line of maximum efficiency identified. To achieve greatest testing accuracy, the pump should always be operated as close as possible to this line of maximum efficiency; this automatically divides the test field into three subdivisions labelled X, Y and Z. Control in these zones is as follows:

- in zone X, to the left of and above the peak efficiency line, throttling control can only force the pump to operate at a lower efficiency and so discharge control should be by speed adjustment alone in this zone.
- in zone Y, below the pump effective minimum speed (often determined by engines tending to "hunt" at low speeds), no further speed control is possible and hence only throttling control is practical.
- in zone Z, the remainder of the test field, control should be by a combination of throttling and speed to force the pump to operate at near peak efficiency, subject to a maximum throttling of say 20 m head equivalent.


Pump Capability Field
Line of Best Efficiencies
2500 rpm
1800 rpm
1400 rpm
Efficiency Limit
Test Duty Field
Zone X
Speed control only
Zone Y
Throttle control only
Zone Z
Speed and throttle control
Total Head, Ht (m)
40
30
20
10
Discharge, Q (l/s)
10
20
30
40
50

Restrictions on the overall diameters of test pumps and components are more severe than for production pumps since it is absolutely essential to have access for electric water-level dipper tapes at all times. If bore casing is of 10 inch diameter, then the maximum suitable column pipe nominal size is 6 inches. Even this would only leave a 1¼ inch annulus in which to freely operate an electric sonde of diameter about 3/4 inch; any casing deviation or bad weld can ruin or jam an electric sonde.

6.6 Prime Movers and Ancillaries

This Chapter will conclude with a brief discussion of prime movers and ancillaries; the discussion is not intended to be exhaustive since the subject more correctly comes under the heading of mechanical and electrical engineering. Specific points pertinent only to boreholes will be discussed.

Pumps fitted with electrical submersible motors are ususally of the radial-flow type, running at 2-pole speeds of about 2,900 rpm, although some mixed flow units are available. Up to about 2 HP, single-phase motors are available but beyond this, 3-phase supplies are required. Electric supply voltage must be good and sophisticated starters are required since the motors are particularly sensitive to voltage fluctuations; a fluctuation of no more than 5% from rated voltage is usually allowed. The high running speeds are necessary to reduce motor diameters sufficiently to be comparable to pump sizes, but this has the drawback of effectively limiting their irrigation bore applicability to discharges below about 25 l/s. Pumps of greater capacity are available but the high speed means that head-per stage increments are at least 20 to 30 metres at higher discharges, making them very difficult to match bore duties which may only range from 7 to 25 m. The motor is fitted below the pump, with the water inlet between them. A non-return rising-disc valve is normally fitted just above the pump, to prevent backward running on shut down caused by return flow of water in the rising main. Backward running can cause the motor to act as a generator producing voltages high enough to damage windings and control gear. The non-return valve also gives some protection against silt settling out of the discharge line on shut down; radial-flow pumps are particularly sensitive to this problem because of their highly-convoluted water passages which can prevent debris flowing through and out of the unit. Rising main for submersibles is usually standard waterworks quality steel pipe, but with tapered threads.

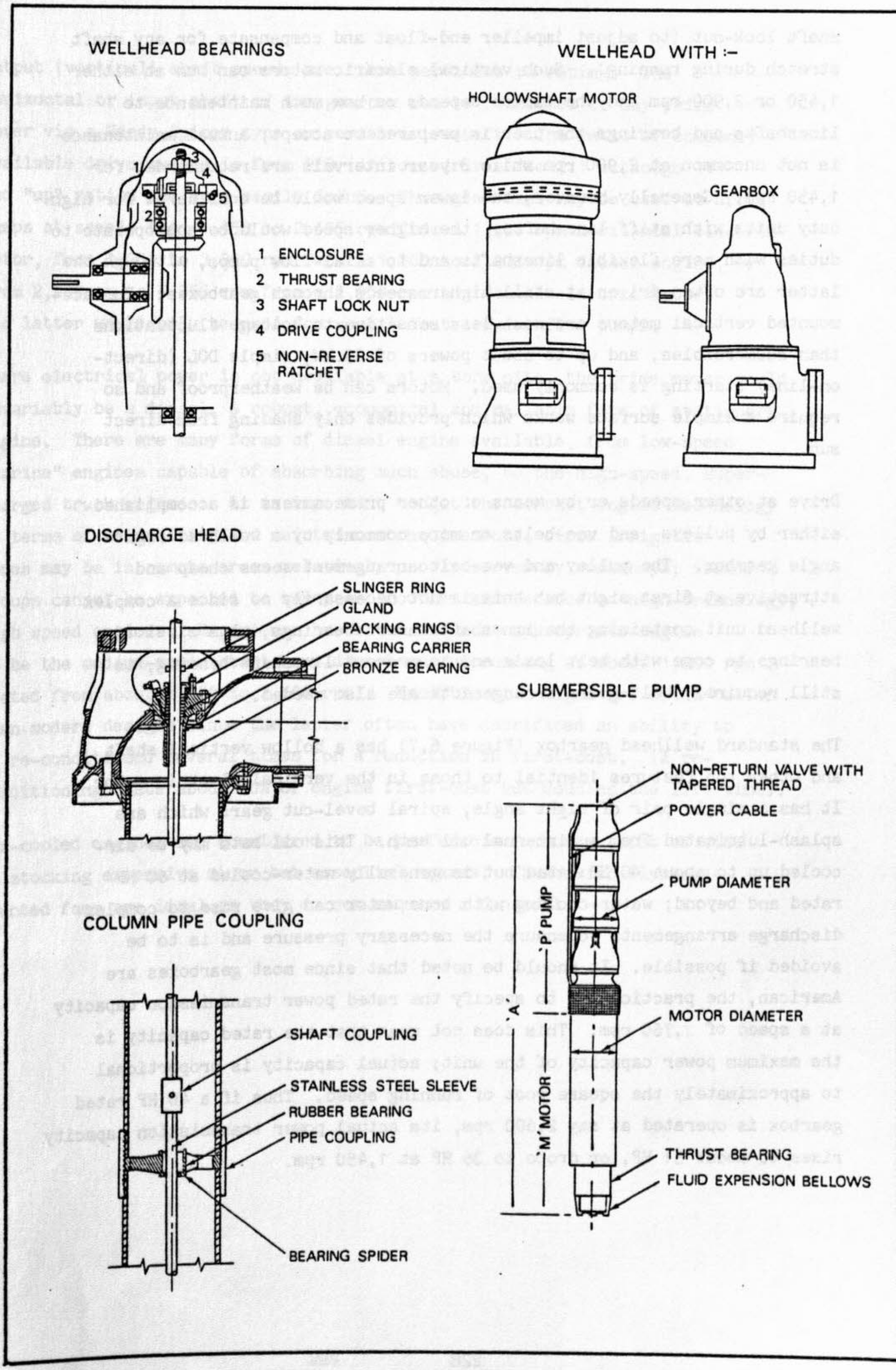
These are necessary to force the threads almost to a "yield" in order to "lock" the pipe joint against torques induced when starting or stopping the motor; without locking, these torques can uncouple the pipes. Surface works for submersibles can be simple and often all that is required is a weatherproof cabinet for the starter.

Lineshaft driven mixed-flow pumps are normally used for higher discharge irrigation bores because of their head-per-stage properties. In recent years, slimmer axial-flow pumps have also been used more frequently. Rising main (or more correctly, column pipe) for lineshafts has to be heavy-gauge steel pipe and is furnished with parallel form threads in order that the male sections fo the coupling can fully engage and locate between them at each joint, the lineshaft bearing carrier or "spider" which is usually made of bronze (Figure 6.7). Since this spider is responsible for the perfect alignment of the lineshaft bearings (usually synthetic "cutless" rubber), the column pipe ends have to be machined absolutely square and never simply cut, in order to achieve the spider alignment. Bearing spacings determine pipe unit lenghts.

The coupling collar is screwed on to one end of each column pipe initially and this end expanded or treated with a thread locking compound so that it never comes undone under normal field-tool torques. This is to ensure the column pipe always "breaks" naturally on the same side of the bearing spider, the lineshaft coupling side. Otherwise it would not be possible to insert tools to uncouple the lineshaft during pump withdrawal. Lineshafts themselves are usually of mild steel and are product-water lubricated although enclosed oil-lubricated versions are available. Lineshafts are of the same length as column pipe units, with square-machined ends and parallel threaded "muff" couplings; they are usually sleeved with 316 stainless steel sleeves where they pass through bearings. Whatever the drive system, the lineshaft passes through the bore head via a cast or fabricated discharge head with a special short stainless steel top shaft, a shaft stuffing box and a shaft bearing. The thrust bearing carrying all the axial shaft loads and pumping thrusts is always located in the driver or gearbox sitting on top of the discharge head.

If drive is electric, it is almost invariably by means of a vertical, hollowshaft electric motor; the drive top shaft passes through the motor hollowshaft to a thrust bearing above the motor and to a unit containing a non-reverse ratchet (to prevent backward running) and an adjustable

FIGURE 6.7 PUMPSET COMPONENTS



shaft lock-nut (to adjust impeller end-float and compensate for any shaft stretch during running). Such vertical electric motors can run at either 1,450 or 2,900 rpm and the choice depends on how much maintenance to lineshafts and bearings the user is prepared to accept; annual maintenance is not uncommon at 2,900 rpm while 3 year intervals are recommended for 1,450 rpm. Generally however, the lower speed would be considered for high duty units with stiff lineshafts, the higher speed would be appropriate to duties with more flexible lineshafts and to axial-flow pumps, although the latter are often driven at still higher speeds through gearboxes. Surface-mounted vertical motors are much less sensitive to voltage fluctuations than submersibles, and up to about powers of 25 kW, simple DOL (direct-on-line) starting is commonly used. Motors can be weatherproof and so require a simple surface works which provides only shading from direct sun.

Drive at other speeds or by means of other prime movers is accomplished either by pulleys and vee-belts or more commonly by a wellhead right-angle gearbox. The pulley and vee-belt arrangement seems cheap and attractive at first sight but this is not necessarily so since a complex wellhead unit containing the lineshaft thrust bearings, plus sideload bearings to cope with belt loads and to prevent lineshaft bending, is still required; belt guard arrangements are also needed.

The standard wellhead gearbox (Figure 6.7) has a hollow vertical shaft and other top features idential to those in the vertical electric motor. It has a single pair of right angle, spiral bevel-cut gears which are splash-lubricated from an internal oil bath. This oil bath may be air-cooled up to about 40 HP rated but is generally water-cooled at 60 HP rated and beyond; water-cooling with bore water can give rise to complex discharge arrangements to ensure the necessary pressure and is to be avoided if possible. It should be noted that since most gearboxes are American, the practice is to specify the rated power transmission capacity at a speed of 1,760 rpm. This does not mean that the rated capacity is the maximum power capacity of the unit; actual capacity is proportional to approximately the square root of running speed. Thus if a 40 HP rated gearbox is operated at say 2,600 rpm, its actual power transmission capacity rises to about 51 HP, or drops to 36 HP at 1,450 rpm.

Output (vertical) shaft speeds are these pertinent to rating. The horizontal or input shaft of the gearbox can be connected to any prime mover via a Hardy-Spicer type universal driveshaft. The ratio of commonly available drive speeds is from 2:3 (driver: driven) to 2:1 although 1:1 and "up" ratios are more easily found. Thus gearboxes may be used to drive pumps at speeds from 1,450 to 2,175 rpm from a 1,450 rpm horizontal electric motor, from 1,200 to 3,300 rpm from a 1,200 to 2,200 rpm diesel engine, or from 2,900rpm to 4,350 rpm from a 2,900 rpm horizontal electric motor; the latter would only be used for sophisticated axial-flow pumps.

Where electrical power is not available at a bore site, the prime mover would invariably be a diesel, a robust, economical and reliable form of stationary engine. There are many forms of diesel engine available, from low-speed "marine" engines capable of absorbing much abuse, to the high-speed, supercharged truck engines. It is important to get the correct engine technology in terms of reliability and maintenance requirements, since irrigation bores may be in remote areas relatively poorly served by workshops; farmer groups cannot be expected to carry out routine maintenance on high-technology, high speed engines. Experience has shown the most appropriate engine to be the medium-speed stationary diesel engine running at about 1,800 rpm (rated from about 1,300 to 2,600 rpm). Traditional designs are often better than modern designs since the latter often have sacrificed an ability to be re-conditioned several times for a reduction in first-cost. (A reconditioning costs about 40% of engine first-cost but doubles the life time).

Air-cooled engines are considered to be preferable from the point of view of stocking expensive minor but essential spares. Those with flywheel-mounted fans are ideal as they do not require a fan-belt.

7. GROUNDWATER CHEMISTRY

7.1 Introduction

A survey of groundwater chemistry by the hydrogeologist normally has three main objectives:

- the determination of the groundwater flow system, including the distribution of the recharge and discharge areas
- the study of groundwater quality in relation to its intended use for irrigation, drinking or industrial use.
- the recognition of any potential corrosion or incrustation problems with tubewell components and pumps.

Before considering the above topics, it will be instructive to those geologists with a limited chemical background to include a brief description of some basic chemical concepts and definitions.

7.2 General

The chemical parameters most important in groundwater chemistry and which should therefore normally be analysed for, are: major ion concentrations, total dissolved solids, pH, alkalinity, hardness, dissolved gases and redox potential. The temperature of the groundwater should also be recorded. Other physical characteristics of water, such as taste, odour and turbidity, are useful to note.

The concentrations of dissolved salts in a water sample are usually expressed in milligrams per litre (mg/l) or their numerical equivalent, parts per million (ppm). The dissolved salts in water exist as dissociated ions: positively charged cations and negatively charged anions, which react with one another in definite weight ratios, that is, according to their equivalent weights. One equivalent weight of cation will exactly combine with one equivalent weight of anion. Therefore, the concentrations of cations and anions in a water sample should balance when expressed in equivalent weight units, commonly milliequivalents per litre (meq/l) or equivalents per million (epm); these are obtained by dividing the concentration of the ions in mg/l by its equivalent weight. If the cation and anion concentrations are not equal, this indicates that either one or more ions present in the water have not been analyssed for, or the chemical analysis is incorrect.

The major ions in groundwater are shown (Table 7.1). During test pumping of boreholes by the JPT, the calcium, bicarbonate, carbonate, chloride and sulphate concentrations can be determined in the field using a Hach portable chemical analysis kit; sulphate by a colorimetric technique (spectrophotometer) and the others by titration. Sodium and potassium are measured in a laboratory by the atomic adsorbtion method.

Table 7-1

Major Ion Constituents of Groundwater

	Chemical symbol	Equivalent weight
Cations:		
Calcium	Ca	20
Magnesium	Mg	12.2
Sodium	Na	23
Potassium	K	39.1
Anions:		
Carbonate	CO_3	30
Bicarbonate	HCO_3	61
Sulphate	SO_4	48
Chloride	C1	35.5

Ions, which are usually present in small concentrations but are often important groundwater quality criteria, are: iron, silica, boron, nitrate, lithium, aluminium, strontium and iodide.

The concentration of total disssolved solids (TDS), which is determined in a laboratory by evaporation, can be measured indirectly in the field using an electrical conductivity (EC) meter. TDS is approximately proportional to EC. The relationship varies depending on the composition of the water, but an approximate expression applicable for most water is:

$$EC \times 10^6 \text{ (in micromhos/cm) at } 25^\circ C \times 0.64 = TDS \text{ in mg/l}$$

Electrical conductivity is temperature dependent and is therefore usually recorded at the standard temperature of 25°C.

The pH of a solution is a measure of the hydrogen ion concentration and hence the acidity or alkalinity. The scale ranges from 0 to 14 with 7 as neutral, below 7 acid and above 7 alkali. The pH of groundwater is unstable and must be measured at the bore head; the JPT pump test team routinely measures pH of the pump test discharge water. The pH recorded in a laboratory analysis often differs significantly from the real pH of the groundwater.

Alkalinity is the capacity of a solution to neutralise acid. In most groundwaters, alkalinity is due to the presence of bicarbonate and, to a lesser extent, carbonate. It is expressed as equivalent titratable amounts of calcium carbonate, that is:

$$HCO_3\ (mg/l) \times \frac{(CaCO_3)}{(HCO_3)} + CO_3\ (mg/l) \times \frac{(CaCO_3)}{(CO_3)} = \text{total alkalinity as mg/l } CaCO_3$$

where the ratios are in equivalent weights.
Total hardness (TH) is a measure of the calcium and magnesium content of a water. It is analysed for by titration and the result is again expressed as mg/l $CaCO_3$. The amount of magnesium in a sample can be found by subtracting the calcium concentration as ($CaCO_3$) from the total hardness. The total hardness may comprise both carbonate and non-carbonate hardness. The latter occurs where the metals are associated with anions, such as chloride, sulphate and nitrate, and may be found from the expression.

TH - Alakalinity = Non-carbonate hardness.

The dissolved gases, oxygen (O_2), carbon dioxide (CO_2) and hydrogen sulphide (H_2S), are unstable and must be measured at the bore head. The O_2 and CO_2 are determined by titration; H_2S is detectable in small concentrations by its 'rotten egg' smell and can be measured by a colorimetric method in the JPT Hach Kit.

The redox potential, Eh, is an indication of the oxidising or reducing tendencies of a water. The potential is measured in millivolts between a metal (normally platinum) electrode and a standard reference electrode using an electronic meter. It is important to prevent the sample from coming into contact with air during measurement. Therefore naturally flowing boreholes are most suitable for testing. The sample should be tapped from the discharge outlet from a point before aeration is possible and passed through a slightly pressurized sampling bottle containing the measuring electrodes. The normal range for redox potential in groundwater is between + 500 mV and - 100 mV. A positive reading is generally indicative of an oxidising system and a negative reading of a reducing system. At present, the JPT groundwater section has no equipment with which to measure Eh; such measurements would be worthwhile if corrosion studies were contemplated.

Table 7.2 is an example of chemical analyses for boreholes in the Kedah-Perlis area.

7.3 Hydrochemical Diagrams

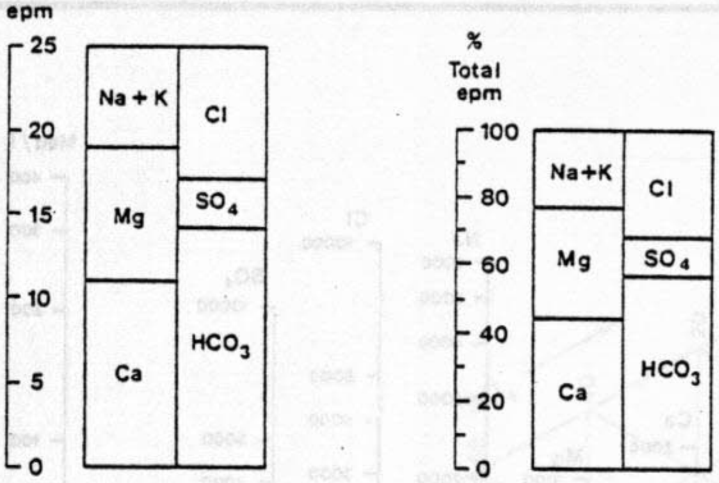
Nuermous methods have been developed to represent water quality data graphically. Some of the more commonly used methods are shown (Figures 7.1 to 7.4). Of the 'pattern' type (Figures 7.1 and 7.2), the Stiff diagram is probably the best as, with this technique, different water

Table 7.2

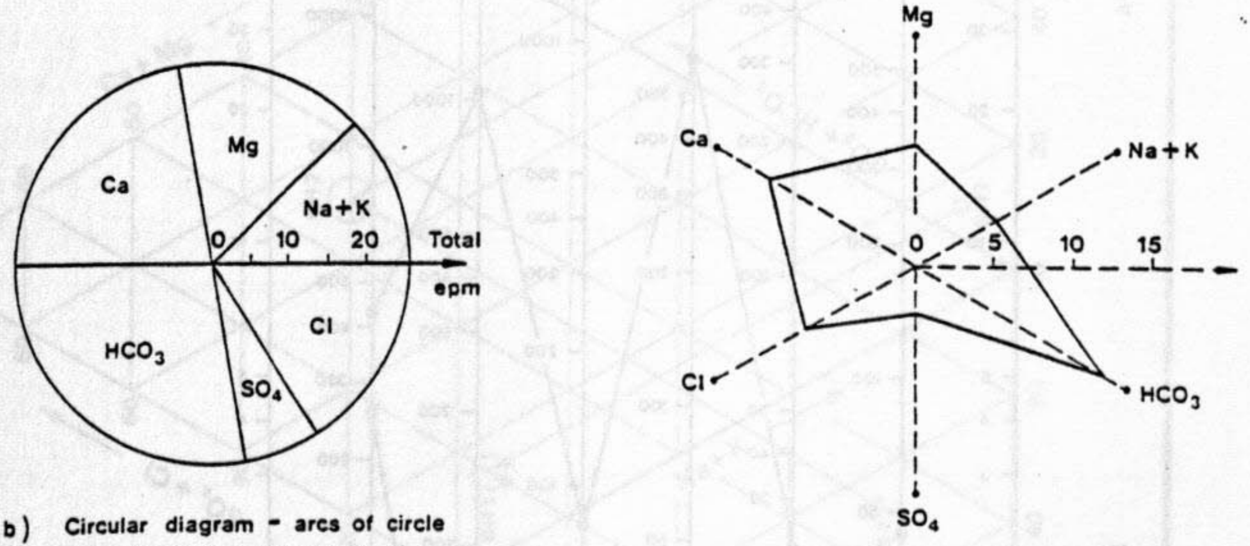
Chemical Analysis of Water Samples from JKR Boreholes, Kedah and Perlis

		Wellhead Analyses					Laboratory Analyses							
Well No.	pH	Carbon Dioxide (mg/l)	Alkalinity (as $CaCO_3$) (mg/l)	Hardness (as $CaCO_3$) (mg/l)	Iron (as Fe) (mg/l)	Manganese (as Mn) (mg/l)	E.C. (umhos)	T.D.S. (mg/l)	Calcium (meq/l)	Magnesium (meq/l)	Sodium + Potassium (meq/l)	Bicarbonate (meq/l)	Sulphate (meq/l)	Chloride (meq/l)
NORTH KEDAH														
GS 736	6.3	68	210	160	0.5	0.5	320	210	2.740	0.484	0.240	3.164	0.083	0.141
737	6.5	16	120	110	0.7	0	270	170	1.960	0.484	0.361	2.541	0.146	0.085
738	6.2	60	140	95	11.0	0	120	135	1.200	0.598	0.268	1.803	0.125	0.197
739	6.3	96	180	170	1.5	0.8	300	215	2.320	0.303	0.642	3.164	0.021	0.085
740	5.6	84	50	50	3.4	0.5	65	50	0.300	0.164	0.300	0.525	0.104	0.113
741	6.5	12	180	200	1.6	0.6	350	265	3.000	0.795	0.837	3.803	0.042	0.225
742	5.7	44	10	30	0.1	0	30	25	0.080	0.082	0.139	0.100	0.104	0.113
743	6.0	12	50	30	10.0	0	50	40	0.100	0.303	0.211	0.295	0.083	0.282
744	6.1	16	60	70	25.0	0	70	55	0.200	0.303	0.158	0.393	0.082	0.338
745	5.9	32	50	30	8.0	0.8	60	45	0.200	0.303	0.190	0.295	0.208	0.141
746	6.1	8	40	100	15.0	0.6	55	45	0.200	0.197	0.170	0.295	0.188	0.141
747	5.9	38	60	50	21.0	1.0	95	65	0.500	0.303	0.194	0.733	0.146	0.141
748	6.7	12	60	70	9.0	0.5	160	100	1.00	0.361	0.145	1.016	0.313	0.225
749	6.2	24	100	110	2.6	0.5	170	120	1.550	0.164	0.117	1.917	0.104	0.141
750	6.4	40	110	110	0.1	4.0	315	190	2.100	0.836	0.365	3.197	0.063	0.056
753	6.3	52	130	90	20.0	0.8	240	165	0.740	1.000	0.843	2.344	0.042	0.197
754	6.2	136	75	120	5.5	0.8	245	160	1.200	0.697	0.373	0.984	0.917	0.253
757	6.5	104	280	250	0.1	0.5	750	575	4.440	1.279	1.873	5.000	0.063	2.028
PERLIS														
GS 761	7.3	80	320	280	0.5(?)	0.3(?)	1.120	645	2.280	4.598	3.575	5.705	0.271	4.479
762	6.7	96	260	310	0.1	0	1.080	605	3.420	3.377	3.151	5.639	0.292	4.085
763	7.1	96	270	335	0.1	0	1.100	615	4.120	2.639	3.505	5.672	0.313	4.310
765	7.1	40	260	250	0.1	0	1.100	665	3.600	2.598	3.573	6.197	0.448	4.338
768	7.0	60	310	310	0.1	0	550	320	5.150	1.033	0.171	6.040	0.021	0.225
769	7.0	40	350	350	0.1	0	700	317	3.040	2.656	0.349	5.639	0.029	0.225
771	7.0	60	295	290	0.1	0	500	300	2.840	2.943	0.265	6.040	0	0.197
778	6.8	68	320	320	0.4	0	680	341	1.320	4.721	0.240	5.672	0.025	0.310
779	6.8	88	280	270	0.6	0	700	362	5.020	1.139	0.358	6.393	0.25	0.366

FIGURE 7.1 REPRESENTATION OF ANALYSIS DATA

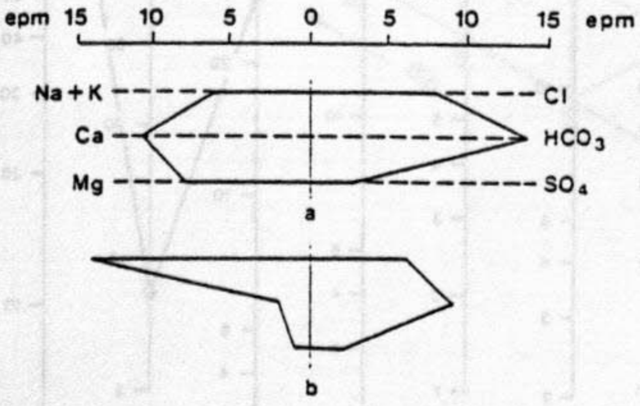


a) Bar graph method, total equivalents per million (epm) or percentage total equivalents



b) Circular diagram – arcs of circle proportional to percentages of each ion, radius proportional to total equivalents per million

c) Radial coordinates – distance from origin proportional to total equivalents per million



d) Stiff diagram – distinctive shapes facilitate comparison of analyses

FIGURE 7.2 LOGARITHMIC NOMOGRAPH DISPLAY

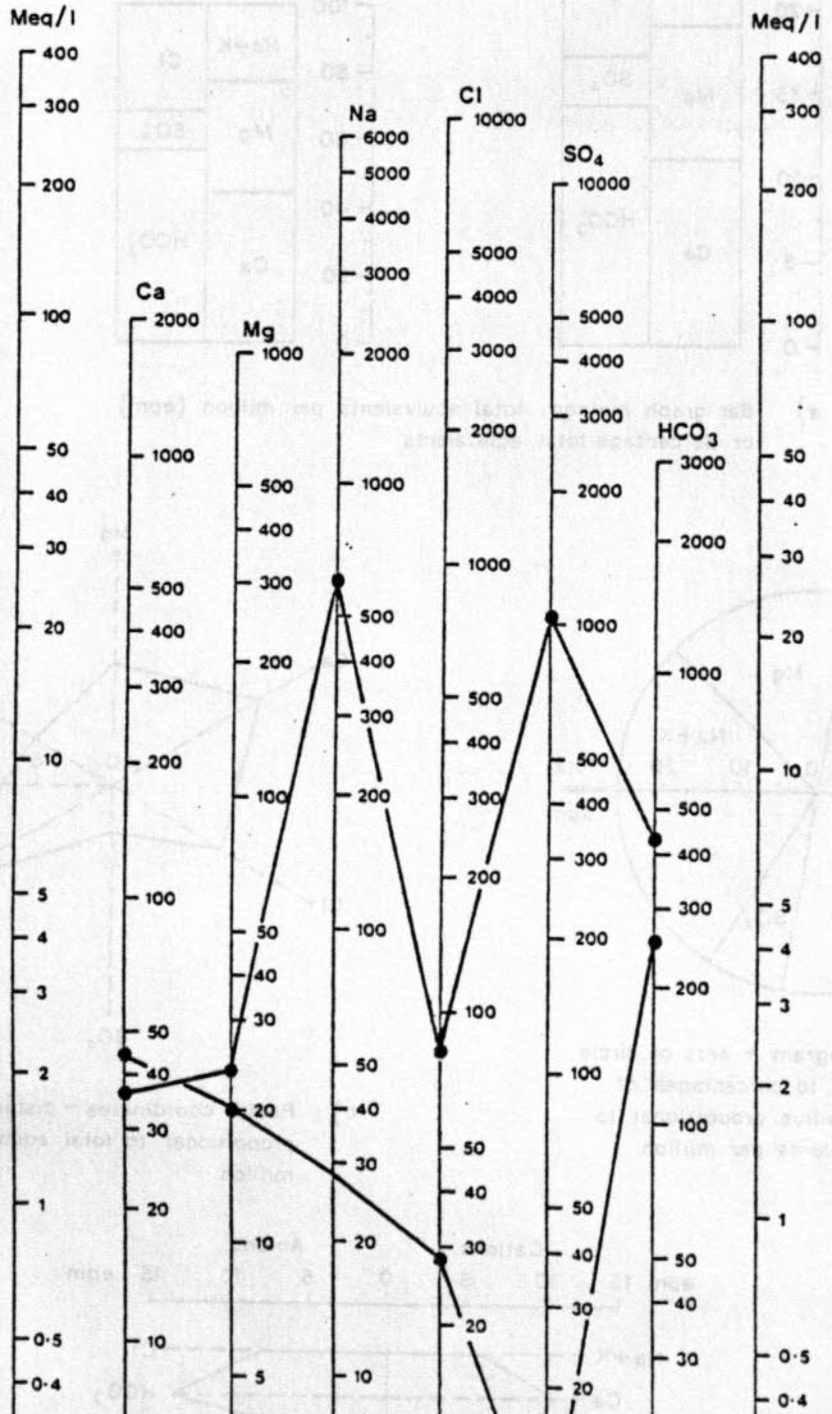
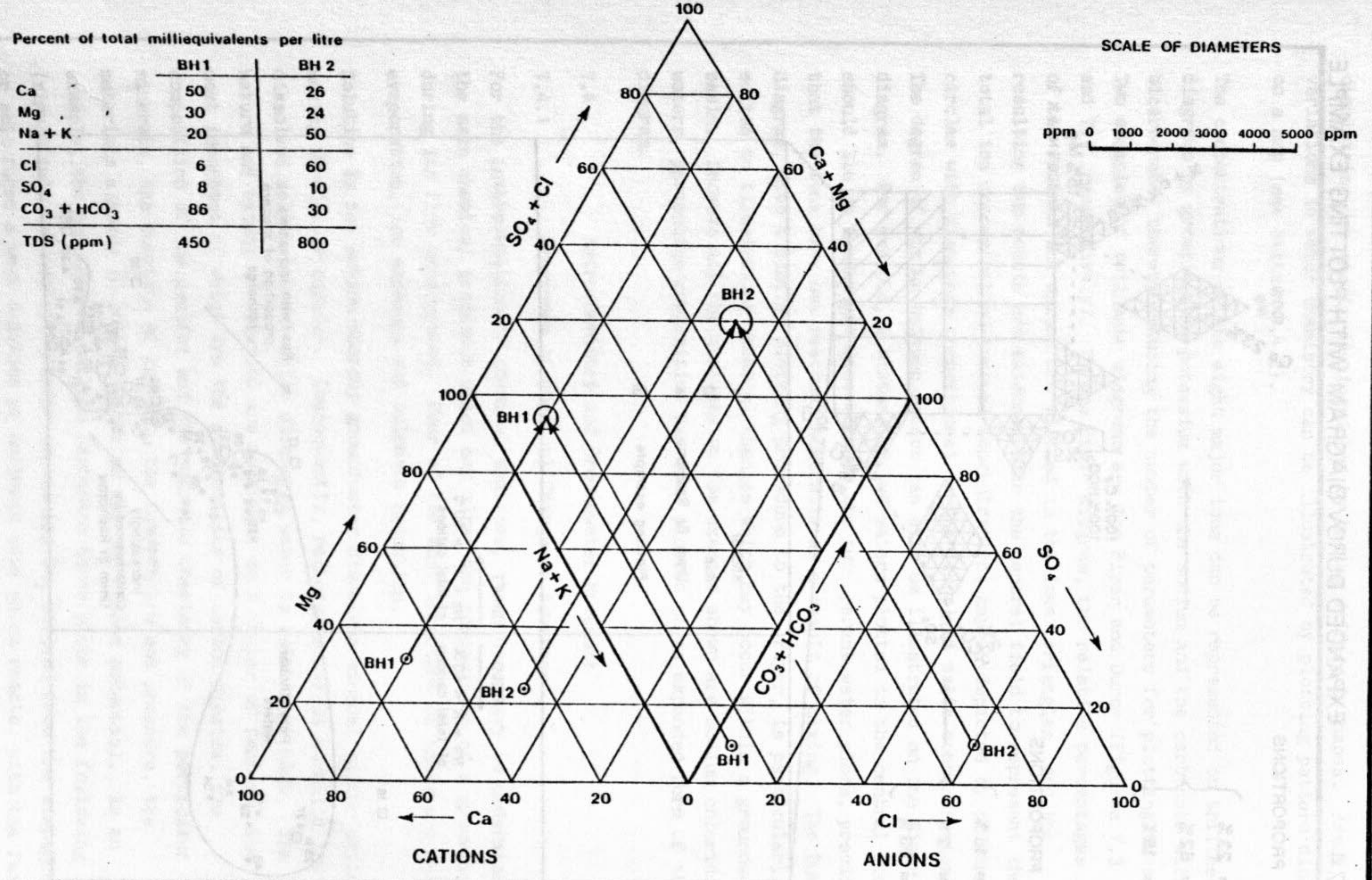


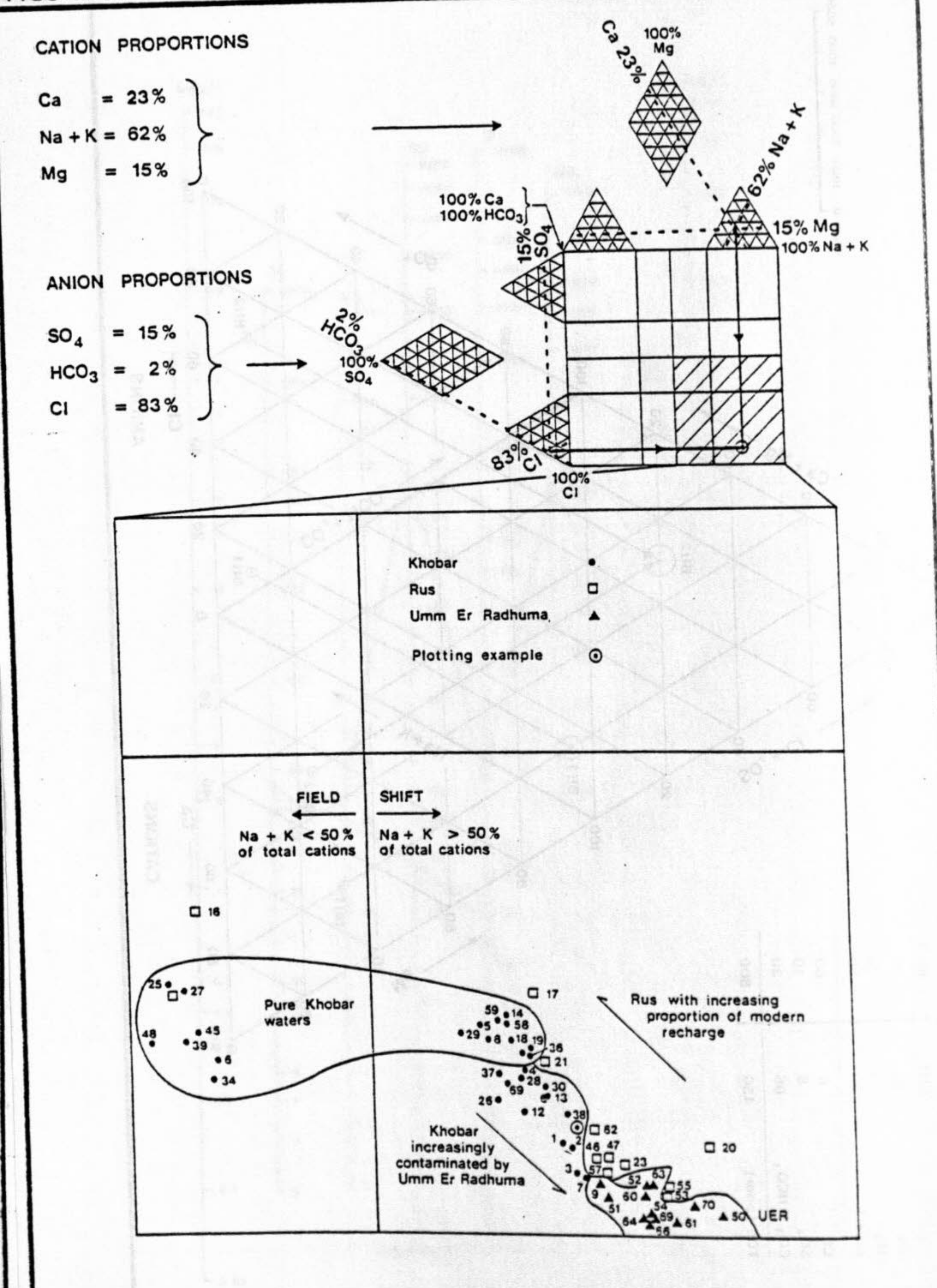
FIGURE 7.3 PIPER DIAGRAM WITH PLOTTING EXAMPLES



Percent of total milliequivalents per litre

	BH1	BH 2
Ca	50	26
Mg	30	24
Na + K	20	50
Cl	6	60
SO_4	8	10
$CO_3 + HCO_3$	86	30
TDS (ppm)	450	800

FIGURE 7.4 EXPANDED DUROV DIAGRAM WITH PLOTTING EXAMPLE



chemistries are easily distinguished by having different shapes. Areal variations in water chemistry can be illustrated by plotting pattern diagrams on a map (see Section 7.4.2).

The concentrations of the eight major ions can be represented on trilinear diagrams by grouping the potassium with the sodium and the carbonate with bicarbonate, thereby reducing the number of parameters for plotting to six. Two examples of trilinear diagrams are the Piper and Durov (Figures 7.3 and 7.4, respectively). On the Piper diagram, the relative percentages of the cations and anions are plotted in the lower triangles, and the resulting two points are extended into the central field to represent the total ion concentration (actual concentrations may be depicted by drawing circles with diameters proportional to the dissolved salts concentration). The degree of mixing between waters can also be illustrated on the Piper diagram. For example, a mixture of two waters plotted in the central field should lie on a straight line between the two parent water plots, provided that the ions have not reacted chemically as a result of mixing. The Durov diagram, with a similar plotting procedure to the Piper, is particularly suited to illustrating chemical changes which may occur within a groundwater basin. Because all the analyses on the example shown are sodium chloride waters, percentage composition has been plotted on an expanded form of the diagram.

7.4 Hydrochemistry and Groundwater Movement

7.4.1 Processes of Chemical Change in Aquifers

For the interpretation of chemical analyses, it is necessary to understand the main chemical processes which may alter the composition of a groundwater during its flow underground. Four of the most important of these are solution, evaporation, ion exchange and sulphate reduction.

Solution is the action whereby groundwater dissolves mineral matter whilst moving through an aquifer. Consequently, recharge water is normally low in dissolved solids content while discharge water is relatively high. The nature and extent of solution are dependent on a number of factors, the most important of which are the availability of carbon dioxide, the composition of the aquifer and the solution chemistry of the particular minerals, the duration of contact, the temperature and pressure, the materials already in solution, the pH and the redox potential. As an example, the solution of calcium carbonate takes place in the following (simplified) manner: groundwater containing CO_2 derived from the atmosphere or soil forms a weak solution of carbonic acid which reacts with the fairly insoluble $CaCO_3$ to produce soluble Ca $(HCO_3)_2$. Magnesium carbonate is added to water by a similar reaction. The solution of calcium (and magnesium) carbonate results in an increase in the pH, alkalinity and hardness of the groundwater. In sedimentary rocks, the most common mineral sources of calcium are calcite, aragonite, dolomite (CaMg $(CO_3)_2$), anhydrite and gypsum. The sources of some of the other important ions in sedimentary aquifer waters are as follows:

- sodium from decomposition of plagioclase feldspars and from clay minerals
- sulphate from gypsum, anhydrite and pyrite minerals
- chloride from halite minerals, from original sea water or from concetration by evaporation.

Evaporation, like solution, can also cause an increase in the dissolved salts concentration of a groundwater; it takes place mainly when the water is in the soil zone.

Ion exchange is the interchange of an ion in solution with another ion on a surface-active material. Normally in groundwaters ion exchange involves the exchange of sodium ions held on clay minerals for calcium and magnesium ions in solution. The capacity for a matrix or a mineral to allow exchange of cations is called the cation exchange capacity (CEC), which is expressed in terms of milliequivalents per 100 g of material. Montmorillonite has by far the highest CEC value (70 to 120) of the clay minerals. Ion exchange causes the softening of the water and may result in all the calcium and magnesium being exchanged.

The reduction of sulphate in groundwater is caused by anaerobic bacteria and it results in the production of hydrogen sulphide gas. It is associated with an absence of dissolved oxygen (used up in oxidation reactions and consumed by bacteria) and a corresponding low Eh value.

The effects of chemical actions such as those discussed above lead to the following generalisations. Young recharge water is usually dominated by calcium and magnesium bicarbonate and throughflow water is typically of a sodium becarbonate type (see Figure 8.2). Parts of groundwater basins where there is limited flow, such as in fine grained formations or near flow-barriers like impermeable faults, are characterised by high chloride waters, mainly because chloride is very soluble and remains in solution through most of the processes which remove other ions, like precipitation, ion exchange and sulphate reduction.

7.4.2 Illustration of Chemical Changes

Variations in groundwater chemistry through an aquifer can be illustrated in the following ways:

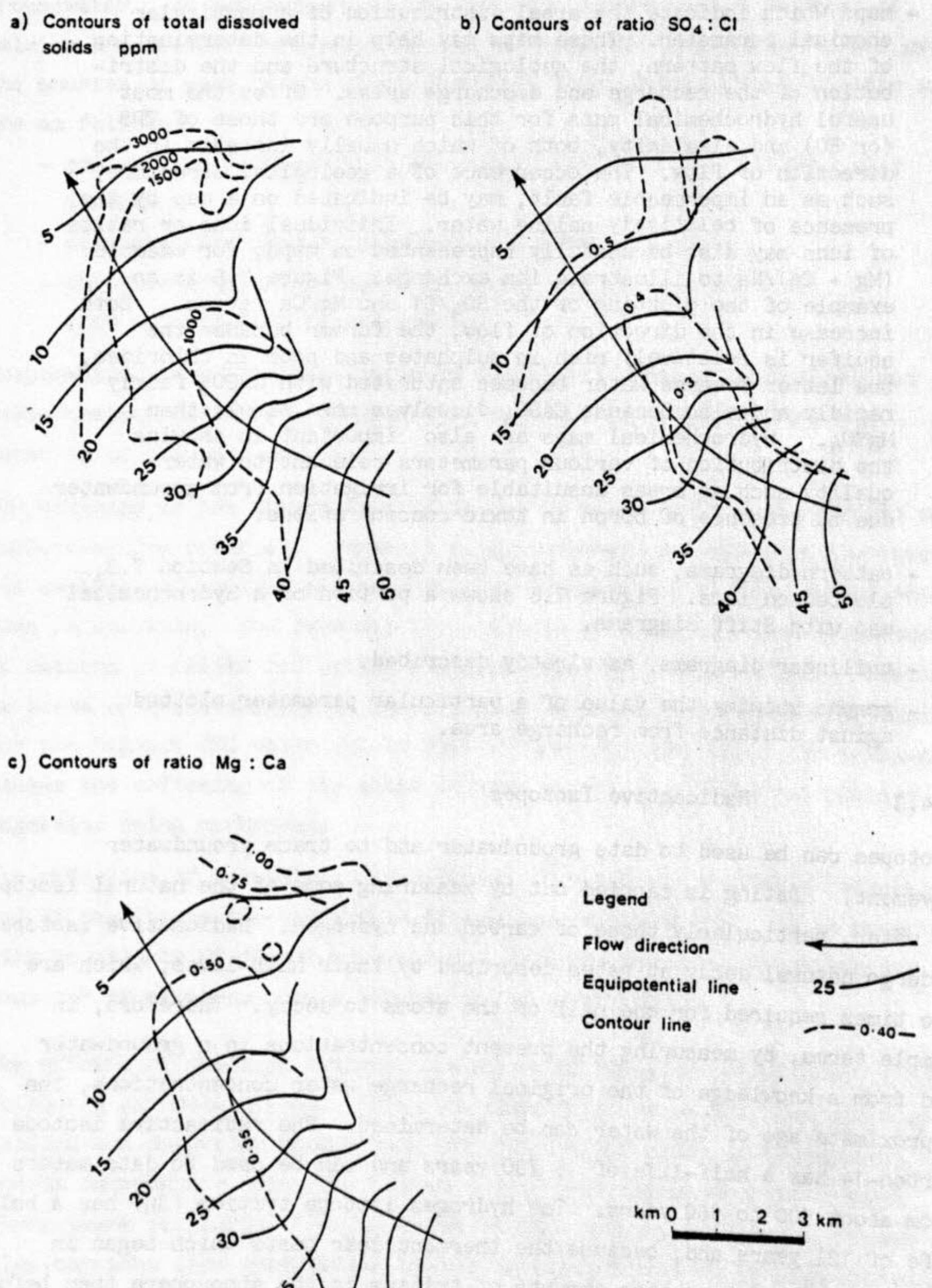
- maps which indicate the areal distribution of a particular chemical parameter. These maps may help in the determination of the flow pattern, the geological structure and the distribution of the recharge and discharge areas. Often the most useful hydrochemical maps for this purpose are those of TDS (or EC) and alkalinity, both of which usually increase in the direction of flow. The occurrence of a geological structure, such as an impermeable fault, may be indicated on a map by the presence of relatively saline water. Individual ions or ratios of ions may also be usefully represented on maps, for example (Mg + Ca)/Na to illustrate ion exchange. Figure 7.5 is an example of the plotting of the SO_4/C1 and Mg/Ca ratios. Both increase in the direction of flow, the former because the aquifer is relatively rich in sulphates and poor in chlorides, the latter because water becomes saturated with $CaCO_3$ fairly rapidly and also because $CaSO_4$ dissolves more slowly than $MgSO_4$. Hydrochemical maps are also important in showing the distribution of various parameters relevant to water quality such as areas unsuitable for irrigation from groundwater due to presence of boron in toxic concentrations.

- pattern diagrams, such as have been described in Section 7.3, plotted on maps. Figure 7.6 shows a portion of a hydrochemical map with Stiff diagrams.
- trilinear diagrams, as already described.
- graphs showing the value of a particular parameter plotted aginst distance from recharge area.

7.4.3 Radioactive Isotopes

Isotopes can be used to date groundwater and to trace groundwater movement. Dating is carried out by measuring some of the natural isotopes in water, particularly those of carbon and hydrogen. Radioactive isotopes undergo natural decay at rates described by their half-lives, which are the times required for one half of the atoms to decay. Therefore, in simple terms, by measuring the present concentrations in a groundwater and from a knowledge of the original recharge water concentrations, the approximate age of the water can be determined. The radioactive isotope carbon-14 has a half-life of 5 730 years and can be used to date waters from about 500 to 900 years. The hydrogen isotope tritium (3H) has a half-life of 12¼ years and, because the thermonuclear tests which began in 1952, produced far greater amounts of tritium in the atmosphere then before, this isotope is particularly suitable as an indicator of whether recharge occurred before or after this date. Natural isotope age determinations can be used to indicate the direction of groundwater flow. Artificial tracers may also be introduced for this purpose, for example, tritirum or 82Br.

FIGURE 7.5 EXAMPLES OF HYDROCHEMICAL MAPS



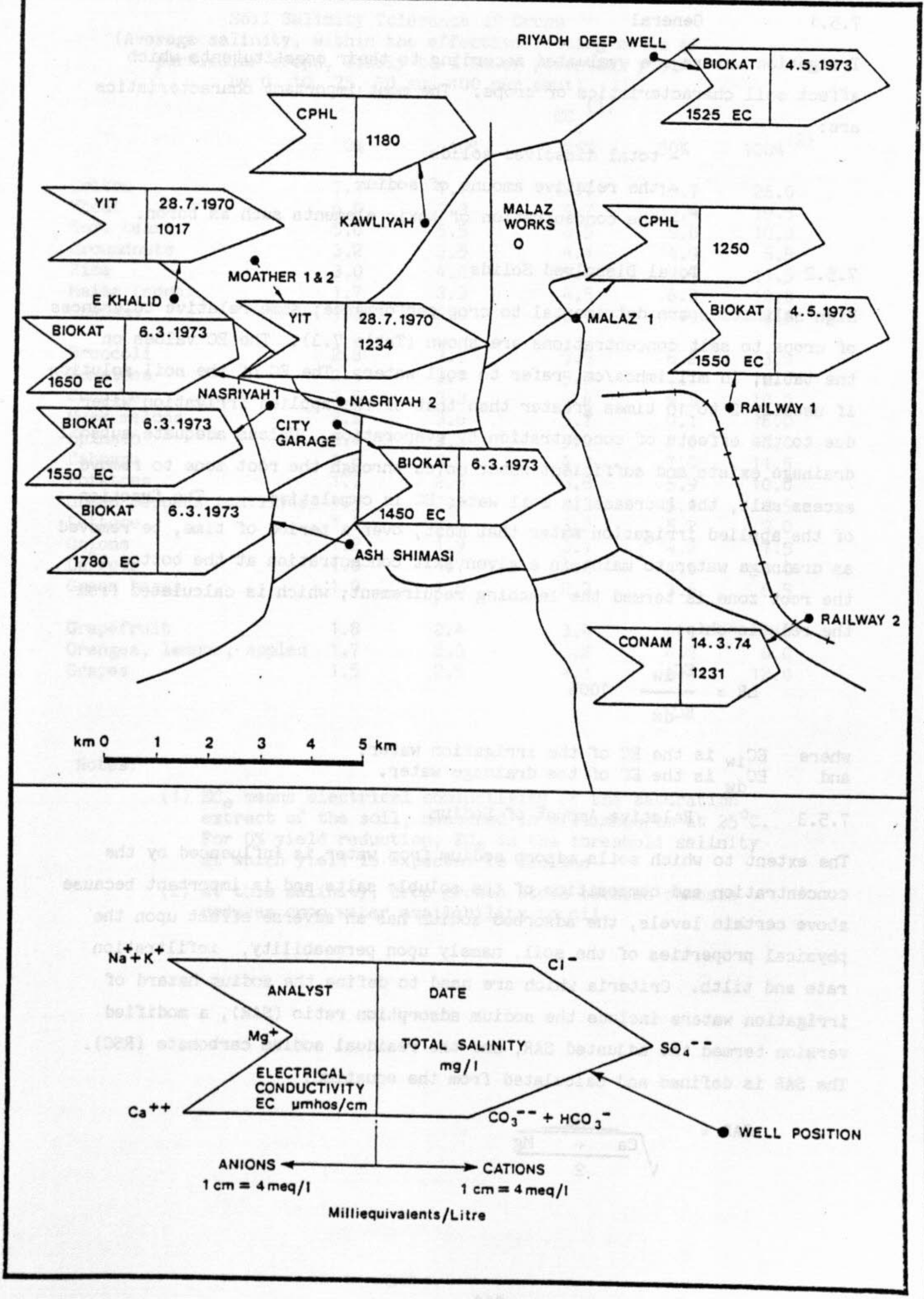
a) Contours of total dissolved solids ppm

b) Contours of ratio SO_4 : Cl

c) Contours of ratio Mg : Ca

FIGURE 7.6

USE OF STIFF DIAGRAMS FOR REPRESENTATION OF CHEMICAL ANALYSES



7.5 Irrigation Water Quality

7.5.1 General

Irrigation waters are evaluated according to their constitutents which affect soil characteristics or crops. The most important characteristics are:

- total dissolved solids
- the relative amount of sodium
- the concentration of toxic elements such as boron.

7.5.2 Total Dissolved Solids

High salinities are detrimental to crop performance; some relative tolerances of crops to salt concentrations are shown (Table 7.3). The EC values on the table, in millimhos/cm, refer to soil water. The EC of the soil solution if usually 2 to 10 times greater than that of the applied irrigation water due to the effects of concentration by evaporation. Unless adequate subsoil drainage exists and sufficient water moves through the root zone to remove excess salt, the increase in soil water EC is cumulative. The fraction of the applied irrigation water that must, over a period of time, be removed as drainage water to maintain a given salt concentration at the bottom of the root zone is termed the leaching requirement, which is calculated from the relationship:

$$LR = \frac{EC_{iw}}{EC_{dw}} \quad 100\%$$

where EC_{iw} is the EC of the irrigation water,
and EC_{dw} is the EC of the drainage water.

7.5.3 Relative Amount of Sodium

The extent to which soils adsorb sodium from water is influenced by the concentration and composition of the soluble salts and is important because above certain levels, the adsorbed sodium has an adverse effect upon the physical properties of the soil, namely upon permeability, infiltration rate and tilth. Criteria which are used to define the sodium hazard of irrigation waters include the sodium adsorption ratio (SAR), a modified version termed the adjusted SAR, and the residual sodium carbonate (RSC). The SAR is defined and calculated from the equation:

$$SAR = \frac{Na}{\sqrt{\frac{Ca \;+\; Mg}{2}}}$$

Table 7.3

Soil Salinity Tolerance of Crops

(Average salinity, within the effective rooting zones of particular crops, likely to a lower potential yield by 0, 10, 25, 50 and 100 per cent)

	$EC_e^{(1)}$				
	0%	10%	25%	50%	100%$^{(2)}$
Cotton	7.7	7.7	12.7	16.7	26.0
Wheat	6.0	7.3	9.7	13.4	19.5
Soya beans	5.0	5.5	6.5	8.0	10.0
Groundnuts	3.2	3.5	4.1	4.9	6.5
Rice	3.0	4.2	5.4	7.5	11.5
Maize (corn)	1.7	3.3	4.5	6.3	10.0
Cowpeas	1.3	2.0	3.1	4.9	8.5
Broccoli	2.8	3.9	5.5	8.2	13.5
Tomatoes	2.5	3.7	5.5	7.7	12.0
Cucumber	2.5	3.3	4.4	6.3	10.0
Musk melons	2.2	3.6	5.7	9.1	16.0
Spinach	2.0	3.3	5.3	8.6	15.0
Cabbage	1.8	2.7	4.3	7.0	11.5
Potatoes	1.7	2.5	3.8	5.9	10.0
Green peppers (chillies)	1.5	2.2	3.3	5.1	8.5
Lettuce	1.3	2.1	3.2	5.2	9.0
Onions	1.2	1.9	3.1	4.2	7.5
Carrots	1.0	1.7	2.8	4.6	8.0
Green beans	1.0	1.5	2.2	3.6	6.5
Grapefruit	1.8	2.4	3.4	4.9	8.0
Oranges, lemons, apples	1.7	2.3	3.2	4.8	8.0
Grapes	1.5	2.5	4.1	6.7	12.0

Notes:

(1) EC_e means electrical conductivity of the saturation extract of the soil, measured in millimhos/cm at 25°C. For 0% yield reduction, EC_e is the threshold salinity at which yield is expected to decline

(2) At this salinity, crop growth stops because osmosis reduces crop water availability to nil.

where the ion concentrations are expressed in meq/l. If an irrigation water has a high SAR (high sodium relative to calcium and magnesium); the sodium may be adsorbed on to the soil replacing calcium and magnesium already there; this reaction can be reversed by adding gypsum. Recently the SAR criterion has been modified to include the effects of the carbonate and bicarbonate content of the water on the adsorption process. The new term is called the adjusted SAR and is defined by the expression:

$$\text{adj. SAR} = \text{SAR}\ (1 + (8.4 - \text{pHc}))$$

where pHc is the theoretical pH of a water in equilibrium with $CaCO_3$. A pHc value greater than 8.4 indicates a tendency for the water to dissolve $CaCO_3$ from the soil, while a value less than 8.4 indicates a tendency for the water to precipitate $CaCO_3$. The adj. SAR is generally greater than the SAR.

The presence of free sodium carbonate or bicarbonate, or 'residual sodium carbonate' (RSC) as it has been termed, in irrigation water is considered to have harmful effects on soil properties. It is calculated by subtracting the calcium and magnesium concentrations from the bicarbonate and carbonate concentrations, all expressed in meq/l. The RSC concept is not now as widely used as before.

7.5.4 Toxic Ions

Three of the most common ions in natural waters which can be toxic to certain crops are boron, lithium and chloride.

Boron, which occurs in most natural waters, is essential for plant growth but the quantity required is very small. It is very toxic to certain plant species, particularly citrus. Table 7.4 shows a classification of irrigation waters according to their boron content for various crops.

Table 7.4

Tolerances of Crops to Boron

Tolerant (2-4 ppm)	Semi-tolerant (1 - 2 ppm)	Sensitive (0.3 - 1 ppm)
Broad bean	Potatoes	Deciduous fruit
Onion	Cotton	Citrus
Cabbage	Tomatoes	Navy beans
Lettuce	Wheat	
Turnip	Maize	
Carrot	Chillies	
Lucerne	Peaks	
	Pumpkins	
	Sweet potatoes	
	Sunflower	
	Lima bean	

Source : Wilcox, L.V. (1960), Boron injury to plants, US Dept. Agric. Bull. No. 211.

Lithium is particularly toxic to citrus; when present in groundwater it is usually derived from micaceous minerals.

Chloride is harmful to some crops if its concentration in the applied irrigation water exceeds about 5 meq/l (180 mg/l), but other crops have much higher tolerances.

7.5.5 Irrigation Water Standards

A multitude of different irrigation quality standards have been proposed over the years. One of the most commonly used is that of the US Salinity Laboratory which classifies waters in term of their SAR and EC (Figure 7.7). It is important when applying this or any other water quality standard to take into account other factors such as soil properties and climate. For example, a high SAR in an irrigation water will obviously be more detrimental to a poorly drained soil containing montmorillonite clay (with its high cation exchange capacity) than to, say, a well drained sandy loam; in the latter case, the SAR of the water may almost be irrelevant. Again, high EC and SAR values will be more harmful to soils in arid areas rather than in high rainfall areas because there will be less chance of the leaching requirement being met.

7.6 Corrosion

7.6.1 Processes

Corrosion is the eating away of a metal by reactions which are electrochemical in nature. An electrochemical action is a chemical change resulting from the flow of electric current between metallic areas of differing potential. Corrosion occurs at the anode. Hydrogen atoms are deposited at the cathode causing polarisation and the slowing down of further corrosion if they are not removed. The conducting fluid, in our case water, is termed the electrolyte.

The anodic - cathodic couples which form corroding cells (galvanic cells) can result from many different causes. These include the coupling of dissimilar metals within a screen-casing string, local impurities in the composition of the metal, differing degrees of surface protection, exposure of different parts to different conditions such as aeration or salt concentration and differences in the treatment of discrete areas of metal, such as cold working, welding and screw cutting. A schematic representation of the corrosion processes under reducing and oxidising conditions is given (Figure 7.8).

FIGURE 7.7 IRRIGATION WATER CLASSIFICATION

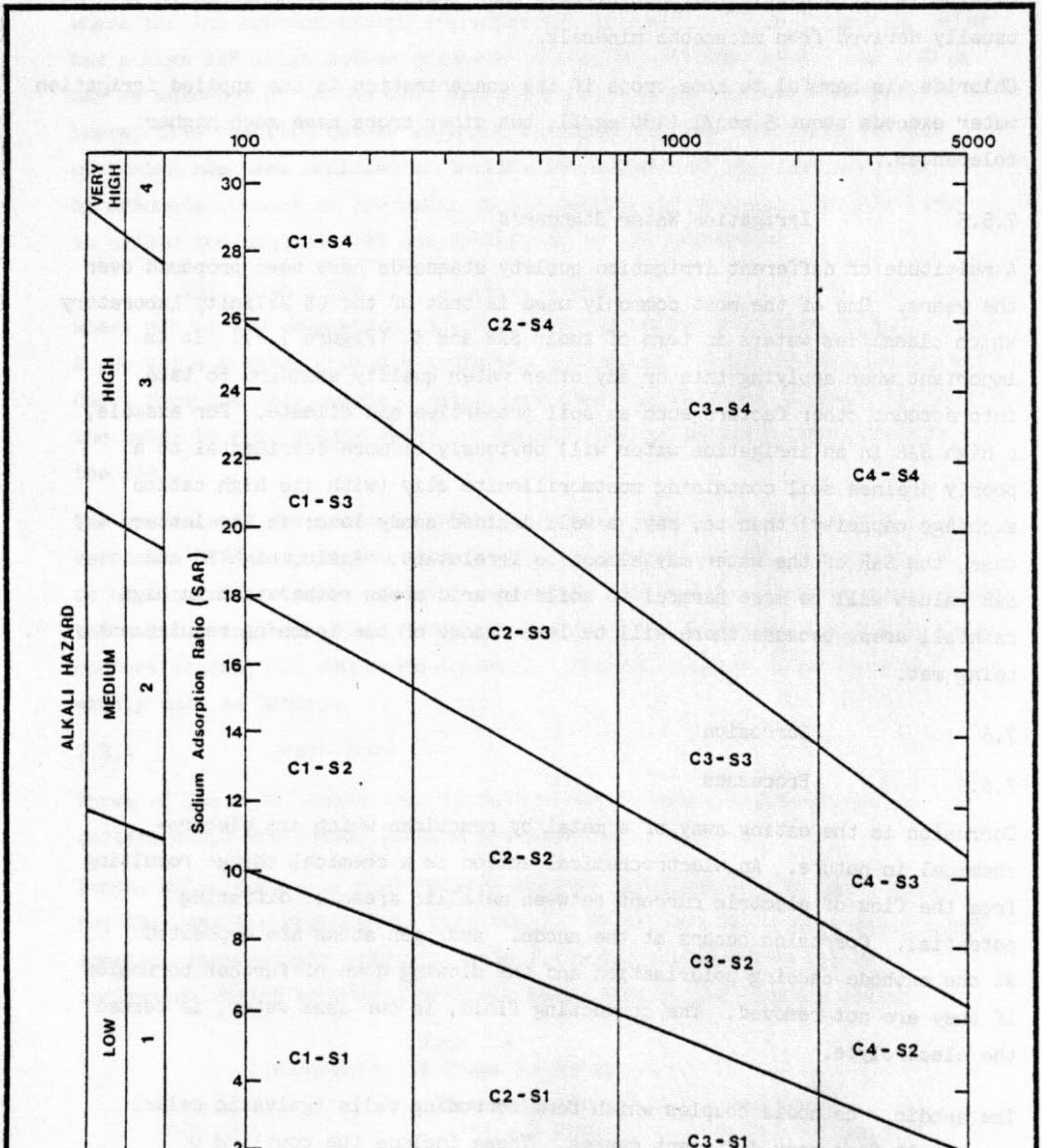


FIGURE 7.8

SCHEMATIC REPRESENTATION OF TWO SIMPLE CORROSION CELLS

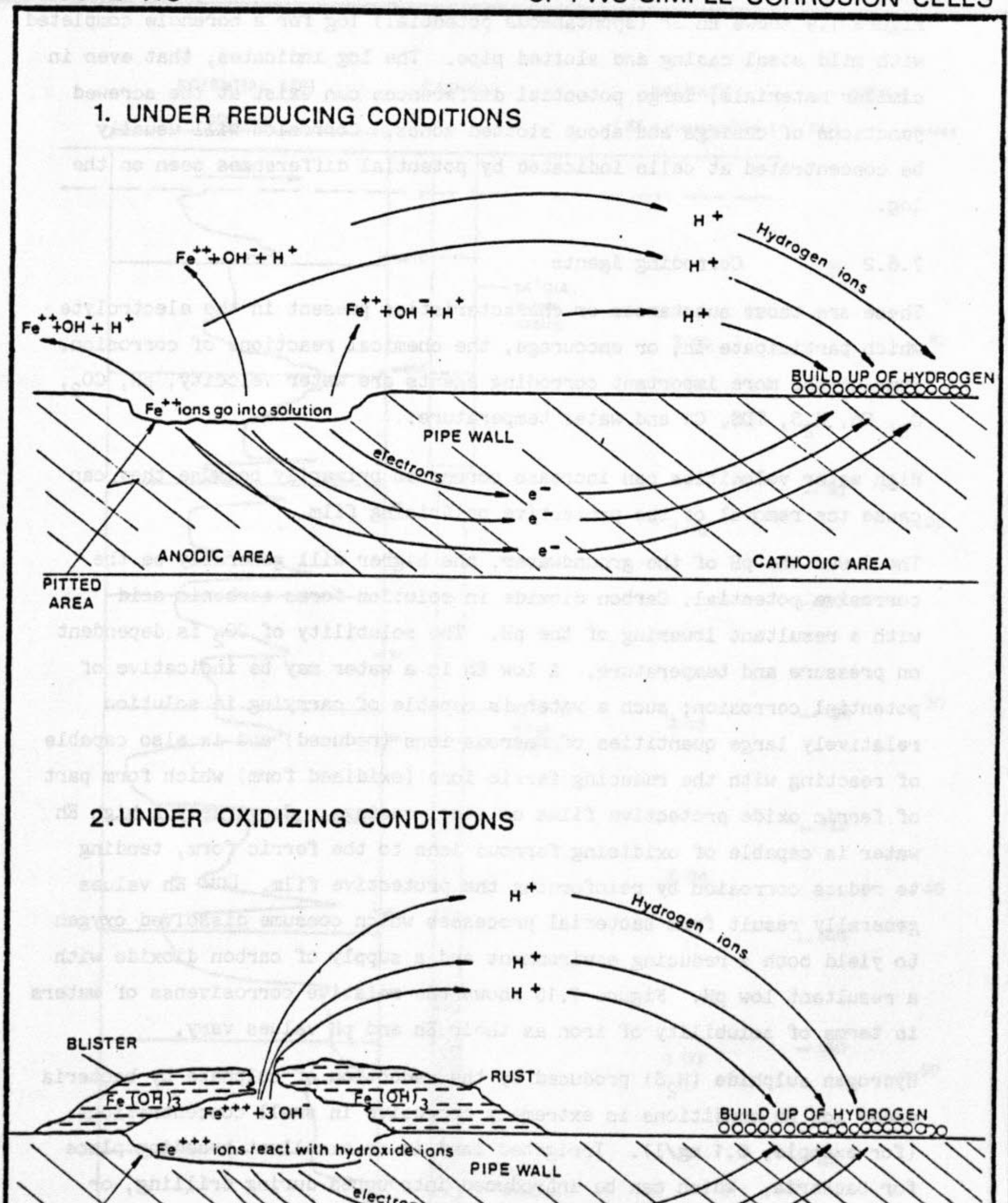


Figure 7.9 shows an SP (spontaneous potential) log for a borehole completed with mild steel casing and slotted pipe. The log indicates, that even in similar materials, large potential differences can exist at the screwed junctions of casings and about slotted zones. Corrosion will usually be concentrated at cells indicated by potential differences seen on the log.

7.6.2 Corroding Agents

These are those substances or characteristics present in the electrolyte which participate in, or encourage, the chemical reactions of corrosion. Some of the more important corroding agents are water velocity, pH, CO_2, O_2, Eh, H_2S, TDS, C1 and water temperature.

High water velocities can increase corrosion primarily because they can cause the removal of the protective polarising film.

The lower the pH of the groundwater, the higher will generally be the corrosive potential. Carbon dioxide in solution forms carbonic acid with a resultant lowering of the pH. The solubility of CO_2 is dependent on pressure and temperature. A low Eh in a water may be indicative of potential corrosion; such a water is capable of carrying in solution relatively large quantities of ferrous ions (reduced) and is also capable of reacting with the reducing ferric ions (oxidised form) which form part of ferric oxide protective films on steel casings. Conversely a high Eh water is capable of oxidising ferrous ions to the ferric form, tending to reduce corrosion by reinforcing the protective film. Low Eh values generally result from bacterial processes which consume dissolved oxygen to yield both a reducing environment and a supply of carbon dioxide with a resultant low pH. Figure 7.10 shows the relative corrosivenss of waters in terms of solubility of iron as their Eh and pH values vary.

Hydrogen sulphide (H_2S) produced by the reduction of sulphate by bacteria under low Eh conditions is extremely corrosive in small concentrations (for example, 0.1 mg/l). Irrigated land is an excellent breeding place for bacteria, which can be introduced into bores during drilling, or which may permanently infect the aquifers.

The total salt concentration and composition of the groundwater also materially affect corrosion rates. In general, the higher the concentration the more corrosive will be the environment. Certain salts, such as any form of chloride, are more important than others because of their capability

FIGURE 7.9 POTENTIAL LOG OF A CASED TUBEWELL

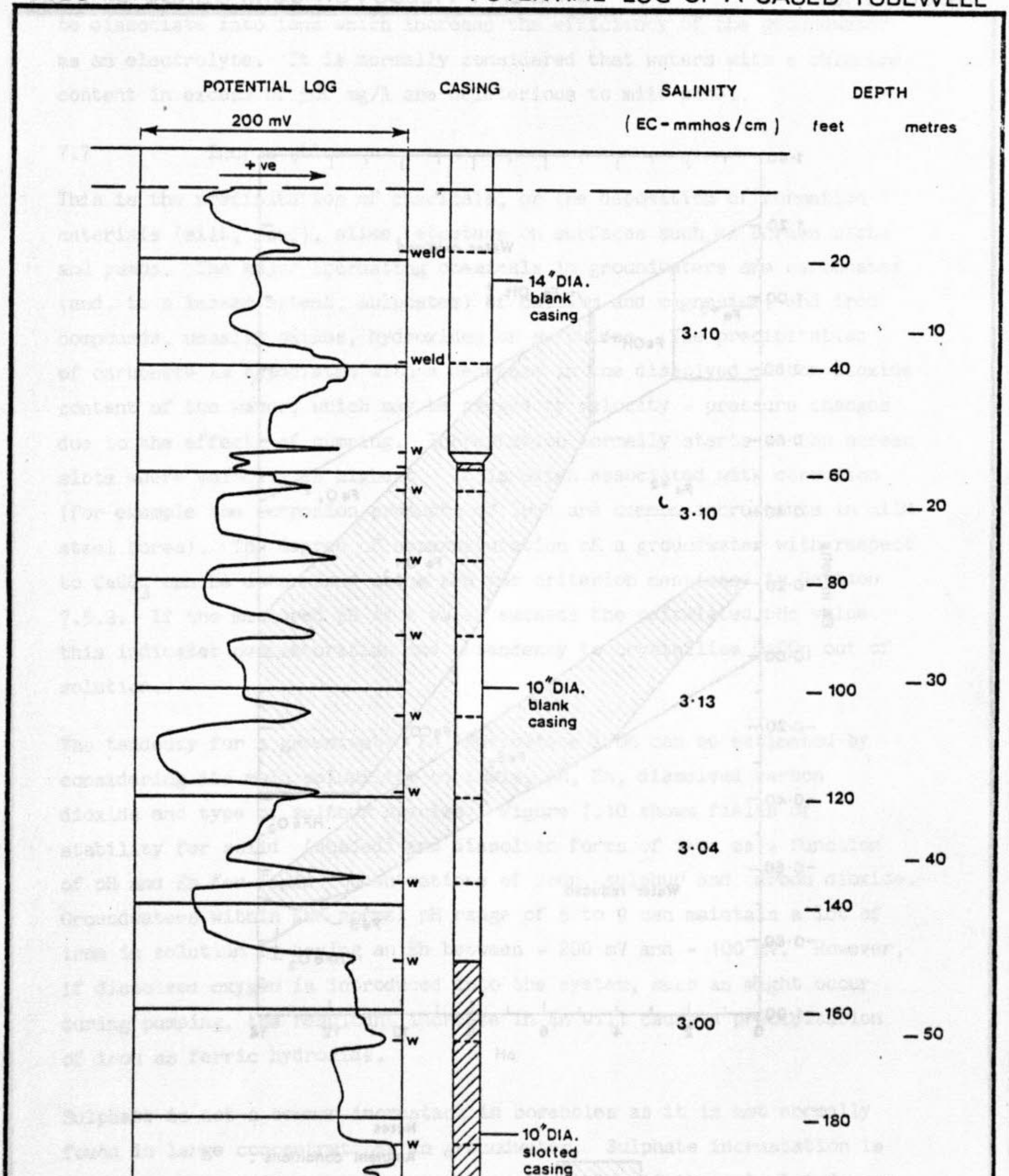
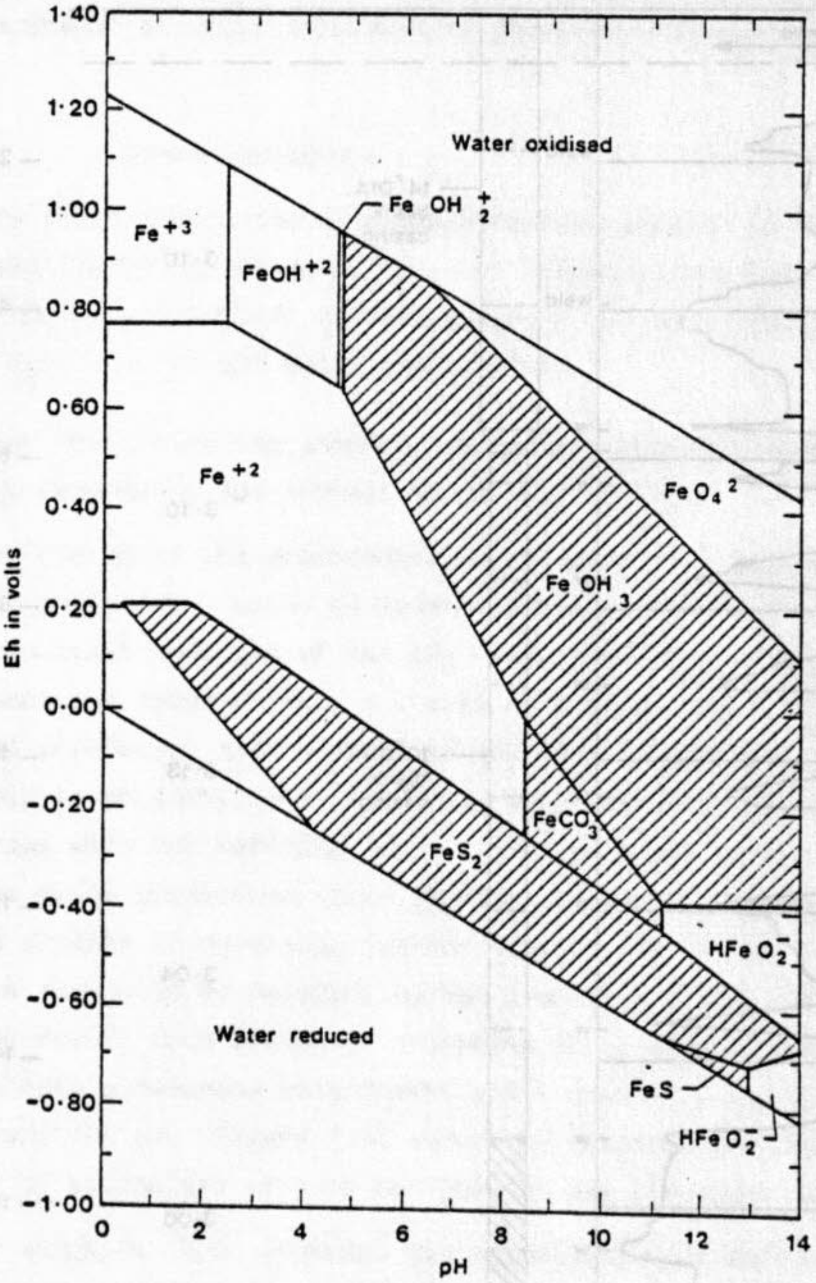


FIGURE 7.10 STABILITY FIELDS FOR COMPOUNDS OF IRON



Insoluble forms ▨

Soluble forms □

Notes

Ambient conditions :

1. 25°C at 1 atmosphere pressure
2. Sulphur activity 96 mg/l as SO_4
3. Carbon dioxide activity 1000 mg/l as HCO_3
4. Dissolved iron 0·0056 mg / l

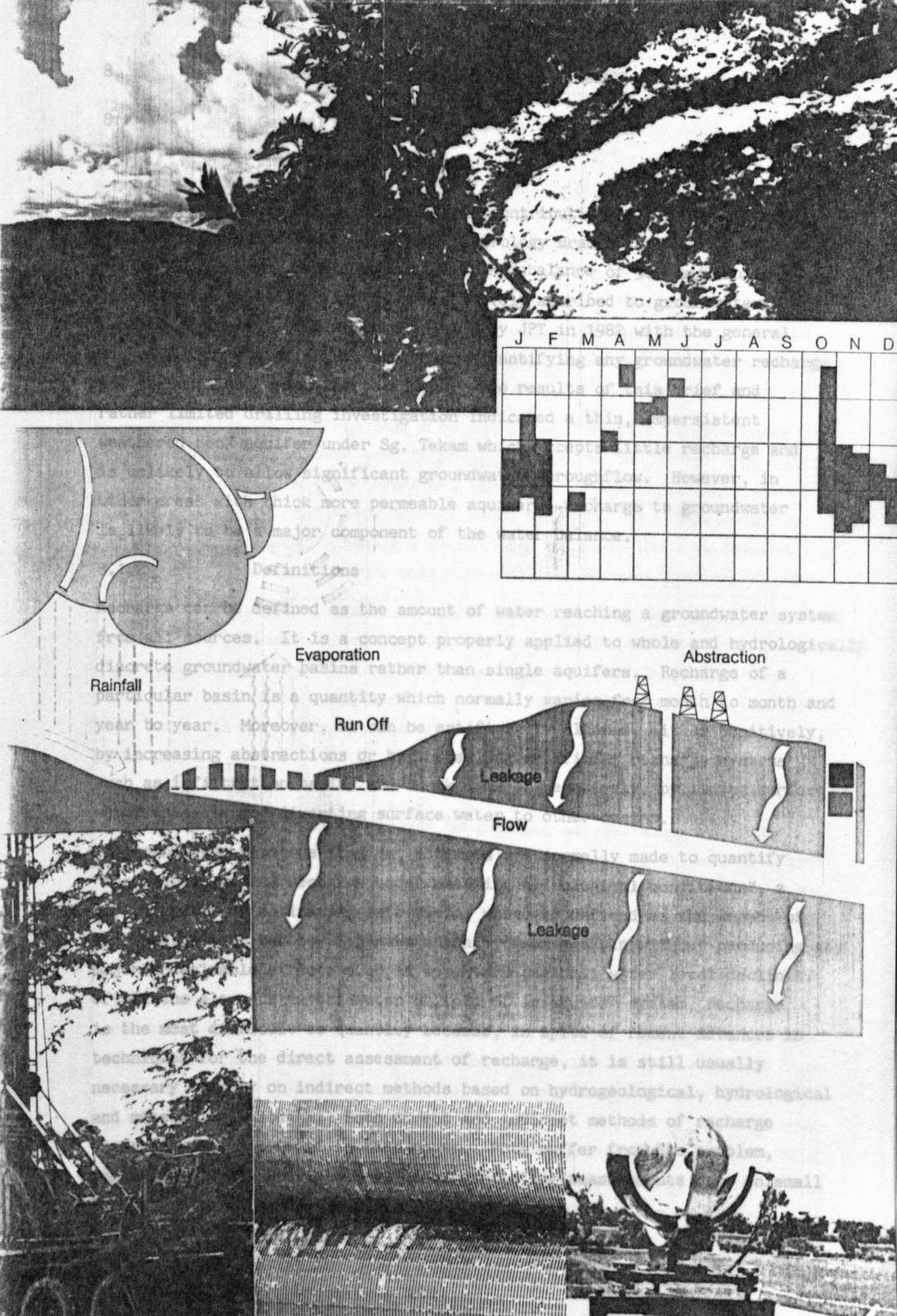
to dissociate into ions which increase the efficiency of the groundwater as an electrolyte. It is normally considered that waters with a chloride content in excess of 500 mg/l are deleterious to mild steel.

7.7 Incrustation

This is the precipitation of chemicals, or the deposition of formation materials (silt, sand), slime, etcetera on surfaces such as screen slots and pumps. The major incrusting chemicals in groundwaters are carbonates (and, to a lesser extent, sulphates) of calcium and magnesium, and iron compounds, usually oxides, hydroxides or sulphides. The precipitation of carbonate is associated with a decrease in the dissolved carbon dioxide content of the water, which may be caused by velocity - pressure changes due to the effects of pumping. Incrustation normally starts at the screen slots where velocity is highest. It is often associated with corrosion (for example the corrosion products of iron are common incrustants in mild steel bores). The degree of supersaturation of a groundwater with respect to $CaCO_3$ can be determined using the pHc criterion mentioned in Section 7.5.3. If the measured pH of a water exceeds the calculated pHc value, this indicates oversaturation and a tendency to crystallise $CaCO_3$ out of solution.

The tendency for a groundwater to precipitate iron can be estimated by considering its main solubility controls, pH, Eh, dissolved carbon dioxide and type of sulphur species. Figure 7.10 shows fields of stability for solid (shaded) and dissolved forms of iron as a function of pH and Eh for fixed concentrations of iron, sulphur and carbon dioxide. Groundwaters within the normal pH range of 5 to 9 can maintain a lot of iron in solution by having an Eh between + 200 mV and - 100 mV. However, if dissolved oxygen is introduced into the system, such as might occur during pumping, the resultant increase in Eh will cause a precipitation of iron as ferric hydroxide.

Sulphate is not a common incrustant in boreholes as it is not normally found in large concentrations in groundwater. Sulphate incrustation is usually only considered to be a potential problem if the calculated concentration of calcium sulphate in the water is in excess of 25 meq/l.


J F M A M J J A S O N D
Evaporation
Abstraction
Rainfall
Run Off
Leakage
Flow
Leakage

8. ASSESSMENT OF NATURAL AQUIFER RECHARGE

8.1 General

8.1.1 Introduction

The JPT Groundwater Group has made some contribution to a water balance studies being carried out by the JPT Hydrology Branch in the Sg. Tekam catchment, Pahang; part of an unexplained inbalance of 500 mm/y in the catchment water balance had been tentatively ascribed to groundwater outflow. Three boreholes were drilled by JPT in 1982 with the general aim of defining aquifer occurrence and quantifying any groundwater recharge and throughflows under the catchment. The results of this brief and rather limited drilling investigation indicated a thin, impersistent weathered rock aquifer under Sg. Tekam which accepts little recharge and is unlikely to allow significant groundwater throughflow. However, in other areas with thick more permeable aquifers, recharge to groundwater is likely to be a major component of the water balance.

8.1.2 Definitions

Recharge can be defined as the amount of water reaching a groundwater system from all sources. It is a concept properly applied to whole and hydrologically discrete groundwater basins rather than single aquifers. Recharge of a particular basin is a quantity which normally varies from month to month and year to year. Moreover, it can be artificially altered, either positively, by increasing abstractions or by installing artificial recharge systems such as infiltration basins and check dams or negatively, by lining surface water channels and diverting surface water to other basins.

In groundwater resource studies, attempts are normally made to quantify "the average annual recharge under existing hydrological conditions", a key parameter in estimating safe yield, which is defined as the amount of groundwater that can be withdrawn annually from a basin without producing any major undesirable effects such as long term regional water level declines. Of all the elements in the water balance of an aquifer system, recharge is the most difficult to quantity because, in spite of recent advances in techniques for the direct assessment of recharge, it is still usually necessary to rely on indirect methods based on hydrogeological, hydrological and meteorological data. Both direct and indirect methods of recharge assessment are discussed in this paper. Both suffer from the problem, common throughout hydrogeology, of extrapolating measurements made on small samples to large, non-uniform areas.

8.1.3 Identification of Recharge Areas

Figure 8.1 shows unconfined groundwater flow in a basin in a humid area. It is clear from this flow net that water moves through the aquifer from the interfluve highlands to the river valley, from recharge areas to the discharge area. As can be seen, under recharge areas the groundwater flow must have a downward component and beneath discharge areas there must be an upward flow component. This fact gives one way of identifying recharge and discharge areas of a basin. A recharge area has net saturated flow directed away from the watertable; that is, there is a downward pressure gradient (pressure decreases with depth). In discharge areas there is normally an upward pressure gradient.

The example shown (Figure 8.1) is one of a gaining (or effluent) river whilst some rivers, or at least portions of some rivers, are losing or influent, that is, their water levels are above those of the adjacent groundwater bodies and there is a net loss of water from the river to the groundwater system. The flow net of such a situation would be practically identical to that shown except that the flow direction would be reversed.

If rainfall and/or river flow is strongly seasonal, the direction of groundwater flow in a river basin may be temporarily reversed, particularly if there are significant groundwater abstractions in the interfluves. Nevertheless, in all these case the definitions given above hold true; that is, in recharge areas pressure (or head) decreases with depth and in discharge areas it increases with depth; this is the evidence that is most easily observed in the field.

Additional evidence for the presence of recent recharge in an aquifer (and thus proximity of a recharge area) can be obtained form the chemical and isotopic composition of the groundwater. Groundwater moving through an aquifer from recharge to discharge areas, particularly under confined conditions, shows fairly consistent chemical changes, which are illustrated (Figure 8.2). Thus calcium bicarbonate water with detectable dissolved oxygen usually indicates local recharge. The presence of the radioactive isotope tritium (H^3) indicates at least a component of recharge within the last thirty years, whereas the proportions of the stable isotopes deuterium (H^2) and oxygen-18 in the water can identify the climatic conditions at the time of recharge and, by inference, its age, provided that the climatic history of the region is known. Other ways of identifying recharge areas can be purely geological, such as the outcrops of confined, dipping aquifer beds, or hydrological such as rivers whose channels cut into aquifer layers.

FIGURE 8.1 FLOW NET ACROSS A GROUNDWATER BASIN

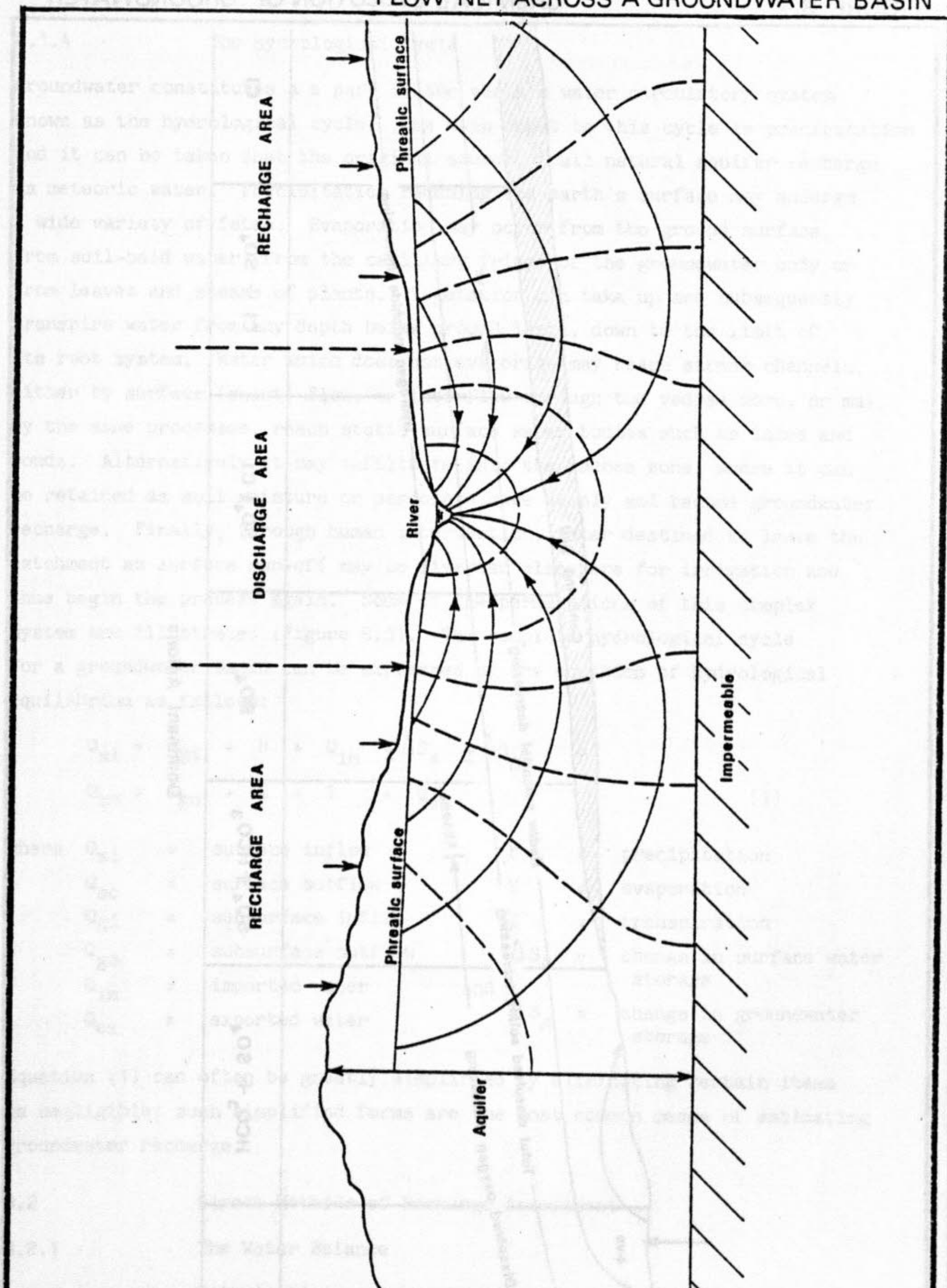
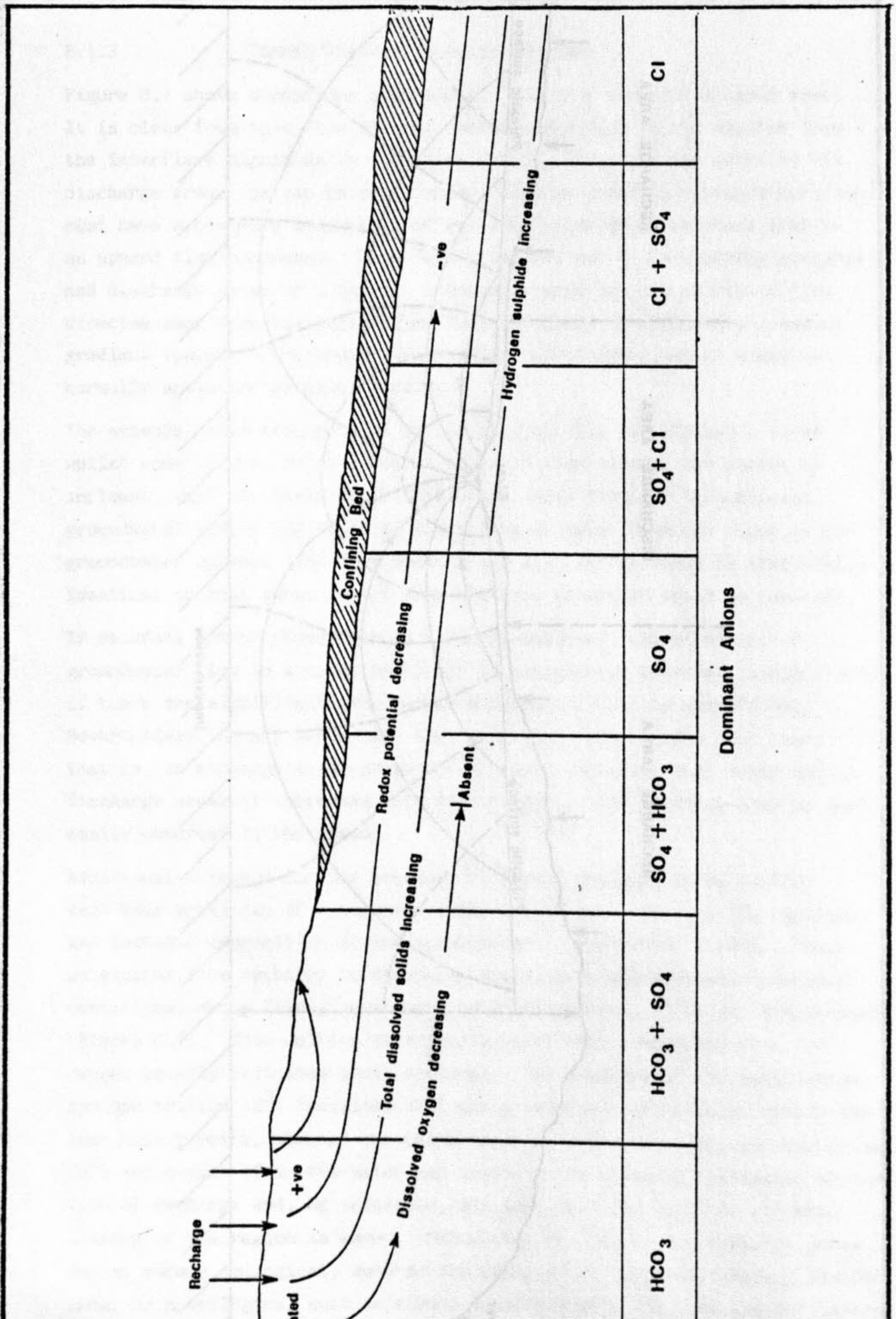


FIGURE 8.2 CHEMICAL EVOLUTION OF GROUNDWATER



8.1.4 The Hydrological Cycle

Groundwater constitutes a a part of the earth's water circulatory system known as the hydrological cycle. The main input to this cycle is precipitation and it can be taken that the original source of all natural aquifer recharge is meteoric water. Precipitation reaching the earth's surface may undergo a wide variety of fates. Evaporation may occur from the ground surface, from soil-held water, from the capillary fringe of the groundwater body or from leaves and steams of plants. Vegetation can take up and subsequently transpire water from any depth below ground level, down to the limit of its root system. Water which does not evaporate may reach stream channels, either by surface (sheet) flow, or interflow through the vadose zone, or may, by the same processes, reach static surface water bodies such as lakes and ponds. Alternatively it may infiltrate into the vadose zone, where it can be retained as soil moisture or percolate more deeply and become groundwater recharge. Finally, through human intervention, water destined to leave the catchment as surface run-off may be diverted elsewhere for irrigation and thus begin the process again. Some of the permutations of this complex system are illustrated (Figure 8.3). The complete hydrological cycle for a groundwater basin can be expressed by the equation of hydrological equilibrium as follows:

$$Q_{si} + Q_{gi} + P + Q_{im} \pm \Delta S_s \pm \Delta S_g = Q_{so} + Q_{go} + E + T + Q_{ex} \qquad (1)$$

where Q_{si} = surface inflow
Q_{so} = surface outflow
Q_{gi} = subsurface inflow
Q_{go} = subsurface outflow
Q_{im} = imported water
Q_{ex} = exported water
P = precipitation
E = evaporation
T = transpiration
ΔS_s = change in surface water storage
and ΔS_g = change in groundwater storage

Equation (1) can often be greatly simplified by eliminating certain items as negligible; such simplified forms are the most common means of estimating groundwater recharge.

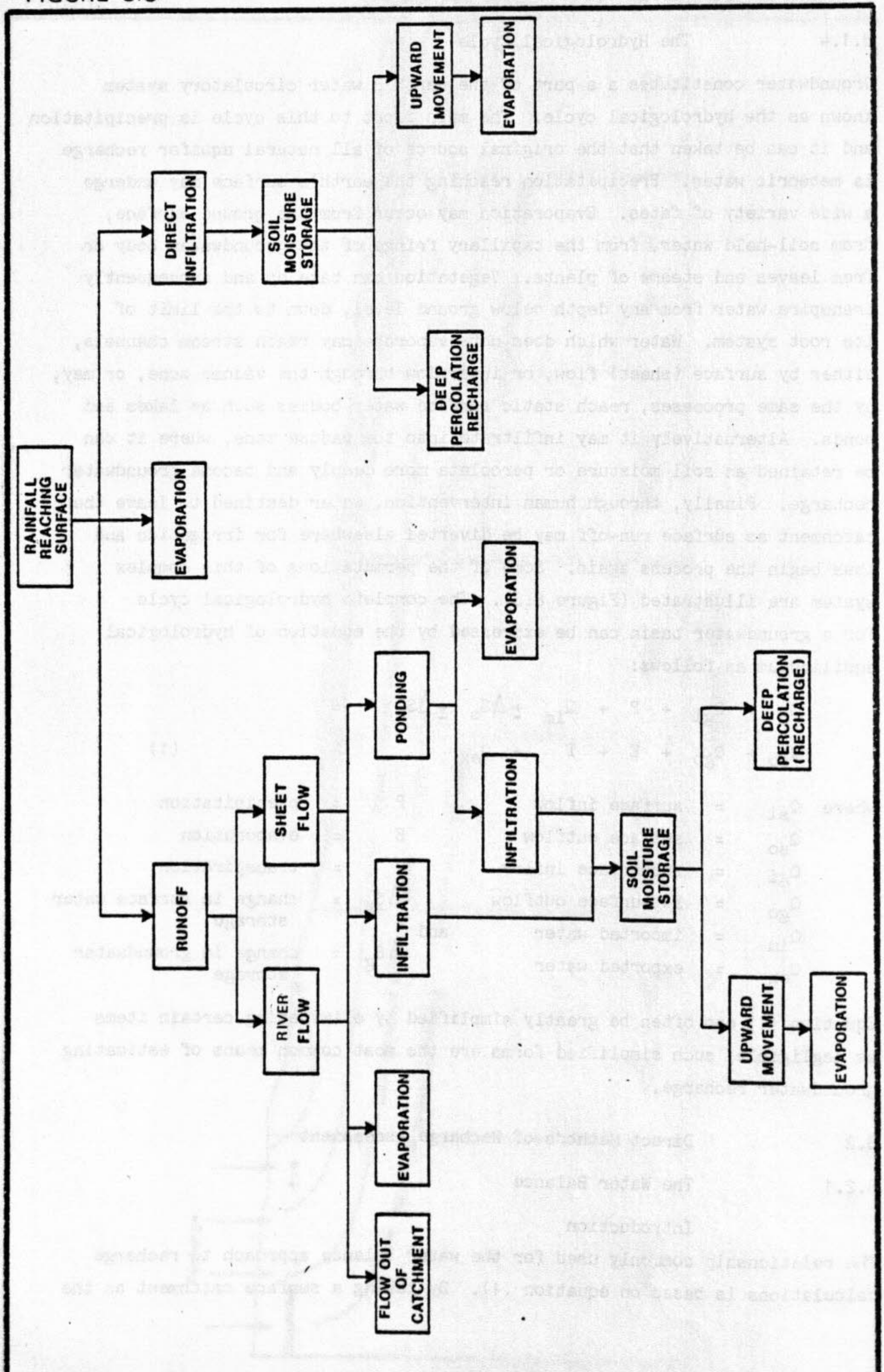
8.2 Direct Methods of Recharge Assessment

8.2.1 The Water Balance

Introduction

The relationship commonly used for the water balance approach to recharge calculations is based on equation (1). By taking a surface catchment as the

RECHARGE MECHANISMS



areal unit, the mass balance equation may sometimes be reduced to:

$$R = P - ET - S_s - RO \qquad (2)$$

where R = recharge
P = precipitation
ET = evapotranspiration
S_s = soil storage
and RO = surface run-off

To use equation (2) for recharge calculations, it is necessary to estimate the four components - precipitation, evapotranspiration, soil storage and run-off. Beside the absolute mass balance, the rates of precipitation, evatranspiration and infiltration, relative to each other, are highly significant in determining the portion of precipitation which becomes aquifer recharge.

All measurements of meteorological parameters and soil properties suffer from the deficiency of being made on extremely small samples of large, usually non-uniform, systems. An understanding of the ways in which these measurements can be best interpreted is thus essential to an appreciation of the problems of recharge assessment.

Precipitation

Although much work has been carried out in recent years on the use of radar for direct measurmeent of areal rainfall, it is still usual to gauge precipitation at a point and then extrapolate these data to estimate precipitation over the area of interest. The object of point precipitation gauging is to measure the rainfall, which would have fallen at that point, had the gauge not been there. Numerous designs of rain gauge are in use throughout the world and there is little standardisation. The main problems, which gauge design and siting must overcome, are errors caused by the following:

- underexposure or overexposure
- the effect of eddies
- splashing
- evaporation
- leakage and overflow
- blockage
- human or animal interference

Correct siting is obviously very important. Overshadowing can be avoided by maintaining a separation between the gauge and the nearest object of at least twice the height of the object. The effect of the gauge on local air flow, causing eddies which lift the air stream over the gauge and reduce the catch, is usually the most important source of error. Besides avoiding obviously exposed sites, the eddy effect can be minimised by keeping the gauge orifice as close to the ground as possible, without allowing splash in from surrounding ground. Splashing into the gauge is minimised by surrounding it with short grass or soft and whilst splashing out of gauge, evaporation and blockage can be largely avoided by good gauge design.

The number of gauges required to assess precipitation over a particular area depends n its size topography, the climate and, of course, on the accuracy required; recommendations on gauge spacing are given by the World Meteorological Organisation. Ideally rain gauges should be sited so as to fairly reflect the distribution of altitude and vegetation in the area concerned; in practice, they have usually to be positioned so as to coincide with observer availability.

Aerial precipitation estimates are obtained by attributing the gauge point precipitation values to specific areas. Three weighting systems are in common use; these are described below:

- the Theissen polygon method is the most widely used technique. Polygons are drawn by joining the rain gauge points in triangular network and then bisecting the sides of each triangle. As any point in the area enclosed by a polygon is nearer to the gauge at its centre than to any other, the rainfall measured at that gauge is attributed to the whole area of the polygon.
- in the triangulation method, rainfall within a triangle is taken as the mean of that measured at the rain gauges at its apexes.
- Isohyets are contours linking points of the same rainfall and are particularly useful for assessing rainfall in areas where there are rapid local variations.

Using one of the above methods, total annual rainfall on some given area can be expressed as volume of water, which is sutiable as an input to equation (2).

Evapotranspiration

Evaporation and transpiration are physically similar processes which cannot be really separated and are therefore grouped together under evapotranspiration. Open water potential evaporation is normally assessed either by measuring the loss from an open pan or by calculation from other measured meteorological parameters. Evaporation pans are conceptually very simple but even for evaporation from lake or pond surfaces, the pan results need correction because of the disparity of size of the water bodies. For evaporation from a land surface, further corrections are required and for these reasons, calculated estimates of evaporation are usually preferable.

Evaporation is contolled by two processes, namely the energy balance and the aerodynamics of which the former usually predominates. These processes are illustrated separately (Figure 8.4). Although it is possible to evaluate evaporation by a consideration of the energy balance alone, the Penman method (Penman, 1948) offers a means of calculating evaporation and potential evapotranspiration by combining the energy balance and aerodynamic processes. This has gained wide acceptance. Penman's equation, in general form, can be written as:

$$E_o = \frac{\Delta}{\Delta+\gamma}\underbrace{\left[R_A\ (1-\alpha)\ (a + b\tfrac{n}{N}) - \sigma T^4\ (c - d\sqrt{e})\ (0.1 + 0.9\tfrac{n}{N})\right]}_{\text{energy balance term}} + \underbrace{\left[\frac{\gamma}{\Delta+\gamma}\ 0.27\ (f + gu)\ (e_s - e)\right]}_{\text{aerodynamic term}} \qquad (3)$$

where E_o = potential evaporation, mm/d

Δ = slope of saturation vapour pressure/temperature curve at temperature T

γ = psychrometric constant

α = albedo (proportion of incoming radiation reflected from the particular surface considered)

R_A = incoming short wave radiation, cal/cm^2

n = actual hours of sunshine

N = theoretical hours of sunshine

σ = Stefan Boltzman constant

T = mean air temperature ($^{\circ}$ Kelvin)

e = mean vapour pressure, mm Hg

e_s = saturation vapour pressure, mm Hg

u = wind run, km/d

a, b, c, d, f and g are constants which vary on a regional basis.

For tropical countries a commonly used form of equation (3) is that after Glover and McCulloch (1958) where:

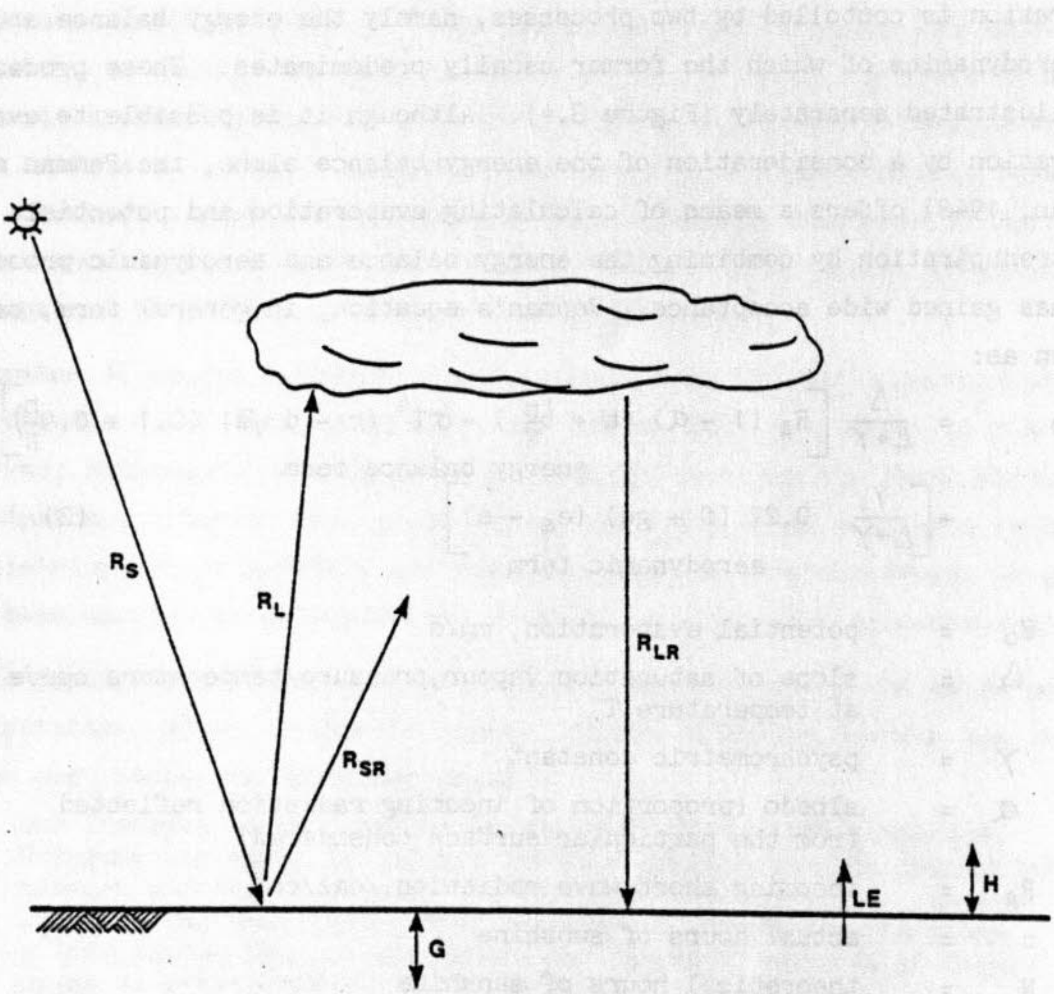
a = 0.29 cos Ø (Ø = latitude) d = 0.08

b = 0.52 f = 1

c = 0.56 g = 0.0067

Although the Penman formula has been found to work well under varied conditions, it does contain a number of approximations and potential inaccuracies. Most obvious among these is the omission of soil heat flux, the assumption here is that, if a long enough period (usually 10 days) is considered, the net effect will be negligible. Another weakness is the fact that measurements taken at 2 m height, are used to evaluate the evaporation process at the earth's surface.

FIGURE 8.4 ENERGY BALANCE AT EARTH'S SURFACE



$R_n = H + LE + G$ where $R_n = (R_S - R_{SR}) - (R_L - R_{LR})$

also $R_{SR} = \alpha R_S$ where α = albedo

$R_{LR} = \varepsilon R_L$ where ε = atmospheric emissivity

where R_n = net radiation

H = sensible heat transfer between surface and air

LE = heat used in converting liquid to vapour, L being latent heat and E, the evaporation

G = heat flux in surface layer of the earth

R_S = incoming solar radiation

R_{SR} = reflected solar radiation

R_L = emitted radiation

R_{LR} = long wave radiation absorbed by water vapour in the atmosphere

In order to calculate evaporation using the Penman method, the meteorological records are necessary for temperature, relative humidity, wind speed (at 2 m height) and sunshine hours or solar radiation. The other factors used are derived from the above measurements, or are obtained from standard tables. The Penman equation gives potential evaporation or evapotranspiration. This is applicable only to systems in which water is not limiting. When water supply is limiting, actual evaporation or evapotranspiration proceeds at a slower rate than potential depending on the degree of moisture deficiency in the soil. This subject is discussed further later in this chapter.

Due to the expense of setting up and maintaining complex meteorological stations, it is usually necessary to work with much lower data density for evapotranspiration than for rainfall. Such complex stations are usually found only at airports and major agricultural development and research stations.

Soil Moisture

(i) Infiltration

Infiltration is the process of movement of water from the surface into the soil or the superficial deposits. The rate at which infiltration occurs can be a crucial factor in how much recharge an aquifer receives. Infiltration is a combination of two physical processes. Flow by gravity takes place through the larger openings of the soil, is relatively rapid and involves appreciable quantities of water. Capillary forces then disperse the water through the smaller pores; these may act in any direction according to the moisture gradient, that is, from wet to dry. Movement under capillarity is relatively slow and the quantity of water in motion is small.

The maximum rate at which water can enter the soil at a particular point under a given set of conditions is termed the infiltration capacity. Infiltration capacity is not a constant, but varies with time during the course of a single precipitation event, as illustrated (Figure 8.5). The infiltration curve is described by an equation of the form:

$$f_t = f_c + (f_o - f_c)\, e^{-kt} \qquad (4)$$

where f_t = infiltration rate at any time

f_o = initial infiltration capacity

f_c = final infiltration rate (constant)

t = time

k = constant dependent on the nature of the soil

FIGURE 8.5 VARIATION OF INFILTRATION RATE WITH TIME

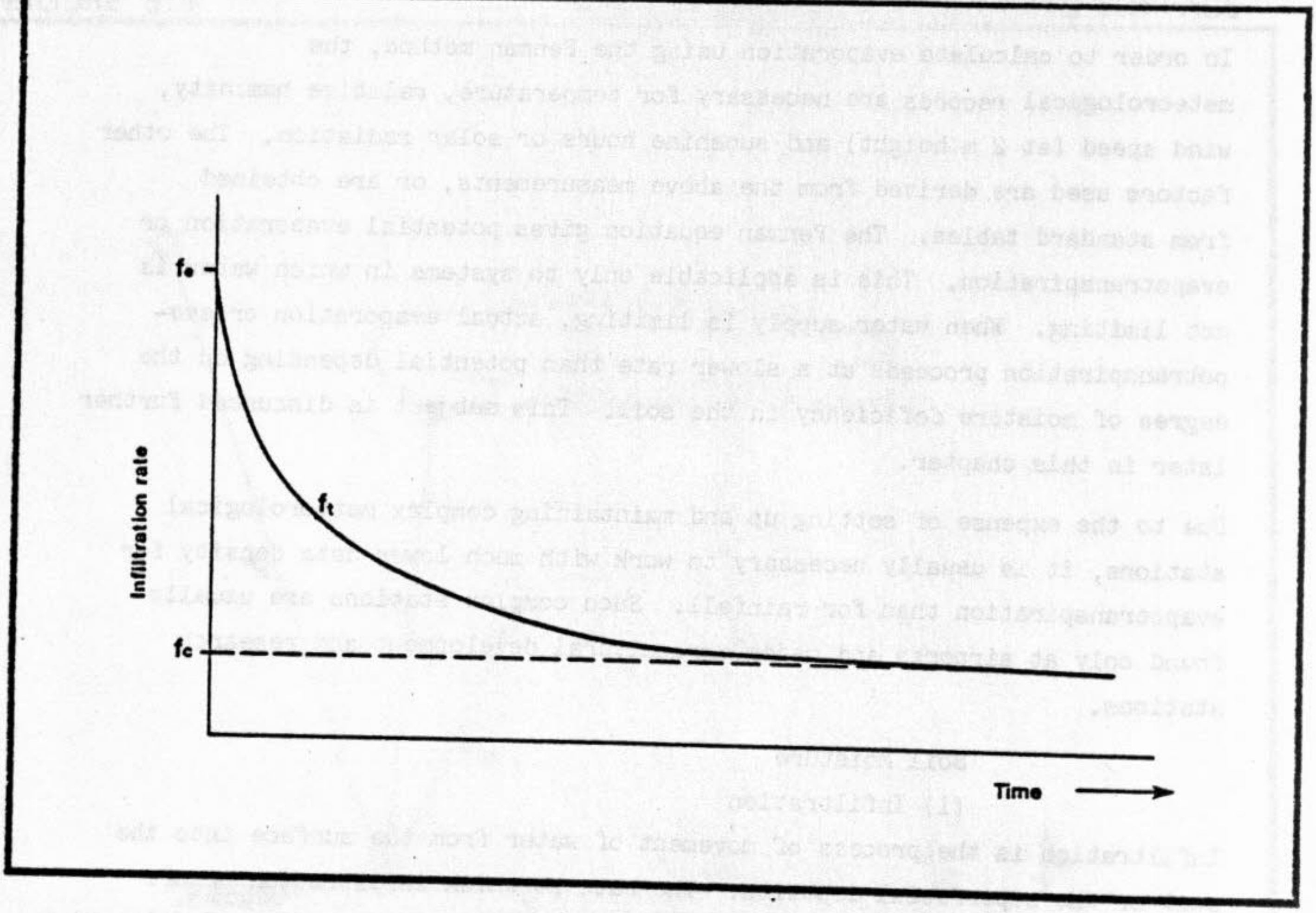
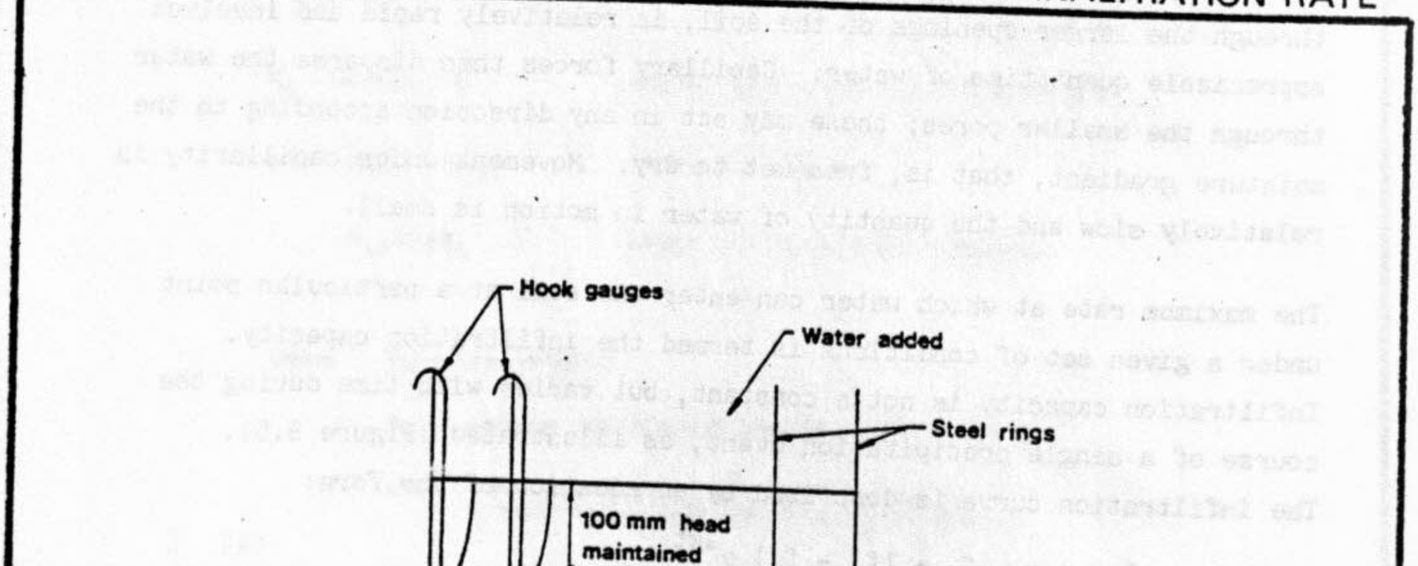


FIGURE 8.6 MEASUREMENT OF INFILTRATION RATE



The asymptotic value, f_c, is a function of soil permeability under saturated conditions. The initial infiltration capacity depends also on the previous precipitation history. Field determinations of infiltration capacity, often an important part of a recharge assessment study, are normally made using two concentric rings driven into the soil, as illustrated (Figure 8.6). A constant head is maintained within the rings and the volume of water added to the central ring is recorded against time. The normal range of infiltration rates for various soil types is shown in Table 8.1.

Table 8.1

Representative Infiltration Rates and Water Retention Properties of Common Soil Types

Soil texture	Infiltration rate (mm/h)	Total porosity %	Total moisture content at field capacity %	Available moisture %
Sandy	25 - 250	32 - 42	6 - 12	6 - 10
Sandy loam	10 - 80	40 - 47	10 - 18	9 - 15
Loam	7.5 - 20	43 - 49	18 - 26	14 - 20
Clay loam	2.5 - 15	47 - 51	23 - 31	16 - 22
Silty clay	0.3 - 5	49 - 53	27 - 35	18 - 23
Clay	1.2 - 10	51 - 55	31 - 39	20 - 25

(ii) Retention of Water by the Soil

All soils have a water retention capability, termed the field capacity, which must be satisfied before significant deep percolation and groundwater recharge can occur. Water is retained in the soil by a number of different mechanisms including chemical bonding to minerals and soil colloids, surface attraction by soil particles and capillary retention within small pore spaces. During dry periods, some of this water may be removed from the soil by evapotranspiration, thereby depleting the moisture content below the field capacity. Water from the upper part of the soil prfoile, usually the top 15 to 30 cm, may be removed by capillary transfer to the evaporating surface. Moisture from the deeper soil layers is normally withdrawn by plant transpiration and only part of the total soil water can be thus removed (the available moisture). Thus the extent to which a soil moisture deficit can develop is determined principally by the type of vegetation and, in particular, on its root depth. This is normally expressed by the root constant. The root constant is a measure of soil water readily available within the root range, expressed as an equivalent rainfall; thus it is not merely a soil property within the rooting depth of a particular plant assemblage but also takes into account the ability of the vegetation to extract water at high moisture tensions. A root constant of 75 mm implies that, during a dry period, evapotranspiration proceeds at the potential rate until a deficit of 75 mm has been built up. After that, evapotranspiration decreases until the permanent wilting point is reached and the plants can extract no more water from the soil.

Theoretically no recharge can occur whilst a soil moisutre deficit exists; any rainfall will first go towards restoring the soil to field capacity. In practice, some recharge does occur at times of moisture deficit, partly due to the enhancement of soil permeability by shrinkage and cracking, and partly due to the normal presence of highly transmissive preferential flow paths in naturally porous media.

Calculation of Recharge

(i) Large Scale Water Balance

Recharge can sometimes be estimated by using equation (2) in its broadest context, at a regional scale. In this approach, precipitation is integrated areally by one of the methods discussed above. The change in the soil storage is taken as zero by assuming that over some convenient water year, the moisture held in the soil at the end of the period is the same as that held at its beginning. Run-off should, of course, be gauged at some suitable point. Actual evapotranspiration is most difficult to estimate but it is possible to make a reasonable estimate; in the rainy season it is often valid to assume that the actual evaporation is equal to the potential; in the dry season an allowance is normally made for field capacity at the beginning and thereafter evaporation from the watertable can be estimated using a function such as that developed by Gardner and Fireman (1958), relating evaporation to depth of watertable. In forest areas, transpiration by trees from the watertable should also be allowed for. Run-off may be difficult to estimate, particularly if the area considered cannot be easily isolated in terms of hydrology.

Despite the difficulties described above, a general water balance is very often a useful first estimate of recharge. It also provides a general check in so far that annual recharge anywhere seldom exceeds 30% of precipitation and is much more often of the order of 5 or 10% of the total rainfall.

(ii) Soil Moisture Budget

The attraction of the soil moisture balance method is that it does not require large scale measurement of run-off, and that it treats the recharge problem on the micro scale, that is on the basis of small areal units, although of course ultimately the results have to be extrapolated to the whole project area. The method makes use of standard hydrometeorological data and is based on the concept of soil moisture deficit described above; the method has been used successfully in monsoonal and semi-arid climates.

When the soil is saturated to field capacity, evapotranspiration takes place at the potential rate. The quantity of water which remains when evapotranspiration and direct run-off are subtracted from precipitation are taken as percolation to the groundwater reservoir. If the soil is not at field capacity, then a soil moisture deficit (SMD) exists and no recharge is assumed to take place. Any infiltration (precipitation less run-off) that occurs reduces the SMD. Evapotranspiration from the soil continues at a lower rate, dependent on the magnitude of the SMD. If the SMD is greater than the root constant, then the actual evapotranspiration is usually taken as 10% of the potential value. Finally, at wilting point, evapotranspiration from the soil ceases. Unfortunately this method of calculating recharge contains at least one conceptual inaccuracy, as it has always been known that some recharge does occur in the presence of soil moisture deficit. Current practice in the United Kingdom is to allow an empirical fraction of effective precipitation in excess of a threshold value to enter the aquifer as direct recharge. These empirical values are normally taken as 15% and 5 mm respectively. Recent work has shown that recharge calculations using the soil moisture budget, performed on the basis of time intervals longer than 10 days, seriously underestimate recharge; ideally calculations should be on a daily basis.

8.2.2 The Use of Lysimeters

In concept lysimeters are very simple; they are devices used to isolate plots of land with their soil profiles so that these can be studied in detail. They were originally developed for crop water use studies but have also been adopted for groundwater recharge studies. To obtain accurate results from lysimetry, it is necessary to ensure that the conditions inside the lysimeter are idential in all respects to those outside.

In practice, lysimeters suffer from a number of severe drawbacks, of which the following are the most serious:

- they are usually relatively small in size so that root development may be restricted and edge effects are relatively large
- the soil and strata within a lysimeter are always to some extent affected by the construction of the lysimeters so that root development and water flow are no longer as natural.

- free drainage from a normal lysimeter means that the base boundary condition is not the same as that on adjacent land and therefore the conditions within the lysimeter may not be representative
- if an adequate surround planted with the same type of vegetation is not provided, a lysimeter may act as an 'oasis' so that conditions within do not correspond with those outside it
- the high cost and lack of mobility.

Nevertheless the lysimeter concept can be adopted to give significant results in recharge studies. The problems listed above can be minimised by careful site selection and design of the lysimeter so that a sufficiently large area (about 100 m^2) of natural soil and aquifer is isolated. Figure 8.7 shows a lysimeter adopted for recharge studies at Nottinghamshire in England. This system has been in use for several years and is considered to give a more accurate measure of recharge than the soil moisture budget approach described above. Its design could be easily adapated to, for example, a thin alluvial aquifer overlying basement.

8.2.3 Isotope Techniques

Natural Tritium Profiles

The presence of tritium in groundwater indicates recharge within the last 30 years. Infact the amount of tritium in rainfall has been related to the testing of thermonuclear devices in the atmosphere and in some years, tritium in rainfall has been particularly high; groundwater derived from rainfall in those years may sometimes be identified.

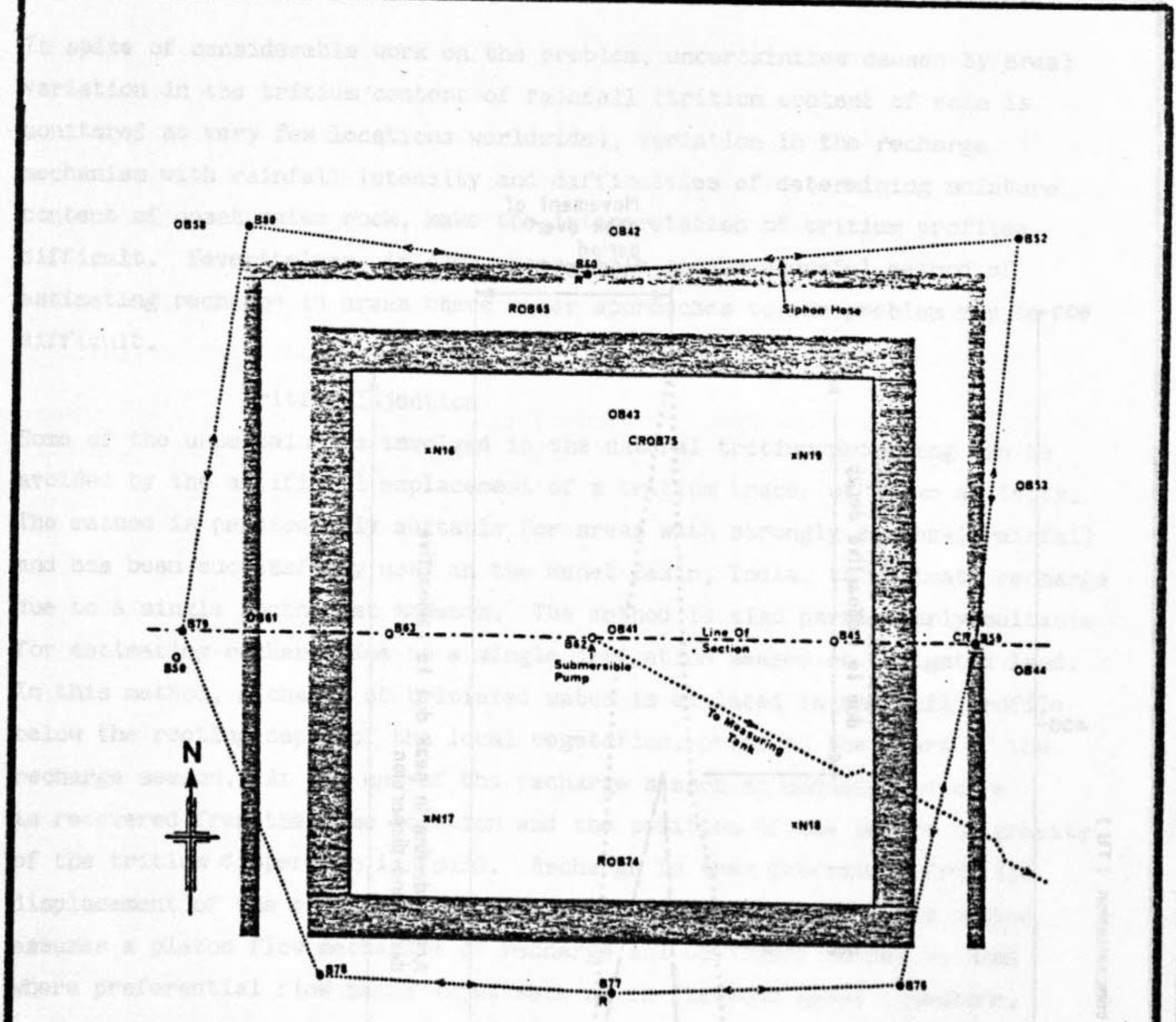
Where a groundwater body is overlain by a thick unsaturated zone, this may have a storage capacity equivalent to several years recharge. On the basis of the assumption that moisture is displaced progressively through the unsaturated zone by a piston flow mechanism, it should be possible to trace the substantial variations in tritium content of rainfall (and hence recharge) which have occurred over the past 20 years. The rate of movement of specific tritium peaks in the profile should be related to the recharge rate by a simple equation of the form:

$$\Delta R = \Delta Y \times S_m \qquad (5)$$

where ΔR = recharge rate
ΔY = rate of displacement
S_m = moisture content

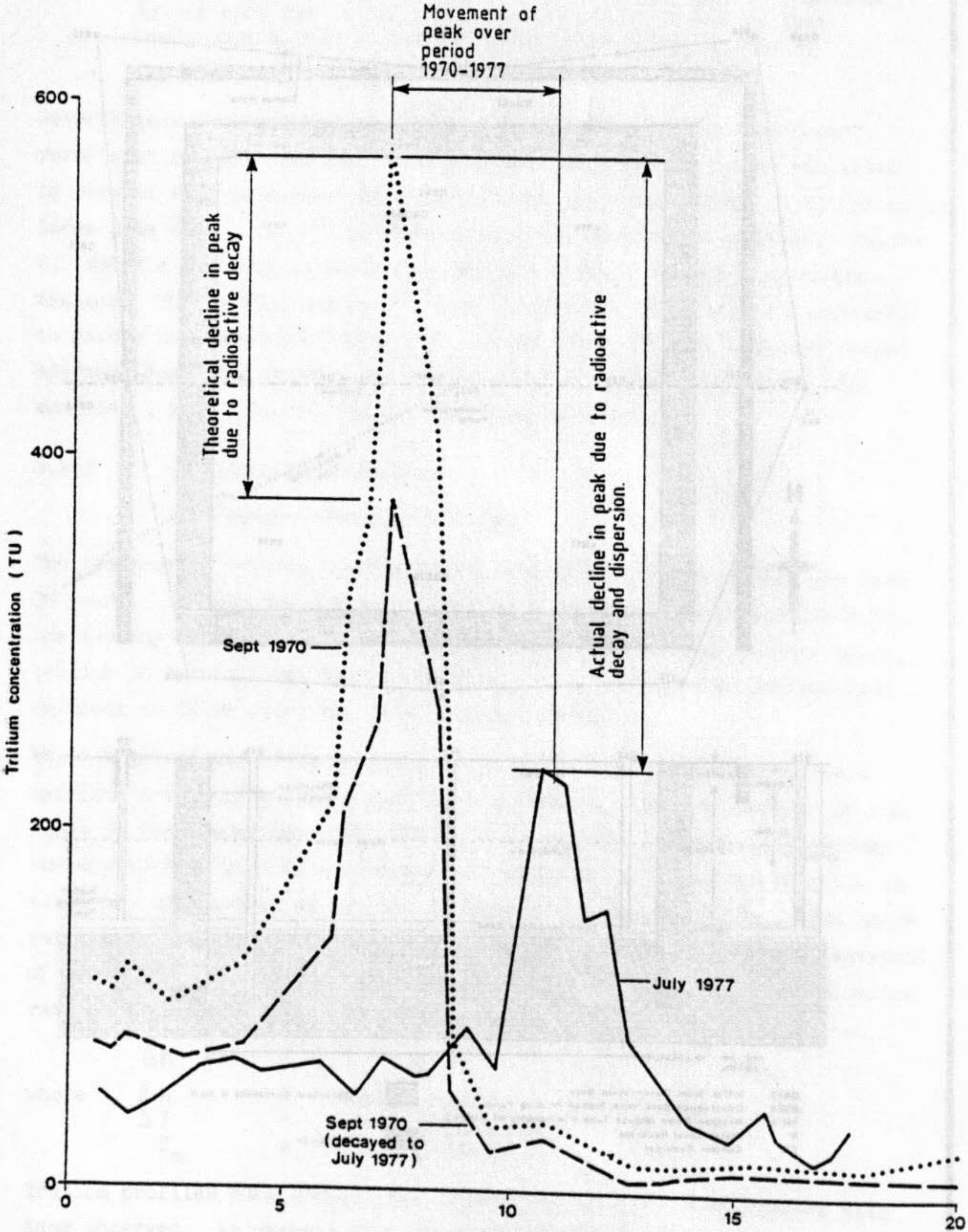
Tritium profiles have indeed been identified and their displacement with time observed. An example from the chalk of Dorset, United Kingdom, is illustrated (Figure 8.8).

FIGURE 8.7 LYSIMETER INSTALLATION IN A SANDSTONE AQUIFER



The water table in the aquifer surrounding the Lysimeter is equalised by the siphon hose. The water table within the Lysimeter is maintained at the same level as that outside by pumping from the central borehole to a measuring tank. Recharge = volume of water pumped ÷ area of Lysimeter.

FIGURE 8.8 VARIATION OF UNSATURATED ZONE TRITIUM PROFILES



Note : The peaks are caused by atmospheric testing of thermonuclear weapons during the period 1962-1964.

In spite of considerable work on the problem, uncertainties caused by areal variation in the tritium content of rainfall (tritium content of rain is monitored at very few locations worldwide), variation in the recharge mechanism with rainfall intensity and difficulties of determining moisture content of unsaturated rock, make the interpretation of tritium profiles difficult. Nevertheless, in some cases, this can be a useful method of estimating recharge in areas where other approaches to the problem may be difficult.

Tritium Injection

Some of the uncertainties involved in the natural tritium profiling can be avoided by the artificial emplacement of a tritium tracer of known activity. The method is particularly suitable for areas with strongly seasonal rainfall and has been successfully used in the Manet Basin, India, to estimate recharge due to a single south-east monsoon. The method is also particularly suitable for estimating recharge due to a single irrigation season on irrigated land. In this method, a charge of tritiated water is emplaced in the soil profile below the rooting depth of the local vegetation, prior to the start of the recharge season. At the end of the recharge season an undisturbed core is recovered from the same location and the position of the centre of gravity of the tritium dispersion is found. Recharge is then determined from the displacement of the original position of the tritiated slug. This method assumes a piston flow mechanism of recharge and obviously cannot be used where preferential flow paths exist such as in fissured karst limestone.

8.2.4 Recharge from Sources Other than Rainfall

Irrigation Recharge

Aquifer recharge resulting from irrigation is usually taken as the difference between water applied and crop consumptive use (crop evapotranspiration). With a centralised irrigation system, direct measurement of water applied is possible but more usually, it is necessary to estimate this by field observation of local irrigation practice coupled with an assessment of the area under each crop; the latter is often obtained by air photo interpretation. Crop water requirements under the prevailing conditions can usually be obtained from published literature. Where field drainage is installed, it is important to gauge this.

Leakage from Surface Water Bodies

Seepage to the groundwater reservoirs from rivers, lakes and canals may be significant but is often difficult to estimate. The direct method is to observe water losses from the surface body, compute evaporation, measure surface inflows and outflows and thus calculate the contribution to the groundwater system. However, this is often difficult to do and it often suffers from the usual source of inaccuracy of comparing two large quantities to find a small difference. It is sometimes advantageous to use a method of estimating seepage that does not involve the volume of the surface water body. If the surface water is directly connected to the watertable, then seepage can be computed by flow net analysis or even a direct calculation of subsurface flow using Darcy's Law and the Dupuit/Forchheimer assumptions.

If strong chemical or isotopic contrast between surface water and groundwater exists, a qualitative estimate of recharge from a surface body can be obtained by studying the mixing ratio using one or more of the hydrochemical graphical techniques.

Leakage from Other Aquifers

Recharge to an aquifer from underlying or overlying aquifers, separated from it by confining beds of restricted permeability, is described by the equation:

$$Q = A \; K_v \frac{\partial h}{\partial z} \qquad (6)$$

where Q = the groundwater inflow
A = area of flow
K_v = vertical permeability of the confining bed
h = hydraulic head
z = the vertical coordinate

The main problem in applying this equation to actual problems such as the multilayer alluvial aquifers in north-east Kelantan, lies in determining K_v. There are three possibilities; laboratory determinations on core samples, pumping tests using the leakance theory and the calibration of mathematical models. As with surface water leakage, hydrochemical and isotope techniques can sometimes provide a qualitative insight into the inter-aquifer leakage.

8.3 Indirect Methods of Recharge Assessment

8.3.1 Watertable Hydrograph Analysis

The methods of estimating groundwater recharge described above attempted to quantify the inflow into the aquifers directly or by computing losses from surface water. The watertable hydrograph method examines the behaviour of an unconfined or phreatic groundwater body under the influence of recharge and, from its behaviour, attempts to deduce the magnitude of this recharge. Water levels in unconfined aquifers often respond quickly to recharge, showing sharp peaks. In climatic environments with a well defined wet season, the watertable hydrographs may sometimes be used for estimating recharge. For short term fluctuations, where the gradient of the water surface can be ignored, recharge can be approximately computed merely from the rise of the watertable in the rainy season. A hydrograph on which such a calculation could be based is shown (Figure 8.9a). In such cases it can be taken that:

$$R = \Delta h \times S_y \times A \qquad (7)$$

where S_y = specific yield of the strata in the watertable zone
and A = area of the recharged basin

Equation (7) tends to underestimate recharge as it ignores groundwater drainage during the time when the watertable is rising; it also ignores any recharge which occurs during watertable recession. A further problem is that the specific yield of the material of the zone in which the water level oscillates is seldom known with any degree of accuracy. Nevertheless, this method often provides a useful check to ensure that the quantity of recharge, calculated by other methods, is sensible.

These complications can often be avoided by basing the recharge estimate on the recession of the hydrograph; in some cases the recession appears to be linear with time (Figure 8.9b). Assuming that this can be interpreted as a constant rate of groundwater drainage, independent of the actual elevation of the watertable, this is extrapolated to a full year and the annual recharge is again estimated using the expression:

$$R = \Delta H \times Sy \times A \qquad (8)$$

where H is as defined by Figure 8.9 and the other symbols are as before.

8.3.2 Throughflow

Recharge to an aquifer, which can be assumed to be in a long term equilibrium, can sometimes be estimated by calculating groundwater throughflow at some convenient place, using the Darcy equation. Ideally this method applies to confined aquifers only, such as is shown (Figure 8.10). The flow system is visualised as originating with infiltration on the outcrop and subsequent passage through an aquifer, confined at top and base by impermeable beds, to the discharge zone, which may involve leakage to upper or lower beds, or discharge to surface water bodies, or evaporation and evapotranspiration. The idea is that exploiting the groundwater by boreholes somewhere near the recharge zone would merely

FIGURE 8.9 TYPICAL WATER TABLE HYDROGRAPHS

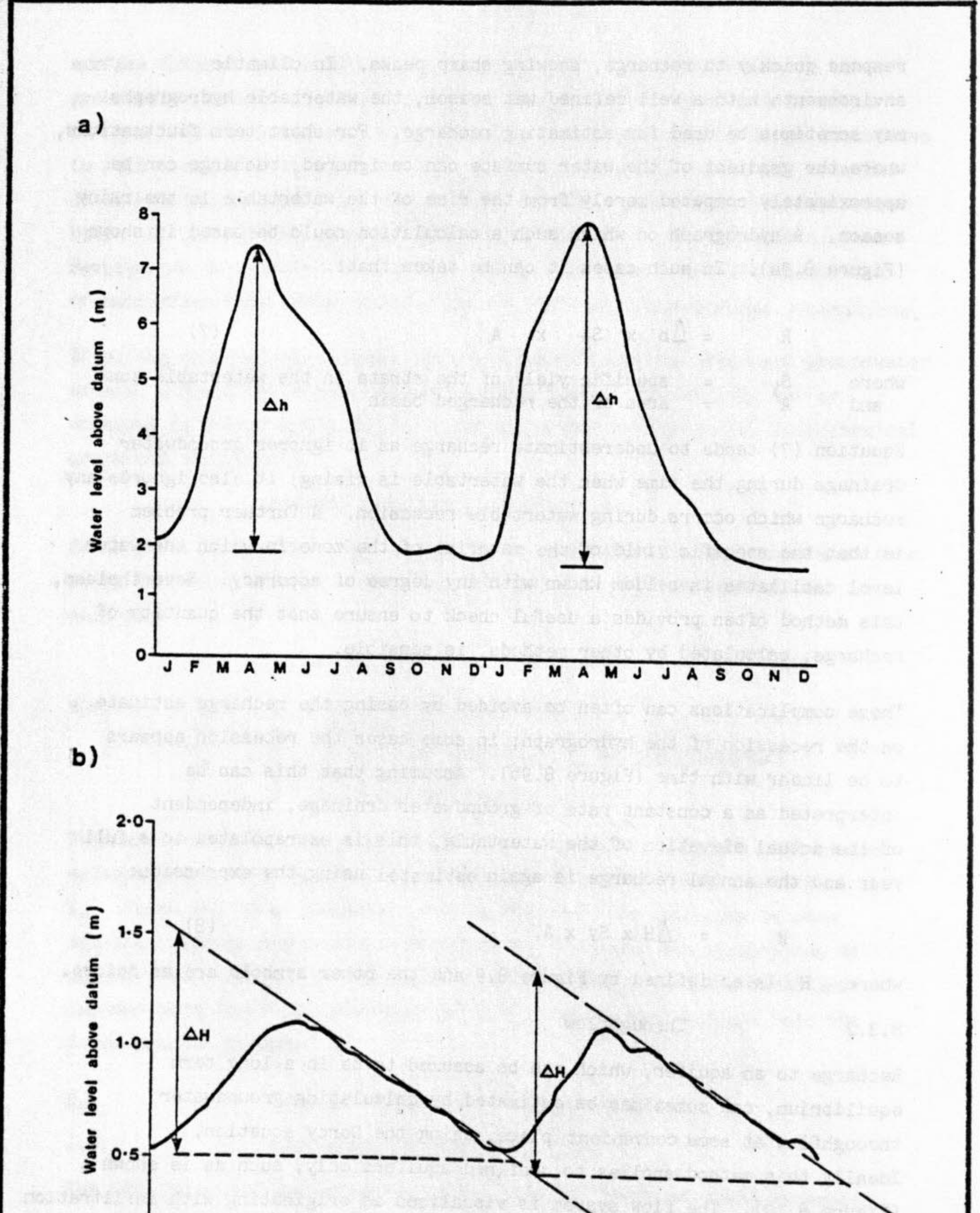
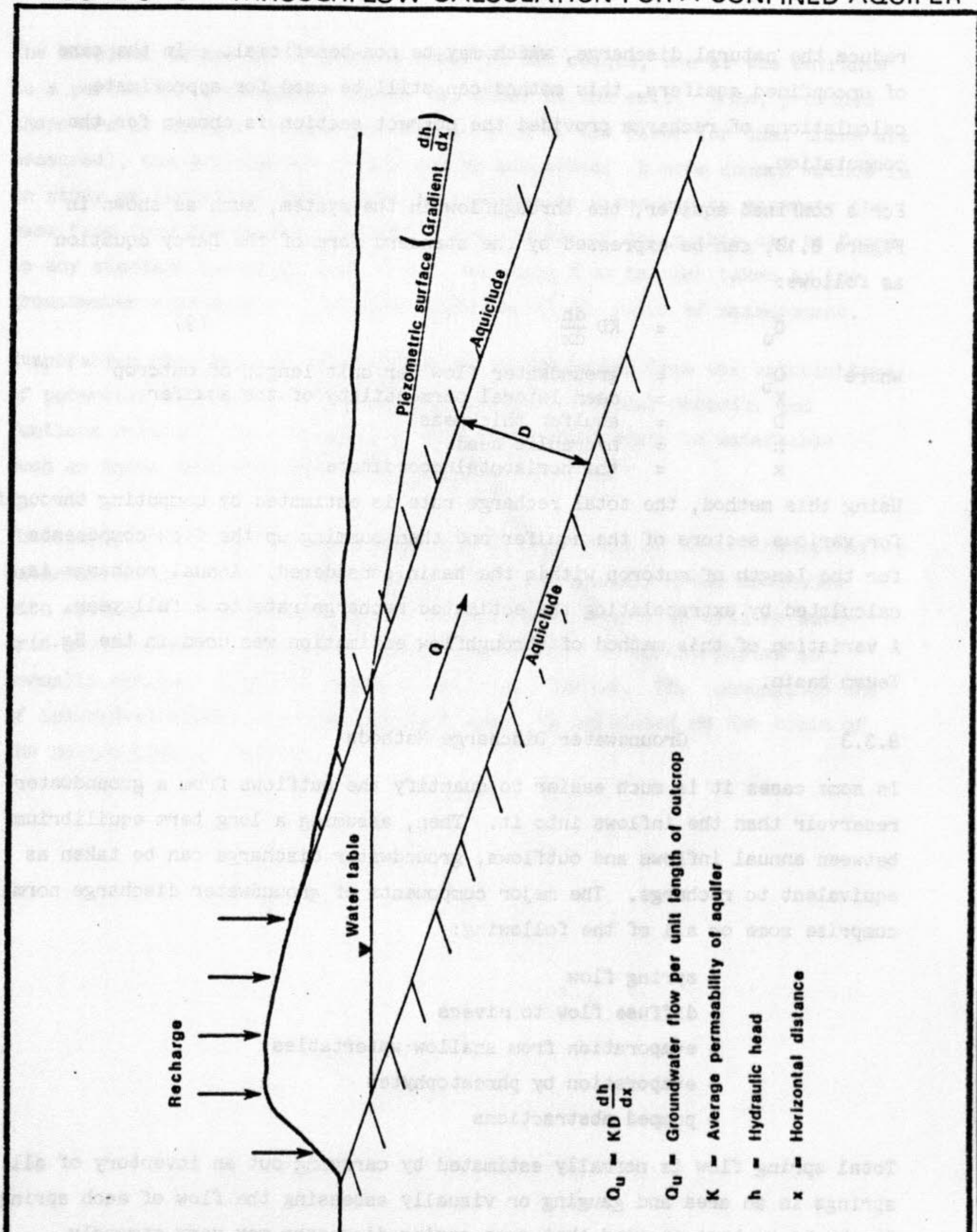


FIGURE 8. 10 THROUGHFLOW CALCULATION FOR A CONFINED AQUIFER



reduce the natural discharge, which may be non-beneficial. In the case of unconfined aquifers, this method can still be used for approximate calculations of recharge provided the correct section is chosen for the computation.

For a confined aquifer, the throughflow in the system, such as shown in Figure 8.10, can be expressed by the standard form of the Darcy equation as follows:

$$Q_u = KD\,\frac{dh}{dx} \qquad (9)$$

where Q_u = groundwater flow per unit length of outcrop
K = mean lateral permeability of the aquifer
D = aquifer thickness
h = hydraulic head
x = the horizontal coordinate

Using this method, the total recharge rate is estimated by computing throughflow for various sectors of the aquifer and then summing up the flow components for the length of outcrop within the basin considered. Annual recharge is calculated by extrapolating the estimated recharge rate to a full year. A variation of this method of throughflow estimation was used in the Sg. Tekam basin.

8.3.3 Groundwater Discharge Methods

In some cases it is much easier to quantify the outflows from a groundwater reservoir than the inflows into it. Then, assuming a long term equilibrium between annual inflows and outflows, groundwater discharge can be taken as equivalent to recharge. The major components of groundwater discharge normally comprise some or all of the following:

- spring flow
- diffuse flow to rivers
- evaporation from shallow watertables
- evaporation by phreatophytes
- pumped abstractions

Total spring flow is normally estimated by carrying out an inventory of all springs in an area and gauging or visually assessing the flow of each spring. It should be kept in mind that some spring discharge may vary strongly seasonally; springs may also exhibit flow variation from year to year. Best estimates are added up to give an annual total for a groundwater basin or a project area.

Diffuse groundwater discharge to rivers is normally estimated from the records of river gauging, particularly those of the automatic recorders.

The simplest method is to gauge a river at two points, one at the entrance to a particular groundwater basin, the other at the exit. Then, provided there are no surface inflows in that stretch of the river (or that these are measured), the groundwater inflow can be estimated. A more common method is to study an individual hydrograph of a stream or river and to separate the base flow from the surface run-off. The methods of doing this can be found in any standard hydrology text book. The base flow is then taken as the groundwater discharge to the river upstream of the point of measurement.

Evaporation from shallow watertables can be estimated from the calculations of potential evaporation calculated from meteorological records, and funtions relating the intensity of evaporation and depth to watertable such as those developed by Gardner and Fireman (1958).

Phreatophytes are plants which take the whole or part of their transpirative water use from the watertable. Such plants are usually trees or bushes with extensive root systems and are particularly common in arid or semi-arid environments. The distribution and density of phreatophytes is normally obtained from the examination of air photos. The consumptive use of individual plants, or that per unit area, is estimated on the basis of the meteorological records.

BIBLIOGRAPHY

General Hydrogeology and Drilling

Anderson, K.E., 1973, Water Well Handbook: Missouri Water Well & Pump Contractors Assn., Inc.

Aramco., Standard Operating Procedure of Sample Description: Arabian American Oil Company.

Bouwer, H., 1978, Groundwater Hydrology: New York, McGraw-Hill.

Campbell, M.D., and Lehr, J.H., 1974, Water Well Technology: New York, McGraw-Hill.

Gobbett, D.J., and Hutchison, C.S., ed. for the Geological Society of Malaysia, Geology of the Malay Peninsular: Wiley Interscience.

Johnson Division, UoP Inc., 1975, Groundwater and Wells: St. Paul. Minn.

Pettijohn, F.J., Sedimentary Rocks: New York, Harper.

Todd, D.K., 1960, Groundwater Hydrology: New York, John Wiley.

Geophysics, Resistivity

Flathe, H., 1963, Five Layer Master Curves for the Hydrogeological Interpretation of Geoelectric Resistivity Measurements above a Two-storey Aquifer; Geophys, Prosp., v. 11, p. 471-508.

Ghosh, D.P., 1971, Inverse Filter Coefficient for the Computation of Apparent Resistivity Standard Curves for a Horizontally Stratified Earth: Geophys. Prosp., v. 19, no. 4, p. 769-775.

Orellana, Ernesto, and Mooney, H.M., 1966, Master Tables and Curves for Vertical Electrical Sounding over Layered Structures: Madrid Interciecia.

Tedford, W.M., Geldart, L.P., Sheriff, R.E. and Keys D.A., 1976, Applied Geophysics: Cambridge University Press.

Van Nostrand, R.C., and Cook, K.L., 1966, Interpretation of Resistivity Data: U.S. Geol. Survey Prof. Paper 499.

Zohdy, A.A.R., 1974, Application of Surface Geophysics to Groundwater Investigations, Electrical Methods: Chapter D1, Techniques of Water Resources Investigations of the United States Geological Survey.

Geophysical Borehole Logging

Schlumberger Limited, Schlumberger Log Interpretation Vol. 1 - Principles and Vol. 2 - Log Interpretation Charts.

Scott Keys, W., and MacCary, L.M., 1971, Application of Borehole Geophysics to Water Resources Investigations: Chapter E1, Techniques of Water Resources Investigations of the United States Geological Survey.

Widco Water Finder Guide, Gearhart - Owen Industries, Inc.

Aquifer and Bore Test Analysis

Bennett, G.D., 1976, Introduction to Groundwater Hydraulics: Chapter B2, Techniques of Water Resources Investigations of the United States Geological Survey.

Kruseman, G.P., and deRidder, N.A., 1970, Analysis and Evaluation of Pumping Test Data: ILRI, Wageningen, Netherland.

Recharge, Groundwater Chemistry

Gardner, W.R. & Fireman, M. 1958, Laboratory Studies of Evaporation from soil columns in the presence of a water table. Soil Science, Vol. 85, No.5.

Hem, J., 2nd edn. Study and Interpretation of the Chemical Characteristics of Natural Water: US Geological Survey Water Supply Paper No. 1473.

Penman, H.L. 1948, Natural Evaporation from Open Water, Bare Soil and Grass. Proc. Roy. Soc., London. Ser. A, 193.

Rushton, K.R. & Ward C, 1979, The Estimation of Groundwater Recharge. Jnl. of Hydrol., Vol. 41, No. 3/4.

Wood, W.W., 1976, Guidelines for Collection and Field Analysis of Groundwater Samples for Selected Unstable Constituents: Chapter D2, Techniques of Water Resources Investigations of the United States Geological Survey.