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Material Presented at Groundwater School, 1965

Part I.

(Hydrogeology, Geophysics, Hydraulics, Pumping Tests)

by

D.A. White

MATERIAL PRESENTED AT GROUNDWATER SCHOOL, ADELAIDE, 1965

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INTRODUCTION

The purpose of this Record is to document the written material which was issued at the Groundwater School held in Adelaide from 29th March to 9th April, 1965. In Records 1965/85 I have dealt with the organization, syllabus, and general scope of the School and it will not be necessary to repeat these aspects here.

The material will be issued in two parts:

Part 1: Hydrogeology, Geophysics, Hydraulics, and Pumping Tests.

Part 2: Drilling, Bore Construction, Chemistry of Groundwater, Utilization.

No attempt was made to edit the material which was written by the lecturer in most cases as lecture notes and not for publication.

HYDROGEOLOGY

.by

E.P. O'Driscoll, Chief Hydrogeologist, Geological Survey of W.A.

In these lectures I propose to deal with the fundamentals of of hydrogeology, and I'm going to start from scratch, and assume the listeners know nothing about it.

Origin of groundwater.

To geologists, groundwater is the water that occurs freely within the zone of saturation. We are not concerned with soil moisture above the water-table.

Groundwater may have any one of three origins. The most uncommon is magmatic or juvenile water, and I do not know of any Australian occurrences of this, although it has been recorded elsewhere, being recognised by its deficiency in chlorine, which is always present in other types of groundwater.

Ref. Simplon Tunnel.

The next type is connate water, which is really fossil meteoric water. When sediments are laid down on the sea floor, or in a lake or swamp, some of the water will be trapped in the spaces between the grains, and held there until it is expelled by the action of outside agents. These could be, for example, the subsequent elevation of the rocks above sea level, allowing the water to flow sideways under gravity and escape. Or the rocks might be folded or tilted in a manner that exposes the elevated edge to intake from rain or surface runoff.

In itself, ofcourse, the availability of such intake doesn't necessarily mean replacement of the connate water, or water of deposition, already within the rocks. There also has to be an outlet somewhere, before actual replacement can take place. Even then, the water will move as directly as it can towards the outlet, and only partial replacement may occur. This matter will be dealt with more fully in the section on groundwater movement.

Connate waters occur quite commonly in Australia.

Examples: North west Murray Basin.
G. A. B. shallow saline aquifers.

Waters of this type almost invariably are too saline for general use, which may be a good thing, as otherwise they would be developed, and probably fail because of pumping.

Usually, however, groundwater is meteoric in origin; that is, it owes its existence to rain falling on the earth's surface, and either directly penetrating underground where it falls, or causing surface runoff which works its way downwards into the rocks at more distant points.

Natural Cycle.

Diagram (Precipitation. Runoff.
(Evapotranspiration. Downward percolation.

The proportion reaching the water-table varies enormously. Note the variable which is vegetative cover. This can be destroyed by man, and the cycle upset - (Siltation of dams). Note that the level of the water-table rises under the hills.

Environmental Factors.

At this point let us consider the factors of environment that affect groundwater occurrence. They are

(i) Rainfall.
(ii) Topography.
(iii) Vegetative cover, which includes evapotranspiration.
(iv) Rock type.
(v) Rock structure.

Consideration of the natural cycle shows us that rain is dissipated in three ways; by downward percolation, surface runoff, and evapotranspiration; and it is obvious that the influence of the various factors overlaps. For example, the amount of downward percolation from rain will be affected by rock type; rainfall may be affected by topography, which in turn is related to to rock structure, and so on.

The importance of this observation really lies in the circumstance that groundwater may occur in two separate areas which are similar in all respects except for one variable, and this variation is one factor which can completely alter the groundwater occurrence. It becomes the dominant factor.

Example: Mt. Lofty. 45"-50" rainfall, very low salinity.
50 miles north, 20"-25", salinity much higher.

(i) Rainfall. So around Mt. Lofty, where the rock types vary but the water quality is invariably very good, the high rainfall is dominant factor.

However, low rainfall does not necessarily mean that all the groundwaters are of poor quality. For instance, domestic quality water has been obtained in certain arid areas of Central Australia, where the annual precipitation is 10 inches or less. One such area is in the vicinity of the Desert Meteorological Station in the Rawlinson Ranges. The reason is that where other conditions are favourable, surface runoff may occur even in very low rainfall areas, providing enough local downward percolation to maintain small areas of good quality groundwater, which elsewhere may be saline, or altogether lacking.

On the other hand, even a relatively high annual rainfall does not ensure the occurrence of substantial volumes of good quality groundwater. The type of precipitation is important because this affects the incidence of surface runoff, and therefore the local intake either from streams or as sheet runoff from the hill slopes.

Furthermore, the rock type and the topography also exert an influence. Consider the case of a desert heath area such as south-east of Lake Alexandrina, where a deep sand profile is combined with low topographic relief. Recent work showed that even with a rainfall in excess of 20 in., the rain water penetrated vertically downwards into the sand no farther than about 16 ft., at which depth it had all been dissipated in wetting the soil profile through which it passed. In such an area, topography is important, because where concentrations of surface runoff occur, the local rainfall is augmented and the depth of penetration greatly increased.

This can result in lenses of good water floating on the more saline water beneath, and an understanding of their origin is important. If such a lens results from localized concentration of surface runoff under natural vegetation conditions and the vegetation is later replaced by dense permanent pastures, runoff might no longer occur, and the good water lens could be gradually exhausted by pumping for stock use.

One may wonder at there being runoff from sand dunes. The simple fact is that a dry sand is far from being as permeable as one imagines, as a simple experiment with the garden hose will show. Ex.: Perth locality.

(ii) Topography. Topographic relief is important for several reasons. In the first place it affects rainfall, which is highest on the mountain ranges, with rain shadows on the lee slopes. This in turn controls the vegetative cover, and so, less directly, the surface runoff and evapotranspiration.

Steep slopes shed rainwater runoff quickly, encouraging not only sheet erosion but the dumping of rock debris as pediments and alluvial fans, which may be local sources of water. Ex.: Orroroo in the Adelaide Hills.

Also these alluvials may act as intakes (much like a sponge) in absorbing surface runoff and passing it downwards to deeper aquifers. Runoff from an extensive hilly catchment may deposit valley alluvials, (e.g. Hunter R. system) and will keep the alluvials recharged, so that large supplies are recoverable.

Hills themselves shed water, and in a dissected range this runoff water is concentrated in the valleys, where downward percolation may be greatly increased. This doesn't mean that larger supplies will necessarily be available, but it does mean more frequent replenishment, and usually lower salinity than the surrounding rocks. Even in a valley where the summer river flow becomes quite saline, the groundwater can be expected to be much better in quality.

Ex. Wakefield R.

Blackwood R. with saline first freshet in winter.
Where streams are intermittent, with sudden floods and long dry intervening periods, replenishment conditions are usually good, but may not be reliable enough for large town water supplies.

Ex. Turner and Yuill Rivers (Port Headland)

Rawlinson Ra.

In slate country, one adverse effect of steep slopes may be the deposition of an impermeable clay blanket in the valleys. In such cases, bore sites placed on the toe of the slopes slightly above the valley floor may give best results.

In dense metasedimentary and igneous rock areas, steep slopes are a disadvantage, as the weathered rock mantle is stripped away as it forms, and usable groundwater may be confined to gentle slopes about half way down.

Ex. Toodyay area where creeks are saline, but wells on the slopes may give fomestic water.

Esperance area.

So far, the emphasis has been on the role played by hills in promoting runoff which contributes to the groundwater. However, the existence of hills and the occurrence of such runoff in arid climates doesn't necessarily provide usable water.

Ex. Mukinbudin shallow bedrock area.

Wyalkatchem, fresh to saline in 5 chains.

A difficulty that faced early workers was that flat terrain did not necessarily mean no local groundwater.

Ex. Gregory, G.A.B. Wade, Murray Basin.

Nowadays so much drilling has usually been done that subsurface conditions can be worked out. This is not always so, however.

Ex. Badgingarra. Arrino, with clay bed, and faults, and alternating Perm./L. Cret.

Well-developed, steep-flanked valleys not only indicate considerable loss by surface runoff, but the valleys may cut down to the water-table, in which case the streams will act as drains into which the groundwater will flow by means of springs.

Ex. Murray R., S.A.

Where, as in the Murray Basin, the original saline connate water can be replaced only if there is an outlet through which it can escape, the existence of such a drain is of great importance. Without it, landholders would find their properties situated in saline groundwater areas, instead of underlain by good quality water which has been able to obtain ingress from distant elevated intakes.

That streams may act as groundwater drains is probably a more common phenomenon than generally realised, because often the action is only intermittent, occurring at times of low streamflow and being reversed to one of recharge during floods. In a few cases, they seem to have little effect, one way or the other, on the surrounding groundwaters.

Ex. Gascoyne R. Comment on barrage, which was not effective because of slow movements and small gradient.

Flat terrains, which often occur in semi-arid regions, encourage the development of salt pans. Surface runoff is sporadic, often torrential, and spreads out over the lower areas only to evaporate and leave behind a residue of salt. Groundwater under the flats themselves is often very salt, (c.f. Bon Bon Stn.) but the contaminating effect of the salt is sometimes quite localized. The pans generally have a clayey floor that is at slightly lower level than the water table round about.

Ex. Yorke Penin. W.A. calcretes.

(iii) Vegetative Cover. Under this heading we can include evaporation and transpiration. Vegetation, while its nature may vary with underlying rock type, depends on climatic conditions for its abundance, and these conditions may in turn be affected by topography. It is of importance because of its effect on surface runoff and also on losses of moisture back to the atmosphere.

Water running quickly off the land surface is lost as a source of local groundwater recharge. True, it may enter the soil or alluvium in a valley downslope and there become a source of groundwater, but it is lost to the area on which it fell as rain. This cuts both ways, of course, because in semi-arid areas where bare hill surfaces shed all their rain, the water may accumulate in the wash at the foot of the hills, and provide a limited source of very good water.

Ex. Victoria Hut.

It is possible that with a heavy vegetative cover, usable water might still have occurred, but vegetation would stop most of the runoff except in very violent storms, and hold the moisture in the soil upslope, possibly to be lost by transpiration etc. in such a semi-arid area.

Where vegetation is thick, the rainwater is held in the soil through which it moves only slowly, and has ample opportunity to soak downwards into the underlying rocks and become groundwater. This not only improves the prospects of obtaining water by drilling, but decreases salinity by promoting movement and the flushing away of soluble salts. On the debit side, transpiration losses increase, but rarely if ever to the extent that downward percolation is exceeded, although the water table may decline. Ex. Booboorowie.

Evidences of the beneficial effect of vegetative cover on the groundwater are found all over Australia, a typical example being the Avon-Mortlock valley (Swan River, W. A.) Less than a century ago these streams were fresh, and wells along their valleys yielded very good water. Some still do, but clearing of the land has increased runoff and evaporation losses to the extent that the rivers are salt, and so are many of the wells. In some places the salt has encroached on adjacent farm lands, killing the vegetation and rendering the land useless for cultivation.

Ex. Sisal hemp areas, N. Africa.

Transpiration losses will affect shallow groundwaters.

Ex. Bridgetown soak; saline well in granite;

Hydrogeologists are sometimes asked their opinion as to whether certain trees or plants indicate the presence of water. Some trees, e.g. W. A paperbark, may grow only in areas where shallow groundwater occurs, but in general tree roots can live on soil moisture alone. Extensive growths along watercourses etc. merely indicate more frequent replenishment and perhaps slightly deeper percolation than elsewhere.
Ex. Gum trees Silverton (Broken Hill).

(iv) Rock Type. Geologists classify rocks under three general headings according to their origin, as igneous, sedimentary and metamorphic. All three families can be further subdivided into different types, some of which are potentially good and others very unpromising as aquifers.

(a) Igneous Rocks. The common plutonic type, granite, is widespread, and its potential as an aquifer varies a great deal. Granite is dense and crystalline, and completely impervious in the hand specimen. However, it weathers readily into quartz and clay, the products having different grain sizes and physical properties, so that the weathered products are readily sorted by agents of erosion. This means that in granite country, sediment and alluvium may have a large proportion of sand, making it very permeable. Although they cannot safely be sited on high ground because the hard rock is at shallow depth, wells and bores are often quite successful in the valley floors, or near the foot of a long slope. Also, the hill crests are often bare enough to shed water like a roof into the clastic material downslope.

Ex. Olary.

Southern N. S. W. - War Service Land Settlement Scheme.

Hd. Rounsevell, S. A

The development of a weathered profile, however, and the importance or otherwise of the mechanical effect of sorting and redeposition by agents of erosion, are of course dependent on several factors.

Ex. W. A. central wheatbelt, with arid climate, flat terrain and thin soil cover.

W. A. Darling Ra. with high rainfall, heavy timber, active streams, very little erosion.

Another thing to consider in granite country is the length of the slope. In very permeable material, unless frequent recharge occurs the water may escape downslope.

Ex. Cooma area.

Till recently, granites have rarely been drilled because of their hardness. Hammer drills have altered this.

Ex. Exfoliation joints in rail cuttings.

The above comments can also apply to other dense igneous rocks such as porphyry.

Ex. Yass "porphyry".

Very occasionally caverns may occur, possibly as a result of mineral

Ex. Wayatinah dolerite, 60 x 30 x 30 r...

However, they are quite unpredictable, and cannot seriously be considered from a hydrological viewpoint. More important, particularly in hypabyssal rocks, is the development of jointing, and decomposition along the margins of dykes due to shearing.

Ex. Serpentine dam.

Northampton diorite dykes. Explain salinity variation between granulite, cherty quartzite, diorite. Increase in Pb and Cu under pumping, in bores near dykes.

As aqifers, volcanic rocks vary, being very good when they are eruptive and contain tuffs; poor when of the quiet fissure type.

Ex. Gragin, (North-east of New South Wales)
Coolah.
Antrim Plateau.

My personal advice to the young field geologist is to treat volcanic rocks with caution when selecting bore sites for landholders. Quite apart from doubtful permeability, the rocks can be very expensive to drill. Ex. Boyanup, (South-west of W. A.). We must also remember that basic rocks like basalt weather to a clay which can effectively blanket the underlying rocks and prevent intake. Ex. Delungra, granite sand.

Sometimes the soil cracks badly. Ex. Blacksoil plains.

(b) Metamorphic Rocks can roughly be subdivided for our purposes into quartzites and marbles; schists and amphibolites; and migmatites and Archaean gneisses.

Quartzites are almost always good waterbearing beds, except when heavily re-silicified.

Ex. Aldgate S. S, Adelaide area.
Pentecost S.S., Kimberleys.
Grant Ra. S.S.

The best type, in my experience, is the fairly friable, sugary kind with an outcrop thickness of 100 feet or more. An ideal boresite would be located near a creek, to reach the top of the sandstone at about 100 feet depth, the bed having a dip of 30°-50°. Limit of depth for large supplies is about 350 feet. Quartzite usually yields the lowest salinity water because of its lack of soluble minerals; and large supplies because it is a competent rock which fractures readily when folded. The joints do not become clogged with clayey decomposition products, and the cracks stay open, allowing ready ingress and movement of groundwater. Ex. Manoora.

Marble, which has similar physical properties but also readily becomes cavernous as a result of solution along the joints, is often capable of giving very large supplies. Also it is easy to drill. However, it can have its disadvantages. Ex. Reef types not extensive, e.g. Brukunga, Angaston, Second Valley.

Also the water is high in carbonate, a disadvantage for industrial use. Dolomitic slates can be very difficult for the geologist, because occasional bores quite unpredictably produce large supplies, but the bulk are very poor.

Ex. Lenswood area. Myponga Dam.

Slates and phyllites, usually associated in the field, can generally be relied on for stock supplies when drilled. In some areas the harder and more competent slates quite consistently yield limited irrigation supplies when first drilled, but this has its dangers, as constant pumping at 2,000 g.p.h. or so, as for market gardens, usually dewaters the rocks.

Ex. Mt. Barker, S. A. Montacute.

Comment on max. and min. depths. for stock. and irrigation.

The schist-amphibolite group represents the highly metamorphosed and very micaceous rocks, and is very variable. As a general rule the bore yields are small, because although intensely sheared the rocks do not have an open joint system. The abundant ferro-mag minerals readily decompose and yield soluble salts to the groundwater, which tends to be saline. Where the schists are "knotted", in my experience they are such poor aquifers that it is rarely justifiable to recommend drilling. Also, intense metamorphism of pre-existing sediments of varying type is often accompanied by small localized "granitisation", which adds to the difficulties of drilling.

Ex. Kanmantoo Gp. (Mt. Pleasant, S. A.)
W. A. Greenstones.

As a general summary, I have observed that although the alluvial products of an eroded granite commonly may be expected to yield very good quality groundwater, the weathered material in situ frequently contains rather more saline water. In comparable rainfall areas, on the other hand, basic volcanic rocks usually provide low-salinity water. There may be physical reasons for this, such as differences in permeability, and the matter might repay a little research. Unweathered but jointed acid igneous rocks, on the other hand, though yielding small supplies, often produce very good quality water, again perhaps for physical reasons such as elevation, and the relative freedom of groundwater movement provided by the joints.

Among metamorphosed or partly altered sediments the rock type itself exerts marked control over both salinity and supply. In strongly folded and well jointed Proterozoic rocks (e.g. Adelaide system) quartzites and marbles invariably provide quite large supplies having a low salinity, reflecting the open nature of the joint system found in competent rocks, and the relative lack of soluble sodic minerals. Rocks of the slatey and phyllitic type have tighter joints, which impede the movement of water through them, and also provide more soluble salts and consequently more saline groundwater. Schists and greywackes (e.g. Kanmantoo Gp.) appear to decompose fairly readily and give saline waters, with the result that even in a restricted area, bore water supplies may vary from one extreme to the other, depending on rock type.

Archaean gneisses, and most of the migmatites I have seen, are on the whole the worst types of rock we meet. Drilling conditions are difficult, supplies are small and often non-existent, and the water is usually very saline. As is often the case with strong metamorphism, the joint system may be very poorly developed. Ex. Boston Island, S. A.

It sometimes happens, of course, that rocks mapped on a regional scale as being Archaean (undiff.) vary in character over short distances. Ex. Toodyay Rly. cutting, 9 rock types. If they can be mapped from outcrops, some individual beds may repay drilling.

Before leaving this section there are one or two special cases of interest which can be mentioned. In parts of W. A. such as Hyden, Wilgoyne, Kalgoorlie, as far south as Esperance, the basement is Archaean, but in some places even drinking water occurs, where runoff is being shed from outcrops. Ex. Hyden
Esperance
Upslope, good water, not permanent.
Downslope, saline, permanent.
Wilgoyne - Old Hstd Well.

As a general rule in these 10 inch rainfall areas, if bedrock is less than 80-100 feet deep, no water occurs. If the bore is drilled more than about ½ mile from the hillslope, the water is saline.

Ex. Breakways.
Ex. Tertiary remnants on ancient peneplain - cf Esperance of calcretes.
Ex. Quartz veins, Darling Ra.

(c) Sedimentary Rocks. Although by force of circumstances many of us work a great deal among metamorphic and igneous rocks, the most important aquifers in Australia are of course the sediments. They are widespread, often contain pressure water of good quality, and in many places are being developed for industrial and domestic use. Unfortunately the users seldom

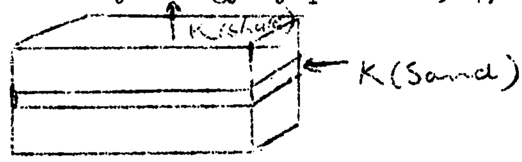
realize that there is a finite limit to the volume of water that can safely be extracted from any aquifer system, and increasing demands continue to be met by expansion of the supply work, till the stage is reached where output exceeds replenishments and the water is being mined.
Ex. Wicherina (Geraldten) Uley-Wanilla.

Water levels will always decline under pumping, which is often seasonal, and the decline in itself need not cause concern, provided that the level recovers each winter. Gradual and continuing decline indicates overpumping, and water levels should be measured at regular intervals throughout the whole year. Ex. Adelaide Plains. Tomago Sands, Long Id, (New York). Need for observation bores.
Watch for contamination, either by trade wastes and domestic effluent, or salt water intrusion.

Sediments can perhaps best be dealt with here according to their environment, which is of two kinds; that in which they were laid down, and that in which they now occur because of subsequent structural movements and the passage of time. Hydrological conditions in the older and well-compacted rocks resemble those in metamorphic rocks; and my subsequent comments refer to the less well-compacted types, covering a time range of about Permian to Recent.

Conditions of deposition are important because they govern the physical nature of sediments as regards grain-size, areal extent and thickness, and the character of the beds occurring in an alternating sequence. The geologist must learn to assess the past history of an area before he applies himself to developing it as a source of water.
Ex. Swamp deposits, individually small, may recur elsewhere.
"Back-dune" swamps (paralic) suggest proximity to sea-coast. May be repeated on a retreating shoreline.
Alternating marine and freshwater beds may mean salinity variations. Ettrick Marl and Buccleuch Beds.
Coralline limestones may end suddenly, because of their reef growth, and effect of muddy water -- Myponga.
Continental type beds may contain gypsum aggregations, with likelihood of local high salinities.

Permian marine beds contain boulders - Pinjarra. There are many such examples, and the point we should remember is never to skip the field investigations or the exploratory drilling, because very often unexpected things turn up.
Ex. Arrowsmith R, (North of Perth)
The original conditions of deposition may still - though not always - affect the groundwater salinity. Connate water is known in several places e.g. G. A. B.: Murray; Rottnest Id. Its preservation in the rocks will depend on other factors such as structure, permeability, intake. However, it is probable that few aquifers now contain precisely the same salinity water as that in which they were laid down. For one thing, this presupposes their complete enclosure, above and below, in impermeable material, which in nature is rare.
Ex. K. Hoeger. (Paper delivered at Canberra Hydrology Symposium 1964)



Ex. Hd. Shaugh area.
What generally happens under hydrodynamic conditions is the partial or imperfect replacement of water.
Ex. Airport bore, Adelaide.
Mayall's Bore, Derby.
It is important to realize that very few rock systems can be in complete hydrestatic equilibrium and also that water transfer in some directions, or through some parts of the system, may be very slow indeed, compared with others.

(d) Sand Dune Supplies. Explanation of how they form, and the limitations of supply. Large supplies in some places where dunes are extensive.

Ex. Port Gregory. (North of Geraldton)
Lancelin (North of Perth)
Onslow.
Miami Beach, south of Rockingham.

(V) Rock Structures.

Under this heading we may include folds, joints, faults, bedding and weathering.

Folding with or without marginal faulting is the cause of our numerous pressure water basins, all that has happened in some cases being that one edge of a permeable sequence has been tilted upwards and exposed to intake from and atmospheric source. Most of our larger basins have more than one aquifer, and these may or may not have a common intake.

Ex. Murray Basin - Knight Sands; Gambier L/s.
Perth Basin - Mandurah-Carnarvon
Canning (inland intake not obvious)
Adelaide (ref. K.R. Miles)
Fitzroy Basin.

Explain why different aquifers have different piezometric levels. Folding is important to the geologist siting bores in hard sediments and metamorphics.

(a) Faulting.

Ex. Northern Perth Basin.
Adelaide Basin.
Pirie-Torrens.

Faulting may dislocate an aquifer and interrupt groundwater movement. This will be reflected in the isohalsines and the isopotentials.

Ex. Arrino; Allanooka; Deniliquin.

Fault zones may be filled with gouge or minerals, and not permeable.

(b) Joints. Discussion. May be well developed in one rock, poorly in an adjacent one. Exfoliation joints. Effective depth to which water can be transmitted. Jointing in dykes. Joint-bedding relationship as a key to overturning in folds.
Mineralisation.
In-filling with clayey products of weathering.
Jointing in limestones, enlarged by solution.

(c) Caverns. How they are formed, and their relationship to the water table.

Ex. Oscar Ra. tunnel.
Poocher Swamp.
Bordertown.
Port Macdonnell.
Second Valley.
Papua.
L/stone blocks at Naracoorte.
Interdune flats at Pyap.
Blowing bores, Maggea, Waikerie.

(d) Bedding planes sometimes, but by no means always, facilitate water movement through rock. In coarse sediments it is common to find alternating coarse and fine laminae, and water will move more freely through the coarse layers, which however, may not extend very far individually. In indurated sediments, such as most Proterozoic and Palaeozoic and many Mesozoic rocks, water movement is more dependent on jointing than on bedding. Its real importance is that it tells us where the rock extends underground, which of course we must know in choosing a bore site. (c.f.Yatala quarry.) Determining the attitude of a bed is often difficult in the field. Ex. Sturtian Qte., Range rd.

The task may be hindered by the development of structures which look like bedding, but are not. (e.g. - jointing, minor slicks.) The only real lead is to observe the attitude of current bedding. **Ex.** Sturtian Qte east of World's End. However, current bedding itself can also be misleading, because it may be on a large scale. **Ex.** N. Bondi Hawkesbury Ss. The field geologist must never rely on one small outcrop when determining attitude. If possible, stand off at some distance and observe the traces the beds make in a hillside, as often this will show their attitude when a close-up look at the outcrops will not.

(e) Weathering may affect rock by

(i) dissolving it away. This mainly affects limestones, dolomites, aeolianites.

(ii) hardening of the rocks by resilicification; such rocks are usually poor aquifers.

Ex. Penticost Ss. (Wyndham)
Some quartzites in Mt. Lofty Ra. area.
Permian sandstone, Poole Ra.

(iii) causing some of its less stable minerals to decay. Felspars and ferromags are usually affected and the hard rock may become a soft clayey mass.

Ex. Keepit Dam.
Torrensian Slates and phyllites.

Sometimes the weathering is a very quick process.
Ex. marcasite, which causes corrosion.
Depth of weathering is variable.
Clay may fill joints and impede the passage of water.
Ex. valleys at Basket Ra. Terra rossa fillings at Mt. Gambier.

In the search for water among crystalline rocks, weathering is of very great importance. In those parts of the Darling Ra. wwhich are granitic, conditions vary from one flank of the valley to the other. Creeks flow westward, and usually the northern flank is bare, or covered with a thin skin of soil and laterite. The southern flank has a much deeper soil and weathered rock cover. (reason.)

The real importance of the weathering is the profile which results. It can be generalized as:

(i) Laterite gravels and clay, to 8 ft. thick. ——
Soil.
Pisolitic laterite with earthy matrix.
Massive laterite, concretionary, iron-cemented.

(ii) Gibbsite-kaolin clays, up to 20 ft. thick. ——
Laterite-Gibbsite yellow silty clay, minor laterite nodules.
Kaolin clay, resulting from complete decomposition of granite in situ; micas absent, felspars kaolinised, some residual quartz grains.

(iii) Ferruginated zone. ——
This is the zone of groundwater fluctuation, where iron may be deposited to form a "hardpan", but usually just discolours the clays.

(iv) Quartz-residual phase, to 20 ft. thick. ——
Mostly quartz grains and some decomposed felspars. Disintegrates into a mass of clayey sand when immersed in water.

or

Mica-quartz-residual phase, found only above schistose parent rock, amphibolite or banded gneiss. Micas are slightly weathered and form about 50% of rock.

(v) Moderately weathered granite or granite-gneiss. (Can't break drill cores in the hand). This grades downward into fresh rock.

The quartz-residual phase of zone (iv) is the important one because of its higher permeability. Local drillers refer to it as "gravel".

Depth of weathering is of first importance in an arid area. Ex. Ernabella. Recent work in the Eastern Goldfields, of W. A., gave results worth outlining. In some mines, which are invariably sited on crush zones, groundwater occurs down to considerable depths. In Hill 60, Mt. Magnet, potable water is found to 150 feet, brackish water in a few joints at 1000 feet, and brine at over 1400 feet.

Away from the crush zones, we found the rocks too poorly jointed for groundwater ~~at~~ depths of 50-1000 feet below the top of the hard rock.

There are four main rock types. Greenstones (amphibolites) weather readily and deeply to a relatively impervious puggy clay, and give poor supplies of saline water. Dips are steep, and drilling conditions usually difficult.

Metasediments, which resemble the greenstones, have similar characteristics; but the quartz-jaspilite bands, which form strong outcrops are usually well jointed to some depth. Where surface runoff concentrates, localized intake conditions are good enough to provide stock quality water.

When they are silicified, metasediments behave similarly to the crystalline rocks.

Granite and gneiss provide bare outcrops that shed surface runoff, and shallow usable supplies frequently are recoverable from the weathered rock periphery, although the bores tend to silt up.

In the northern areas particularly, a leached pallid zone similar to the Darling Range quartz-residual phase gives good supplies, enough to provide irrigation at Albion Downs.

Quartz veins also remain as a hard, resistant frame-work in weathered rocks; break into blocks, and admit surface water fairly readily. In earlier days, successful wells were often sunk on such veins.

Breakaways are worth investigating, especially when laterite-capped. Describe.

Valley alluvials are generally very clayey and lime-cemented, and a poor water source. However, rubbly ironstone gravel deposits at the foot of hillslopes often yield good stock water. Leached calcrete and sometimes an open box-work of opaline silica commonly occur near the bottom of large drainage lines, often close to salt lakes, and these are invariably good. At Wiluna they give irrigation water, this area being at a relatively high level. Farther south around Sandstone, which is lower, they tend to carry salty water; and still lower and farther south they are absent, the alluvial valleys being salt.

Not all calcrete waters are of good quality, of course. As a general rule, water along the margin is good, but it may become salt near the centre. Ex. Pilbara.

A feature of the calcretes is that their occurrence is readily predicted by photo-interpretation. So also are the

Alluvial Fans, because of their spreading stream pattern and dense mulga growth, as well as an unusual photo-pattern of cusp-shaped groups of scrub at the top of the fan, and thought to be points of fresh-water intake. On the ground their dense scrub growth, clay-sealed surface, and light flood-washed debris can be recognised. The groundwater is usually good near the intake, and on large fans for perhaps as much as a mile downslope.

This ends the section dealing with factors affecting groundwater occurrence. Let us now consider how it moves.

Groundwater Movement.

For our own convenience we arbitrarily divide groundwater into non-pressure and pressure water. Terms "water table", and "hydraulic or piezometric surface".

Non-pressure waters usually have a local intake or series of intakes which may be quite extensive, and respond quickly to rain.

Ex. Fluctuations in w/table at (Pinjar
(L. Gnangara
(Golflands
(Mt. Gambier

Ex. Broome, in relatively flat bush country;
Co. Cardwell
Hunter valley alluvials
Drains acting as intakes east of Coorong.

The type of intake is important, and its duration.
Compare flash runoff to streams, swamps and lakes.
The chemical quality can vary.

Ex. Blackwood R.
Wakefield R.
Rhine R.
Stinking Gully Creek.

Unless augmented by surface runoff concentrations, rain does not penetrate very far into most rocks.

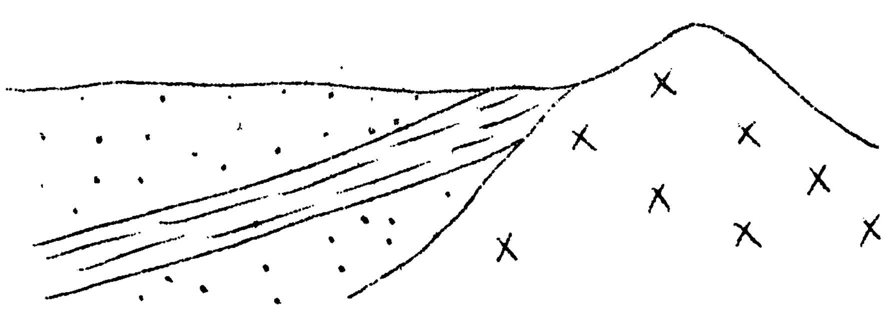
Ex. Co. Cardwell experiment.

Sometimes it rides on a clay band at shallow depth. Perched.

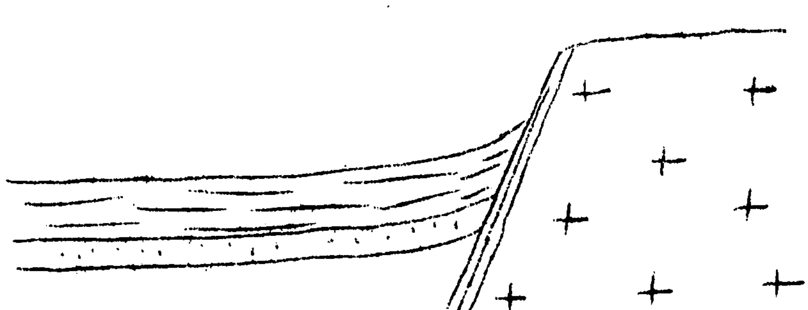
Ex. West coast, S. A.

As water freely moves downslope along an aquifer, it may become confined, and under pressure. Illustrate.
Types of intakes.

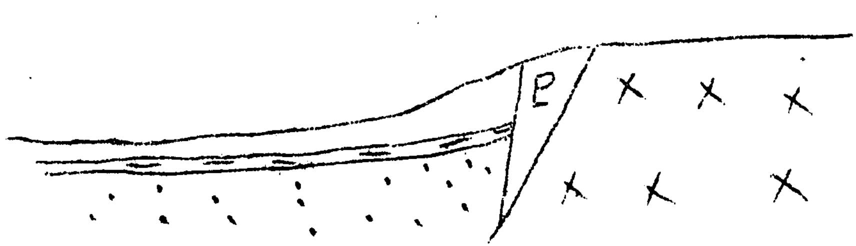
Simple overlap, e.g. G. A. B.



Faulted. Adelaide Plains.



Perth coastal



Effects of block faulting in Mingenew area.
Comment on Canning Basin.

Origin and use of term "artesian".

All ground water will flow under gravity if it can, although it is subject to other influences such as capillarity, osmosis.

Groundwater gradients.

Consider limits of steepness and flatness. Effectiveness of Gascoyne barrage.
Reason for watertable being high under hills.

Pressure water.

Where an intake is high up, water entering it will build up a pressure that is transmitted to all parts of the aquifer. However, it will not cause movement or replacement unless there is an outlet somewhere.
Ex. G.A.B. mound spgs., and gulf of Carpentaria.
Murray Basin springs in river.
Perth Basin, submarine springs.
Flowing bores will locally relieve the pressure. Elastic compaction.
Decline of flows.

Water moving downslope under gravity will take the easiest path through an aquifer. Rate of movement. Note that water can move up-dip.
Differential replacement. Ex. Adelaide Airport bore.
Arrino.
Furthermore, with gradual replacement of saline water in a thick aquifer, the best quality water usually is found near the top, the salinity tending to increase with depth.
Ex. Port Lincoln.
Mandurah.

However, in the Perth Basin, water in the bottom of the South Perth Shale is poorer in quality than that in the underlying Claremont Sandstone. This may be a permeability effect.

The degree of replacement of connate water will vary according to the rate of movement possible for the intake water.
Ex. South side of Murray main basin.

Variations in water level.

Tidal effects. Ex. Torrens Island.
N.W. New South Wales.
Barometric effects.
Caution, beware instrumental errors.
Effect of a rising water face. Ex. Leeton sandbeds.
India.

Springs. Origin, and method of development.
Can they be damaged? Can natural supply be increased?
Mound springs.

Depth to which water occurs. In permeable rocks, the limit to which intake water will penetrate is theoretically governed only by the intake head available, and the relative densities of the waters. Practical considerations.

Salinity.

Origin. (i) Cyclic salts.
(ii) Weathering of minerals.
Consider the control exercised by rock type and by rainfall.
Salinity rises during pumping.
Ghyben-Hertzberg theory. Ex. Pacific atolls.
Derby - modified type.

Nitrates, Iron.

Presence; and origin.

Contamination.

Lodes. Ex. Northampton Pb, Cu.
N.S.W. arsenic mine.
Mine dumps. Ex. Captains Flat.
Marble Bar.
Industrial wastes. Ex. Mil-lel.
N.S.W. abattoirs.
London chalk.
Plating works, Adelaide.
Poocher septic tank.

We have very little knowledge of the travel of pathogens, and research is needed.

Things to remember.

Water is completely logical in its behaviour. If it seems to do something unexpected, be assured that this is either because you haven't got all the necessary facts, or you haven't interpreted them correctly.

In tackling a problem of hydrogeology, a field man must be able to assess the relative importance of the factors listed earlier which control groundwater occurrence. This ability comes only with training and experience. In my opinion it takes a minimum of about four years varied experience before a learner can be relied on to work without fairly direct supervision, and even this minimum will depend on his ability, and more especially his common sense.

Given the ability to assess field conditions properly, the hydrogeologist has certain tools at his command. These will include photo-interpretation; (Ex. Hackathorn, etc.) surface mapping to help determine stratigraphy and structure, and particularly the possible presence of surface intakes; (Ex. Regional Mapping); drilling, which gives us detailed information on stratigraphy, salinities, and water levels; and palaeontology.

Often much of the drilling has already been done by landholders, and the first step in almost every project is a field census to collect every available bit of information on well locations, bore logs, surface levels (barom. etc.), and salinities. (Field salinities, Techtron or Philips.) Farmers often have chemical analyses tucked away among their records, and it is worth spending a fair amount of time to get them, and any other records. Remember that one missed fact may result in an incorrect interpretation. One missed bore record may result in expensive drilling to obtain the same information.

The study of the hydrodynamics of an area must include cross-sections, an isopotential map, and a salinity map. If you have enough information to compile these, you have all you should reasonably need. If not, go and get it. You cannot base an adequate opinion on inadequate data.

Hydrogeology falls in general into one of three categories, which necessitates a flexible approach.

(i) Census Work, often a continuing and long-term project to collect data, and give us a proper knowledge of the area. This work is good basic training. Comment on procedure.
(ii) Government projects, such as town water supplies. This usually involves officers from other departments. Comment.
(iii) Advice to landholders. This is at once the most difficult and the most rewarding of our work.

Always take your time. Usually it is advisable to do a limited bore census of surrounding properties, followed by a look at the local geology, then a mental summing up. I personally have found it best to call first on the farmer, say who you are, ask him just what he has in mind, and what drilling etc. he and his neighbours have done. Then explain that you will have to walk over the area to examine it; that this usually takes some hours; and you will report back when finished. Make sure he will be home.

At all cost, politely discourage him from coming with you. Your ambling about may impress him as pointless; he'll drive you up the wall asking questions; and the neighbours probably won't talk openly in his presence.

With the neighbours, just say who you are and that you are doing a groundwater survey. Ask what they know about local drilling, quarries, tunnels, mines in the area. Never discuss another neighbour's business, even if asked a direct question. Cross-check information; some farmers will suppress information about failure bores, for example. Lastly, never rush them. The average landholder will take an hour or more to get around to telling you what you want to know, and it is always time well spent. However, what you are told has to be realistically interpreted; e.g. hard rock; unlimited supply; good water; deep drilling.

When a landholder asks for geological advice, that advice to some extent will be conditioned by his financial circumstances. Comment.

But whatever advice you give, make sure you explain your grounds for it, and make sure the farmer understands what his chances are. He's going to put a lot of money on your advice, and he's entitled to be told precisely what you think, and why. This end-of-the-day talk with the farmer is the hardest thing for the young field man to learn to handle. First you have to make up your own mind what to recommend and why; then you have to explain it all to the farmer, who may have other ideas of his own; then you have to resist being coaxed into agreeing, against your better judgement, to a bore site that isn't as good as the one you suggest, but is where the farmer wants the water. Sometimes a compromise is possible, and this is one reason why early on you should find out what he himself has in mind.

This vrings me to another point. Always write a report of an inspection, and send the farmer a copy. This ensures

(i) the inspection is adequately done.
(ii) the landholder has a record, and so do you.
Comment.

Divining. Comment.

GROUNDWATER INVESTIGATION OF THE POLDA BASIN EYRE PENINSULA, SOUTH AUSTRALIA

by

R.G. Shepherd, Assistant Senior Hydrogeologist, Mines Department, S.A.

ABSTRACT

The Polda Basin, located near the west coast of Eyre Peninsula, is an area of fresh and brackish groundwater. In area the basin exceeds 1,200 square miles, extending inland for more than 30 miles and with a maximum length of about 50 miles.

Water occurs at depths varying from a few feet to more than 50 feet. The aquifer, which is aeolianite capped by dense limestone, has an average thickness of about 15 feet.

At the base of this aquifer is a sandy clay which is the upper bed of the Tertiary succession in the area. A stratigraphic bore near Polda has shown that Tertiary sand and clay have a thickness of about 200 feet. Sediments of Jurassic age occur beneath the Tertiary and consist of clay, sand and lignite, resting on Precambrian basement at a depth of 560 feet.

Water occurring in the Tertiary and older sediments is brackish, with salinity increasing to more than 7,000 parts per million in the deeper aquifers.

In the upper aquifer there is a considerable area where good quality water occurs. Test drilling has shown that there are three separate areas where salinity of the groundwater is less than 1,000 parts per million. In the first of these, near Polda homestead, an area of 50 square miles has been proved. A second area of probably at least 200 square miles, lies south west of Polda. Drilling is in progress in this area. In addition, a third area of at least 25 square miles occurs north west of Polda homestead.

Very saline groundwater, in places exceeding 14,000 parts per million, occurs in a swampy depression a short distance west of the Polda area. A sandy clay occurs in the swamp and is the upper bed of the Tertiary succession.

Groundwater contours show that there is a westerly fall in the water table of about 5 feet per mile.

The basin has outlets in the form of springs along the coast and in numerous salt lakes in the coastal area. Intake to the basin is from local rainfall which averages 16 inches per annum. In areas where the water table is at shallow depth, e.g. in the Polda area, a relatively large proportion of rainfall probably reaches the water table.

Good quality water in the vicinity of Polda homestead is now being pumped through a 15 inch pipeline to the trunk main at Lock, 20 miles to the east. The water is pumped from a 200 yard trench at the rate of 1 m. gallons per day. Pumping lowered the water level about 4 feet before July, 1964, but winter rainfall completely replenished the aquifer.

Pump tests on two 15 inch bores were done in 1963 and these showed that yields of 20,000 gallons per hour could be obtained. Recently four 43 inch diameter bores were drilled at half mile intervals south of the trench. Two of these were pump tested for 7 days and yielded 38,000 and 42,000 gallons per hour respectively, the specific yields being 46 and 66 gallons per minute per foot of drawdown.

The two remaining bores had much lower yields because the aquifer is thinner and is probably less permeable.

These results show that this type of bore is an effective method of obtaining large yields from the Basin.

SURFACE WATER TECHNIQUES

by
G.T. Roberts, Geological Survey, S.A.

The general significance of surface water hydrology is discussed in the context of groundwater studies. The overlap zone between surface and groundwater is examined and the major points of interaction are considered in terms of recharge to groundwater and losses of groundwater to surface run-off and evapo-transpiration.

It is suggested that hydrogeologists should familiarise themselves with the main techniques in use at the present time to measure surface water resources with the ultimate object of delineating ground and surface resources in detail, and providing the basic data to complete inventories for specific areas.

The lecture is designed to survey the common basic techniques in use in the field and in the office for the measurement and recording of flowing surface water. Brief mention is also made of static water in the form of lakes and swamps.

Streams and Rivers

Two main measurement categories are noted.

1. Water controlled by artificial structures.
2. Natural flows.

Category 1. The main types of artificial flow measurement structures are described and the limitations of accuracy noted.

Category 2. This provides the main problems for hydrographers. Two main operations are involved.

(a) Measurement of water level or stage.
(b) Measurement of discharge.

(a) Water Stage

Various problems associated with the measurement of water stage are noted. Modern water stage recorders are described together with basic details of installation and maintenance.

(b) Discharge

The theory of discharge measurement is outlined. Field techniques in common use are described, including current meters, chemical or radioactive agents and so-called indirect methods such as slope-area techniques. Emphasis is placed on simple measuements which can be used to estimate flood flows in remote areas.

Office procedures are briefly outlined and illustrations are given of the interpretation of hydrographs, particularly in the field of grandwater studies.

GEOPHYSICS

INTRODUCTION

by
W. Wiebenga, Bureau of Mineral Resources, A.C.T.

This discussion will be restricted to shallow basins or valleys. Although deep basins are fundamentally not different, the question of size and depth requires a more structural approach involving different techniques common in oil exploration.

Geophysics in ground-water exploration yields two types of information. Although they often come from the same set of data, distinguishing between them will clarify what may be achieved by applying geophysics. The two types are:

(a) The quality or character of the rock.
In this case the survey data are translated into geological terms or into quantities useful to engineers or hydrologists.

(b) The location of the rock.
This mainly concerns depth measurements; the accuracy of these involves the typical geophysical problem of how to sort the relevant data from background noise or disturbing factors.

Since it is not possible to compress the whole field of geophysics into a few hours, only a limited number of the more common methods and their applications will be discussed. These are shown in the accompanying Table 1.

An inspection of the Table indicates that the main problem facing the user of geophysical data is their translation into geological terms and engineering quantities. For this reason the geophysicist generally requests a geological report, which should include available drillhole data and logs, borehole water yields, rainfall data, etc. Geological data and theory can then be applied as controls in the interpretation. The following examples may serve as an illustration.

In a hydrological seismic survey in Queensland a shallow layer with a 7000- ft/sec velocity was disclosed. Either "weathered bedrock" or "compacted sediments" could fit the interpretation. Because no controls were available at the time of the survey, and because theory indicated that no favourable conditions existed for the occurrence of aquifers, this formation was <u>named</u> bedrock. From then on for practical reasons bedrock was <u>defined</u> as a formation with a velocity of 7000 ft/sec or more.

In the same area the average ground-water resistivity was about 60 ohm-metres or less (control). With a formation porosity for unconsolidated sediments of about 35%, this corresponds to a formation resistivity of about 250 ohm-metres. Hence, subsurface formations with resistivities in excess of 250 ohm-metres are dry or partially dry.

During the same survey, chemical and geological theory were used as controls to outline permeable sub-surface zones suitable for recharging. Clays, deposited in a saline, marine environment, have absorbed salts which are not easily flushed out by fresh water. Also the clays are more or less impermeable. In this survey large variations in formation resistivity were disclosed which could not be explained by variations in ground-water salinity or formation porosity. Therefore the low-resistivity formations were interpreted as clayey formations of low permeability, and the higher resistivity formations as permeable sands suitable for recharge. However, the controls are not applicable in coastal areas where sea water may be expected to invade the near-surface sands.

Another example of geological control was shown in a survey for underground water at Yuendumu, Northern Territory. Resistivity surveys indicated shallow subsurface valleys. Magnetic traversing showed the presence of high, sharp anomalies caused by near-surface bodies. Because the association of magnetite with coarse, unsorted sands or gravels is very common, localities with high, sharp anomalies within the valley zone were considered as favourable targets for drilling.

TABLE 1 - GEOPHYSICAL METHODS IN HYDROLOGY

METHOD	ROCK CHARACTERISTIC	GEOLOGICAL APPLICATION RELATES TO	APPROXIMATE RANGE	CORRELATED WITH	ACCURACY DEPTH MEASUREMENT
Seismic Refraction	Sound Velocity (ft/sec)	Unweathered Bedrock Slightly Weathered to Weathered Bedrock Weathered Bedrock) - Compacted sediments Very Weathered Bedrock) - Water-saturated sands & gravels Clayey Unconsolidated sediments Dry sands	(ft/sec) 13000 ± - 20000 ± 8000 ± - 15000 ± 6000 ± - 8000 ± 4000 ± - 6000 ± < 4000 ± 1000 ± - 2000 ±	Porosity Cleavage, Bedding, Minerals, Faults or Shears Groundwater level in unconsolidated sands	Under favourable conditions 10% of depth
Resistivity Depth Probing	Resistivity (ohm-metres)	Non-porous Unweathered Bedrock Dry, sandy formations Fresh water formations) - 1000 p.p.m. Brackish formation) - 1000 - 3000 p.p.m. Saline formations) - 3000 p.p.m.	(ohm-metres) ∞ > 300 ± 21 - 300 ± 6 - 20 < 6	Major Factor: Salinity of Pore Solution Minor Factor: Porosity and Cementation	Often uncertain but Qualitatively Correct To be used with controls
Gravity (Reconnaissance Method)	Density (g/cm^3)	Subsurface Valleys or Basins	(g/cm^3) 1.9 - 2.7	Porosity Seismic Velocity Overburden and Bedrock	Accuracy depends on available control, usually not very great
Magnetic	Induced & Remanent Magnetisation	Undulations in Bedrock Basic Dykes		Bedrock Features Magnetite Concentrations in sands and gravels	Sharpness of Anomalies gives indication of depth

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GEOPHYSICS IN ARTESIAN BASINS

by
W. Wiebenga, Bureau of Mineral Resources, A.C.T.

Artesian basins cover hundreds to thousands of square miles. Although there is no fundamental difference between the exploration of large, deep basins and small, shallow basins, the exploration cost of large basins is very high. Hence, it is important to start investigations with cheaper reconnaissance methods, followed up by more expensive methods to obtain detail.

Important items to know about a basin are: the boundary, the shape and the thickness of the sedimentary fill. In connection with these problems, the basement may be defined as a more or less old, metamorphosed sedimentary or igneous rock of low porosity (and permeability) and high density in which the chance to find aquifers is very low. This definition is rather loose and elastic and may be changed or adapted to the specific requirements of a geophysical survey. For instance, basement may also be defined as a rock with higher seismic velocities (say 14,000 to 20,000 ft/sec), with higher specific gravities (say 2.4 to 3.0), or as a rock containing many intra-basement magnetic features. The character of rocks filling the basin generally show a strong contrast with basement rocks.

The geophysical techniques in artesian basin investigations resemble those used in oil exploration. A logical investigation sequence would be:

1. Geological and hydrological survey

   Mapping of outcrops, old river courses, etc. Collecting of data of existing bores (yield, depth, temperature, drillers' logs, gamma ray logs, bore-water resistivity and salinity). The above information can be used as a control for the interpretation of geophysical data.

2. Reconnaissance aeromagnetic surveys of ground magnetometer surveys

   The cost of aeromagnetic surveys is high for small areas. However, for very large areas the cost per square mile becomes very low. Magnetic surveys generally indicate the boundaries of the basin by high-gradient, sharp magnetic anomalies. Rough, qualitative depth estimates are also possible.

3. Reconnaissance gravity surveys

   These are usually made along existing roads or tracks, with a station interval of one to five miles. Shape and approximate depths may be determined with the available geological control. Areas may be selected for detailed gravity surveys e.g. to determine the course of sub surface valleys. A big disadvantage of gravity surveys is the requirement of accurate topographical surveying (levelling), which can become very expensive. For the determination of shallow sub surface valleys or channels, a dense observation net is required, which is not very practical if large areas are involved.

4. Seismic refraction surveys

   In large areas, it is more economical not to design continuous seismic refraction profiles, but to place the seismic refraction "spreads" at regularly "spaced" intervals, viz. to sample an area.

   The advantage of seismic refraction work is that it gives both depth and seismic velocity of a formation. Seismic velocity correlates with rock porosity (rock type). From seismic refraction data a basement- or bedrock- contour plan can be constructed.

Together with geological information, seismic refraction data provide excellent control for magnetic and gravity surveys.

5. <u>Resistivity surveys</u>

These can be used to detect locations of salty, impermeable clays, or to determine the approximate boundary between fresh- and salt-water sediments.

Seismic refraction data, and available borehole information give excellent control for resistivity surveys (resistivity depth probing or electrical sounding).

With commercially available resistivity equipment, the depth range covered is usually less than 500 ft. For deeper penetration, special equipment has to be built.

6. <u>Drilling</u>

After interpreting and combining the data of the surveys mentioned above, target areas for drilling may be selected. Boreholes should be drilled with rotary drilling and mud. This makes it possible to apply <u>electrical logging</u> methods.

With electrical logging, aquifers and salt- and fresh-water horizons are easily recorded.

SEISMIC REFRACTION METHOD

by

E.J. Polak, Bureau of Mineral Resources, A.C.T.

When an explosive charge is detonated in a shallow hole, the seismic waves radiate in all directions. Geophones placed on the ground detect these waves and send corresponding electric impulses along a cable to be amplified and photographically recorded. Waves travel from the shot point with velocities dependant on the properties of the rocks traversed and on the type of wave motion.

When the recording station is located some distance from the shock source the refracted longitudinal wave is the first to arrive. In this wave the rock particles are excited along the direction of propagation. Subsequently there is a stronger vertical movement caused by the arrival of the refracted transverse wave, in which the movement of particles along the refractor is normal to the direction of propagation. If, as is generally the case, there is more than one sub-surface interface, there will be a corresponding number of arrivals of each of these wave types. Generally the last wave to arrive (apart from an air wave) is the ground roll, a very complicated movement of ground particles resulting from the passage of the shock wave through the near surface layers. If the velocities of at least two of the waves are known it is possible to calculate all the dynamical properties of the rocks traversed.

Although the vibrations produced by an explosion consist predominantly of longitudinal waves, they are refracted at a discontinuity and travel along an interface as both longitudinal and transverse waves. The "transverse" wave discussed above is that which travels along the interface as a transverse wave but between the surface and the interface as a longitudinal wave. A wave travelling along a discontinuity must enter and leave the discontinuity at the critical angle given by Snell's Law (equation 1).

On a seismic record the shot moment is recorded, followed by deflections on each trace giving the times of wave arrivals at each geophone. Timing lines are superimposed on the record at 10 millisecond intervals.

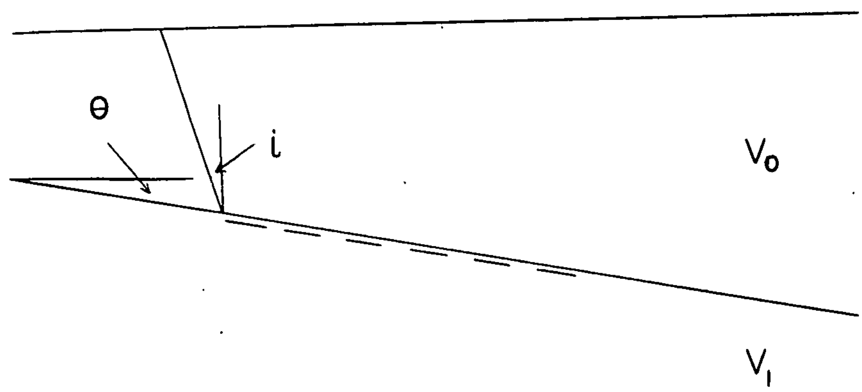
The arrival time of each wave at each geophone is plotted against the distance of the geophone from the shot point, and a time-distance curve is obtained. Figure 1 shows a time-distance curve obtained from field results. The reciprocal of the slope of a segment of the T/D curve gives the wave velocity in the refractor. Figure 2 shows a four layer geological cross section, with the velocities in the lower layers higher than those in the upper ones. The waves are refracted at the interfaces according to Snell's Law. If the same geophone spread is shot in opposite directions the velocities obtained from the T/D curves are numerically equal only if the interface in question is parallel to a plane containing the geophones. If the interface slopes away from the shot point the estimated velocity will be less than the true velocity. Conversely the estimated velocity will be higher than the true velocity if the interface slopes towards the shotpoint. If the travel times in both directions are known the critical angle of refraction, the angle of slope of the interface and the true velocity in the refractor can be calculated (equations 2 to 4).

The depth of the refractor can be calculated from the time-distance curve and several methods for doing this are given in the text books. Most commonly the second segment of the T/D curve is projected back to intercept the time axis at the shot point and the intercept time t, is obtained. The thickness d_o of the top layer is then given by equation 5. In the same way the thickness d_2 of the second layer is obtained from the third segment of the curve (equation 6) and, in the general case, d_{n-1} by use of formula 7.

This method gives the depth of the interface below the shot point. To find the depth to an interface along a traverse a set of geophones is placed along a straight line and shots are fired at both ends of this line (Fig. 3). The depth of the refractor is then calculated from equation 8. V_a is an average velocity and can be found from the equation (7). This is known as the reciprocal geophone method.

For accurate depth determination by the seismic method each layer must be characterised by a higher seismic velocity than the one immediately above it. In ground water investigations, in areas where a perched water table exists, velocity reversal may occur, and a low velocity bed may be sandwiched between two of higher velocity. In this case there is no refracted wave travelling along the upper surface of the low velocity layer. To determine the depth of the underlying refractors a velocity must be assumed for waves in the low velocity layer.

In ground water investigations the velocity of seismic waves commonly varies between 700 and 24000 ft/sec. The velocity in a rock depends on many factors, among them being composition, consolidation texture, porosity, fluid content, pressure and direction of propagation. As the velocity in water is 5000 ft/sec, velocities lower than this indicate that the rock is dry or only partially wet. Velocities in excess of 15000 ft/sec characterise consolidated, low porosity rocks, from which no appreciable quantity of water can be obtained. Fig 4 gives the velocities of seismic waves in those rocks likely to be of interest in groundwater investigations.



$$\sin i = V_0/V_1 \qquad \text{---(1)}$$

$$V_1 = 2\cos\theta \; V_u V_d/(V_u + V_d) \qquad \text{---(2)}$$

$$i = 1/2\left[\sin^{-1}(V_0/V_d) + \sin^{-1}(V_0/V_u)\right] \qquad \text{---(3)}$$

$$\theta = 1/2\left[\sin^{-1}(V_0/V_d) - \sin^{-1}(V_0/V_u)\right] \qquad \text{---(4)}$$

i = critical angle of refraction

θ = angle of dip

V_0 = upper layer velocity

V_1 = lower layer velocity

V_u = up dip velocity

V_d = down dip velocity

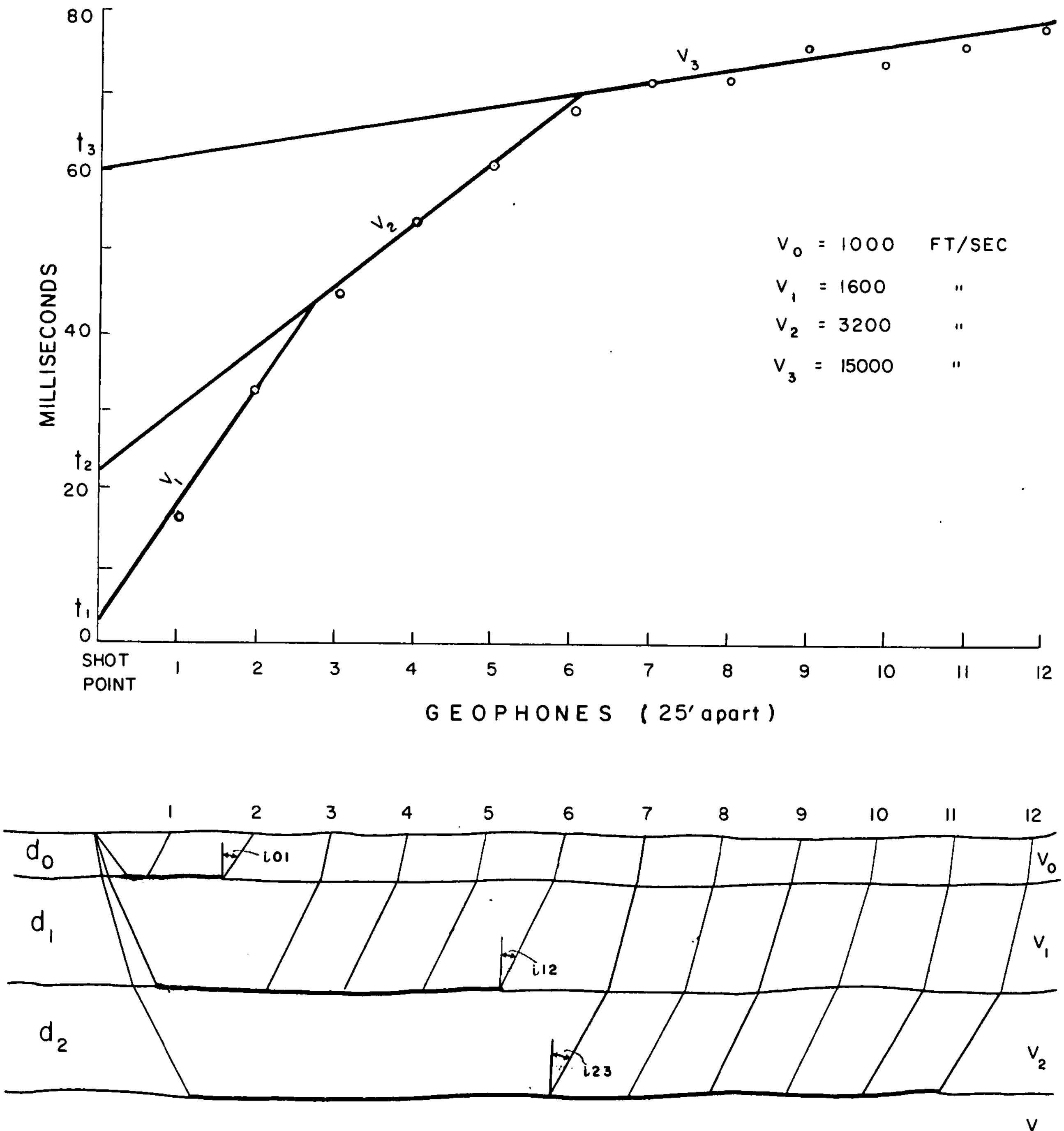


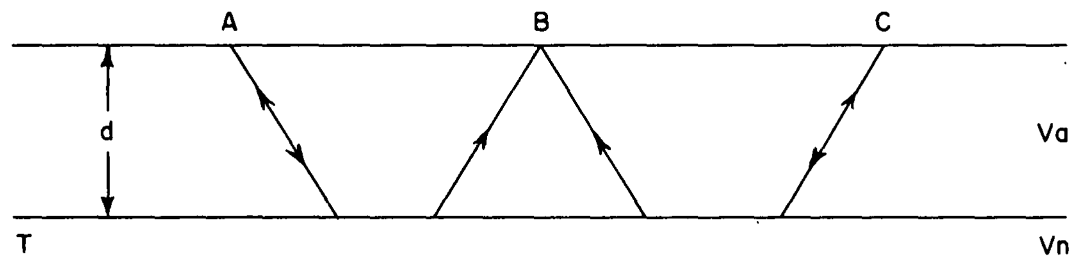
FIG. 1

$$d_0 = \frac{t_1}{2}\,\frac{V_1\,V_0}{\sqrt{V_1^2 - V_0^2}} \quad \text{------------ (5)}$$

$$d_1 = \left(T_2 - \frac{2d_0\sqrt{V_2^2 - V_0^2}}{V_0\,V_2}\right)\frac{V_1\,V_2}{2\sqrt{V_2^2 - V_1^2}} \quad \text{------------ (6)}$$

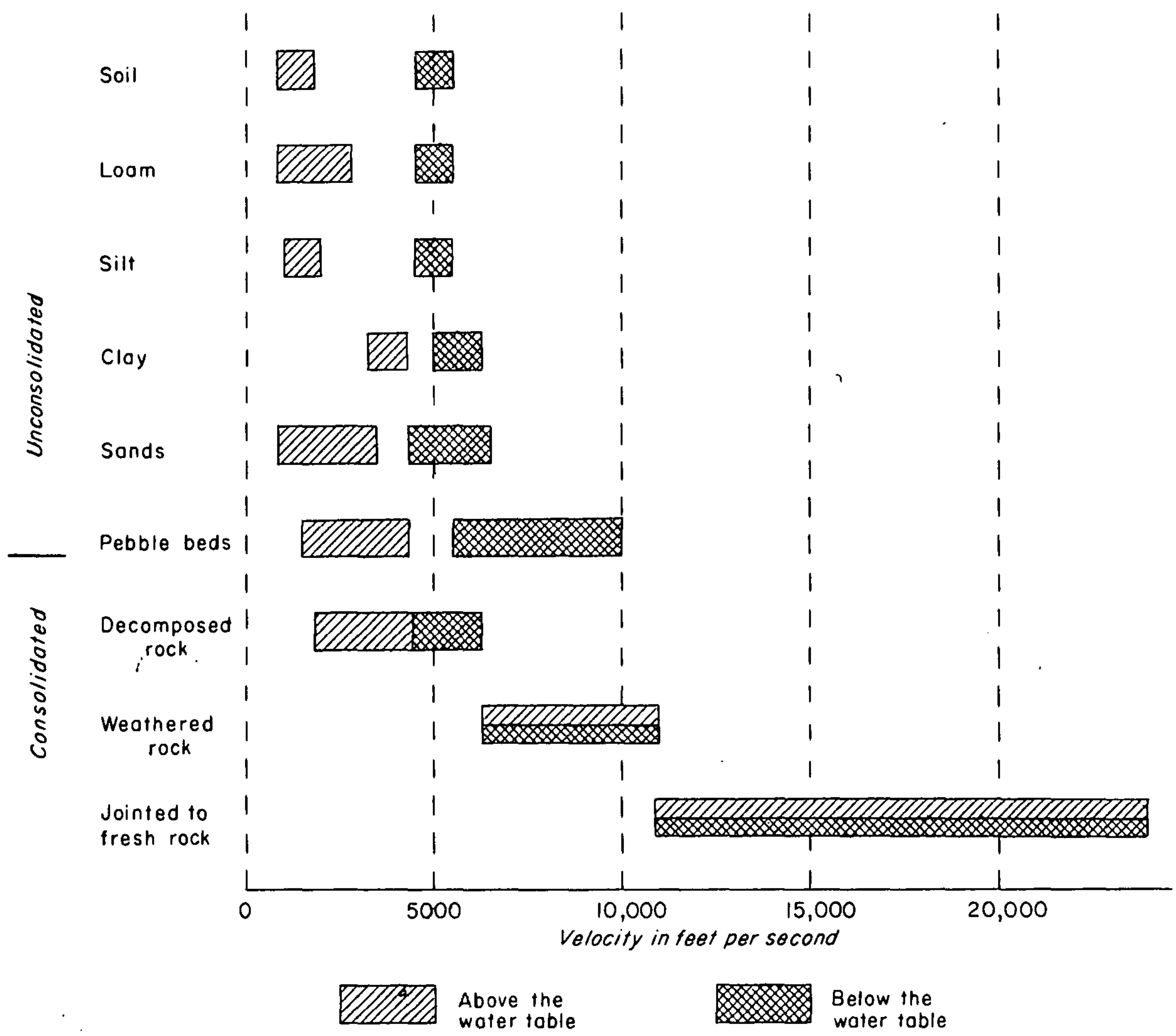
$$d_{n-1} = \frac{V_{n-1}\,V_n}{2\sqrt{V_n^2 - V_{n-1}^2}}\left(T_n - \frac{2d_0\,\sqrt{V_n^2 - V_0^2}}{V_0\,V_n} - \frac{2d\,\sqrt{V_n^2 - V_1^2}}{V_1\,V_n} - \frac{2d_{n-2}\sqrt{V_n^2 - V_{n-2}^2}}{V_{n-2}\,V_n}\right) \text{---(7)}$$

FIG. 2



$$d = \frac{\text{Time AB} + \text{Time CB} - \text{Time AC}}{2} \times Va \quad \ldots\ldots (8)$$

FIG. 3



SEISMIC VELOCITIES

FIG. 4

RESISTIVITY METHOD

by

P.E. Mann, Bureau of Mineral Resources, A.C.T.

General

In the resistivity method, current is supplied to the ground by a pair of electrodes and the potential is measured between two addition points. The ratio of potential to current, multiplied by a spacing factor, gives apparent resistivity as a function of spacing and, hence, as a function of depth penetration. This application makes it possible to determine the depth to basement or bedrock, to water level and to beds of stratigraphic importance as long as the formation boundaries are marked by changes of resistivity or resistivity discontinuities. This procedure is called "electrical sounding", "electrical drilling", or "resistivity depth probing".

If the spacing is kept constant and the arrangement is moved as a whole, horizontal variations in character of variations in depth of a given formation may be determined. This is called "resistivity traversing".

Units

The resistance to current of an element of length 1 c.m., and cross section A cm^2 is:

$$\text{Resistance (ohm)} = Po \frac{l(cm)}{A(cm^2)}$$

dimensions in brackets. Po is a material constant called resistivity, and is expressed in ohm-cm or ohm-metre. Conductivity (σ) is the reciprocal of resistivity, and is expressed in mho/cm or mho/metre.

Resistivity as a hydrological parameter

When the resistivity method is applied to hydrological problems, the following assumptions are generally made:

(a) The current flow is electrolytic (by ions in pore solution).
(b) The rock pores are interconnected.
(c) The resistivity of the mineral grains or rock frame is very high, and for practical purpose treated as infinite.
(d) Rocks under the water table are water saturated, rocks above the water table are dry (high to infinite resistivity). Between these two groups is a transition zone which is often too thin to be taken into account.
(e) With most commercial resistivity meters measurements are made with low___ frequency (square) A.C. current to counteract polarization. In the theory the A.C. current is generally treated as a D.C. current.

For water saturated rock the following relation holds approximately:

$$P = Pw\ F = Pw/P^m \qquad (1)$$

in which

P	rock resistivity,
F	formation factor,
Pw	resistivity pore solution,
P	porosity, and
m	cementation factor, ranging between 1.25 (for shallow unconsolidated sands) and 2.2 (for highly cemented sandstones).

Also, the resistivity of the pore solution is approximately inversely proportional to the salinity S or:

$$Pw = 5000/S \pm \text{ ohm metre} \qquad (2)$$

for temperatures of about 20° C, and S is the salinity in p.p.m.
As an example, take a near surface unconsolidated rock in which P is about 0.33 or 1/3, and m= 1.3, and t = 20° C.
From (1) and (2) it follows that:

$$P = 26000/S \text{ ohm metre}$$

This shows that if the porosity of sediments varies within narrow limits, resistivity measurements may be used to estimate salinity conditions. Vice versa, if the resistivity or salinity of the pore solution is accurately known, resistivity measurements may give the porosity of the rock.

Resistivity inhomogeneous medium

Imagine two current electrodes (Fig.+) C_1 and C_1 at distance L apart ($C_1 C_2$ =L). In line and between the current electrodes are two potential electrodes P_1 and P_2. $P_1C_1 = r_1$, $P_1C_2 = r_2$, $P_2C_1 = R_1$, $P_2C_2 = R_2$.
Then the ground resistivity is

$$\rho = 2\pi \frac{V}{I} \frac{1}{\frac{1}{r_1} - \frac{1}{r_2} - \frac{1}{R_1} + \frac{1}{R_2}} \qquad (4)$$

where V is the potential between P_1 and P_2 and I the current between C_1 and C_2.

In a non homogeneous ground (e.g. with vertical discontinuities or layered formations of different resistivities) the resistivity P is replaced by an "apparent" resistivity Pa.

Electrode Configuration

Many electode arrangements are possible but two are commonly used.

In the Wenner system (see Fig. 2a)

$$P_1C_1 = P_2C_2 = a = \frac{1}{3} L \qquad P_2C_1 = C_2P_1 = 2a$$

where a is the electrode spacing.

Substituted in (4) gives:

$$\rho = 2\pi a \frac{V}{I} = 2\pi a R \qquad (5)$$

If horizontal discontinuities in the sub-surface are present, the same expression is used but the ordinary resistivity is replaced by the apparent resistivity P_a, i.e.

$$P_a = 2\pi a \frac{V}{I} = 2\pi a R \qquad (5a)$$

In electrical sounding the separation "a" is gradually increased from 1 ft 6 in (log a = 0.2) to say 400 ft (log a = 2.6) by amounts which give equal increments of 0.1 in log a. For convenient interpretation log P_a is plotted against log a on a scale of say 1 unit in the logarithm = 2 inches. Theoretical two-layer and three-layer type curves have been calculated and plotted on the same scale.

In resistivity traversing a suitable electrode spacing is adopted and the wholw array moved along traverses.

In the Schlumberger system (Fig. 2b) the distance $P_1P_2 (= \Delta)$ is kept small compared with L; usually $L > 5\Delta$. The centre of P_1 and P_2 is located midway between C_1 and C_2.
$r_2 - R_2 = R_1 - r_1 =$ and, r_1, R_1, r_2 and R_2 equal $\frac{L}{2}$ approximately.

Substituted in (4) gives:

$$P = \frac{\pi}{4}\frac{L^2}{\Delta}\frac{V}{I} = \frac{\pi L^2}{4\Delta}R \qquad (6)$$

and if the resistivity is replaced by the apparent resistivity:

$$P_a = \frac{\pi}{4}\frac{L^2}{\Delta}\frac{V}{I} = \frac{\pi L^2 R}{4\Delta} \qquad (6a)$$

Equation (6a) may also be written as a differential equation:

$$P_a = \frac{\pi L^2}{4I}\frac{dV}{dl} \qquad (6b)$$

In electrical sounding the distance L (or L/2) is increased with each measurement until the potential between P_1 and P_2 becomes too small to read accurately then the distance Δ is stepped up to L/5 and the process repeated until the limits of the equipment are reached. For interpretation log Pa is plotted against log L/2. Theoretical two and three layer curves have been calculated. In resistivity traversing the distance L is made very large, and the potential electrodes P_1P_2 are moved between C_1 and C_2.

Interpretation technique

It can be shown that for "practical" purposes the Wenner and Schlumberger two layer curves are identical if the apparent resistivities (on the log scale) using either system are plotted against the same "current electrode spacing" parameter. Hence one set of interpretation curves can be made up for both systems. A theoretical discussion on interpretation techniques can be restricted to the Wenner system, implying that this automatically applies to the Schlumberger system.

For a two layer Wenner problem the apparent resistivity Pa is

$$Pa = P_1(1 + 4F) \qquad (7)$$

in which F is a converging series in terms of a/h_1 and

$$k = \frac{P_2 - P_1}{P_2 + P_1} \qquad (8)$$

P_1 and P_2 are resistivities in the first and second layer, h_1 thickness of first layer, and "a" the Wenner electrode separation. k is a parameter which determines the shape of the two-layer curves, in which log P_a/P_1 is plotted against log a/h_1. Curve fitting techniques are used to determine resistivity and depth from the data.

Multi-layer interpretation using the two layer curve method

In the instances where there is a multiple number of horizontal layers (Fig. 3) having resistivities P_1, P_2, P_3, ... and thicknesses h_1, h_2, h_3, ... respectively, a technique of resistivity interpretation is used, involving the repeated application of two-layer curves. The method was suggested by Hummel (1932), who showed that two layers having resistivities P_1 and P_2 and thicknesses h_1 and h_2 could be considered as one theoretical layer of thickness $(h_1 + h_2)$ and resistivity P_m, where

$$(h_1 + h_2)/P_m = (h_1/P_1) + (h_2/P_2) \qquad (9)$$

This relation is approximately valid if the assumption is made that there is no flow of current across either of the boundaries between the layers in the section beneath the potential electrodes. This condition will be satisfied where P_2 is less than P_1 and less than P_3, but it will not be satisfied where P_2 is greater than P_1 and P_3.

In the condition where P_2 is greater than P_1 and P_3 another relation, given by Maillet (1947), will be approximately applicable:

$$(h_1 + h_2)P_m = h_1P_1 + h_2P_2 \qquad (10)$$

This assumes that all the current crosses the boundaries in the area under consideration, which is approximately correct if the spacing between the current electrodes is relatively large compared with the thickness of the top layer.

Plate 2 shows a plot of Hummel's relation, and Plate 3 a plot of Maillet's relation, each on a double logarithmic scale. It is seen that the two sets of curves are similar, each being the mirror image of the other about the line k=0. This means that if both equations 9 and 10 are plotted on a double logarithmic scale in terms of P_m/P_1 and $(h_1+H_2)/h_1$, then the curve for a value of k equal to y from equation 9 will be of the same shape as one obtained from equation 10 with k equal to -y; but this will appear as a mirror image.

Since the condition for satisfying the Hummel relation is that P_2 is less than P_1, i.e. k is negative, and the condition for satisfying the Maillet relation is that P_2 is greater than P_1; i.e. k is positive, both sets of curves can conveniently be incorporated in one set of help curves. (Plate 4)

By use of this set of help curves in conjunction with the two-layer type curves, it is possible to interpret a multi-layer case by successive reductions of the top two layers to single equivalent layers.

Ambiguity in Hummel and Maillet Relations

Equations 9 and 10 may be written in the forms:

$$(h_1 + n h_2)/P'_m = (h_1/P_1) + (n h_2/n P_2) \qquad (11)$$

and

$$(h_1 + n h_2)P'_m = h_1 P_1 + (n h_2)(P_2/n) \qquad (12)$$

Hence, using Hummel's relation for a low-resistivity layer between two high-resistivity layers, a layer of a certain resistivity P_2, and thickness h_2 may be replaced by a layer of higher or lower resistivity and larger or smaller thickness respectively as long as the ratio h_2/P_2 is kept constant.

Using Maillet's relation for a high-resistivity layer between two lower resistivity layers, a layer of a certain resistivity P_2 and thickness h_2 may be replaced by a layer of lower or higher resistivity and larger or smaller thickness, respectively.

This illustrates the basic ambiguities inherent in resistivity depth probe interpretation. Additional control in the form of borehole information or seismic data is required to make depths and resistivity estimates accurate or reliable. Nevertheless, valuable qualitative information may be obtained.

Multi layer curves

In addition to two-layers curves, several authors have published three layer curves. Although useful, they still do not give the complete answer, and the basic ambiguity mentioned in the previous paragraph is not removed. For a quantitatively correct interpretation controls are necessary at some places.

Depth control

The following examples illustrate how depth control is applied in electrical sounding.

(a) A seismic survey in a shallow basin discloses (Fig. 4) dry soil and sand (velocity 2000 ft/sec) to 20 ft, water saturated sands (velocity 5000 ft/sec) to 100 ft, then weathered bedrock (velocity 7000 ft/sec). Bores in the area indicate saline water in or close to the weathered bedrock. The porosity of the sediments is about

0.33 or 1/3. Electrical sounding data give 5 ohm metre to 20 ft, 20 ohm metre to 150 ft and 1 ohm metre below 150 ft, i.e.

$$P_1 < P_2 > P_3$$

The resistivity results are reduced to the seismic results by applying the equivalence rule based on Maillet's principle, viz.

$$P' = \frac{hP}{h'}$$

Electrical sounding indicates a 20 ohm metre (slightly brackish) layer to 150 ft. However, the bedrock is not 150 but 100 ft which gives an adjusted resistivity.

$$P_2 = \frac{150 - 20}{100 - 20} \times 20 = 320$$

Substituting in (1) $P_W = P_2 P^m$ the values $P = \frac{1}{3}$, $m = 1.3$ and $P_2 = 320$ gives $P_W = 16$ ohm metre and using the formula $S = 5000/P_W$ (for $t = 20^{\circ}C$ app.) gives $S = 5000/16 = 300$ ppm salt. This means fresh water in the sediments to a depth of about 100 ft.

(b) A seismic survey in a shallow basin discloses sediments of 4000 to 6000 ft/sec to a depth of 100 ft, overlaying unweathered bedrock of 15000 ft/sec. However, geological evidence suggests about 20 ft weathered bedrock (say granite wash), too thin to be detected by the seismic. The porosity of the sediments is about 0.33 or $\frac{1}{3}$.

Electrical sounding indicates 100 ohm metre (fresh water) to 80 ft, 21 ohm metre (fresh to brackish water) to 200 ft and below 200 ft a nearly infinite resistivity, i.e.

i.e. $$P_1 > P_2 < P_3$$

However, the depth of the bedrock is not 200 ft but 100 ft. The resistivity data are made into agreement with the seismic results by applying the equivalence rule based on Hummel's principle $P' = \frac{h'P}{h}$

$$P_2 = \frac{100 - 80}{200 - 80} \cdot 21 = 3.5 \text{ ohm metre}$$

Substituting in $P_W = P_2 P^m$ the values $P = \frac{1}{3}$, $m = 1.3$ and $P_2 = 3\frac{1}{2}$ gives $P_W = 0.8$ ohm metre and using the formula $S = 5000/P_W$ (for $t = 20^{\circ}$ app.) gives $S = 5000/0.8 = 6000$ ppm salt. This means saline water above the unweathered bedrock, and below about 80 ft within the weathered bedrock.

The two examples above show that seismic and resistivity methods form a good combination in hydrological investigations.

Resistivities of sands and clays

Instead of assuming an average porosity of 0.33 for shallow, unconsolidated sediments a range of $P = 0.33 \pm 0.08$ may be considered:

$P = 0.25$ for unsorted sands and gravels,
$P = 0.33$ for clayey sands and
$P = 0.41$ for sorted clays.

Further, assume for a concrete example fresh ground water ($S = 100$ ppm, $P_W = 50$ ohm metres). Table 2 shows the parameters computed with the usual formulae.

TABLE 2

Name	P	P^m	P for $P_W = 50$ (ohm metres)
sands and gravel unsorted	0.25	0.165	303
clayey sands	0.33	0.237	211
clays sorted	0.41	0.314	159

Note that the resistivity of sand and gravel could be about twice the resistivity of clay.

Now it frequently happens that much lower resistivities are found (say down to 10 ohm metres) than may be expected from the data, i.e., the ratio between observed possible maximum and minimum resistivities is much higher than 2 (exclude dry formations). In these instances there may be some interesting explanations such as :

(a) The sediments were originally deposited in a saline environment. In contrast to clays, sands show a marked capacity to absorb salt. In the subsequent geological history ground water has flushed out the sands whereas the unpermeable clays have retained the salt. This means that the low to very low resistivities are associated with clay, medium to moderately high with sands and gravels, and very high resistivities with dry or partly dry formations.

(b) Weathered bedrock is often formed by the breakdown of minerals forming salts and clayey formations. Hence, weathered bedrock is often associated with low resistivities in contrast to the flushed out overlying sandy formations. In coastal areas where there may be saline water encroachment, the general range of resistivities may be very low. Water mixing may take place, and it will be difficult or impossible to distinguish sands from clays by resistivity work only. All which may be said from resistivity measurements is; that the water, irrespective of whether it is in sand or clay, is saline.

Fig 1.

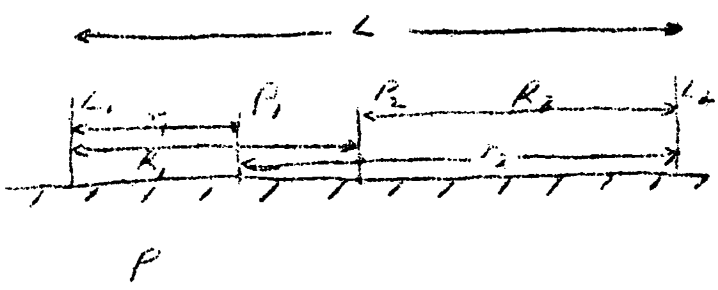


Fig 2(a) Wenner configuration

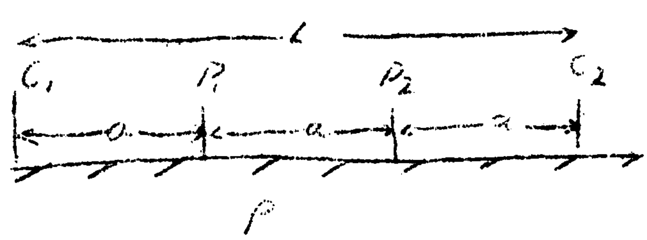
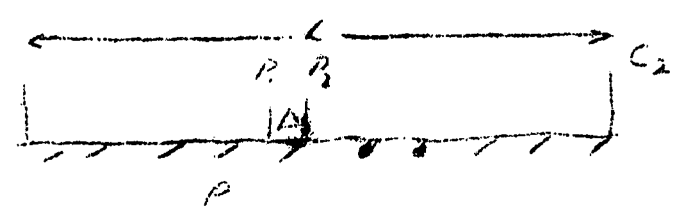


Fig 2(b) Schlumberger configuration



$L > 5\Delta$

Fig 3

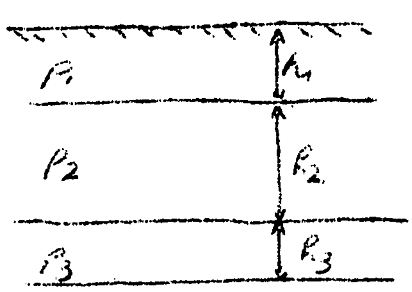
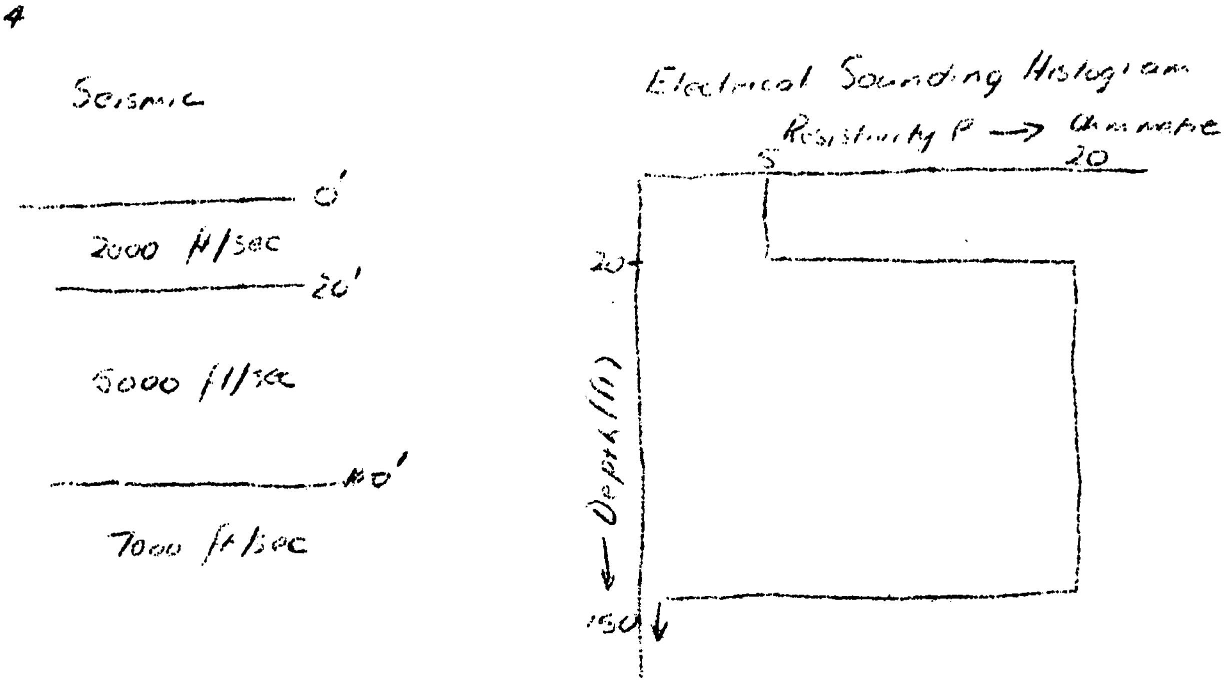


Fig 4



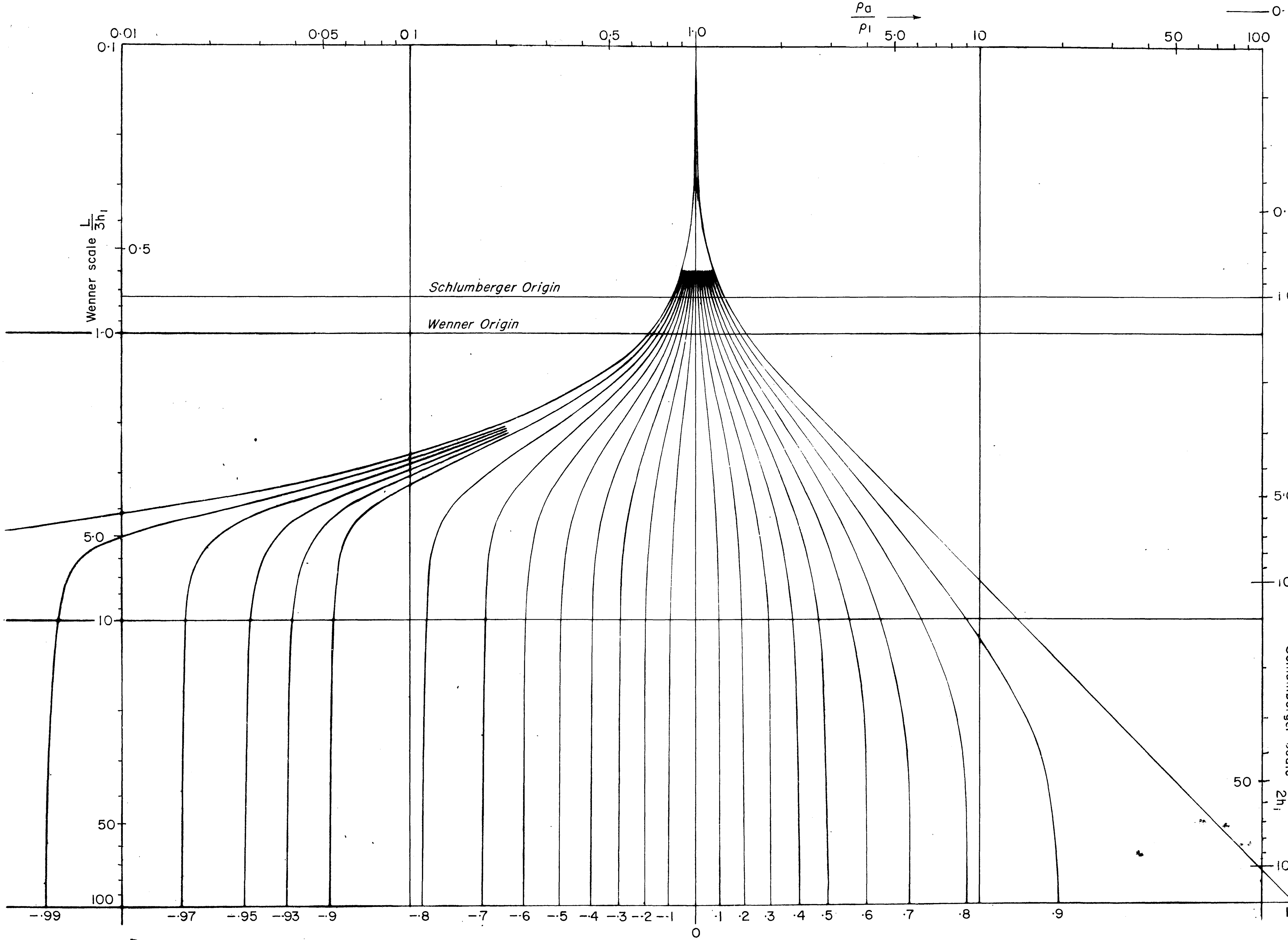
RESISTIVITY EQUIPMENT

TABLE 1.

Instrument & Manufacturer		Approximate Duty free cost	Approximate weight & size	Resistance ranges (ohm)	Current and frequency	Output Voltage	Remark:
Megger Earth Tester (i) 0-1000 ohm (ii) 0-3000 ohm	Evershed & Vignoles Ltd. London.	£250	23 lb 14" x 7" x 7"	(i) 0-1, 10, 100, 1000 (ii) 0-3, 30, 300, 3000	Commutated DC 50-300 cps hand cranked generator	90V	Power insufficient for C_1-$C_2 \gg 400\pm$
Y.E.W. specific earth resistivity tester. L-10	Yokogawa Electric Works, Japan	£160	23 lb	0-0.3, 3, 30, 300	Commutated Dc 200-300 cps hand cranked generator	370V	Same as above
Tellohmeter	Nash & Thompson Surrey England	£220	30 lb 14" x 11" x 5"	0-0.3, 1, 3, 10, 100, 100, 10,000	A.C. 11 cps Battery operated vibrator.	150V	Same as above
Geophysical Megger	Evershed & Vignoles Ltd., London	£850	Generator) 15" x10"x10") 104 lb ohmmeter) 12"x10"x 6"	0-0.3, 1, 3, 10, 30	Commutated DC 25-60 cps hand cranked generator	110V	Very bulky. Power insufficient for C_1-$C_2 \gg 2000\pm$
Terrameter	A.B.E.M. Co. Stockholm Sweden	(i)£740	G-Box 18 lb 14" x 8" x 6" V-Bos 13 lb 14" x 8" x 6"	0-1.0, 100, 10,000	Approx. A.C. 4 cps square wave from battery operated transistor oscillator	100, 200 400V	Power insufficient for C_1-$C_2 \gg 500'\pm$
		(ii)£770	High power G-Box 180 lb	0-0.1, 1.0	A.C. Power source accumulator	10, 100, 300, 400V	No practical experience available
Resistivity meter RM1	Bureau of Mineral Resources	-	Detector 15" x 9" x 8" 3 Battery boxes 100 lb.	0-.01, 0.1, 1.0, 10 100, 1000	D.C. Dry cell batteries	4.5, 9. 18, 45, 90, 270, 540V.	Slow operation used for C_1-$C_2 < 3000'$

NOTE: C_1-C_2= current electrode spacing. Table is not intended to be exhaustive.

PLATE I



Two-layer Type Curves for resistivity depth probe interpretation   Value of k from − 1·0 to + 1·0

TO ACCOMPANY RECORD No. 1965/18

G16-32

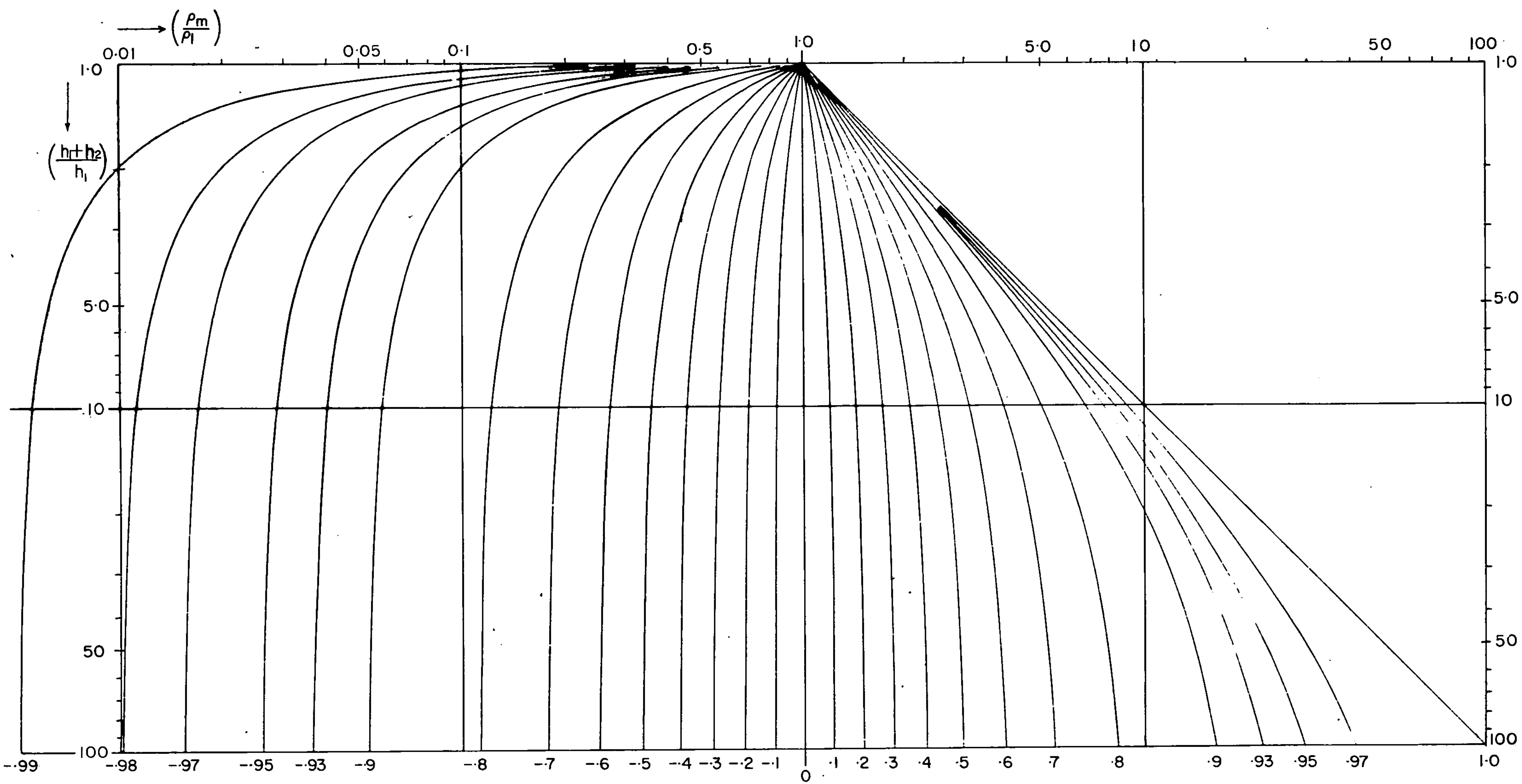
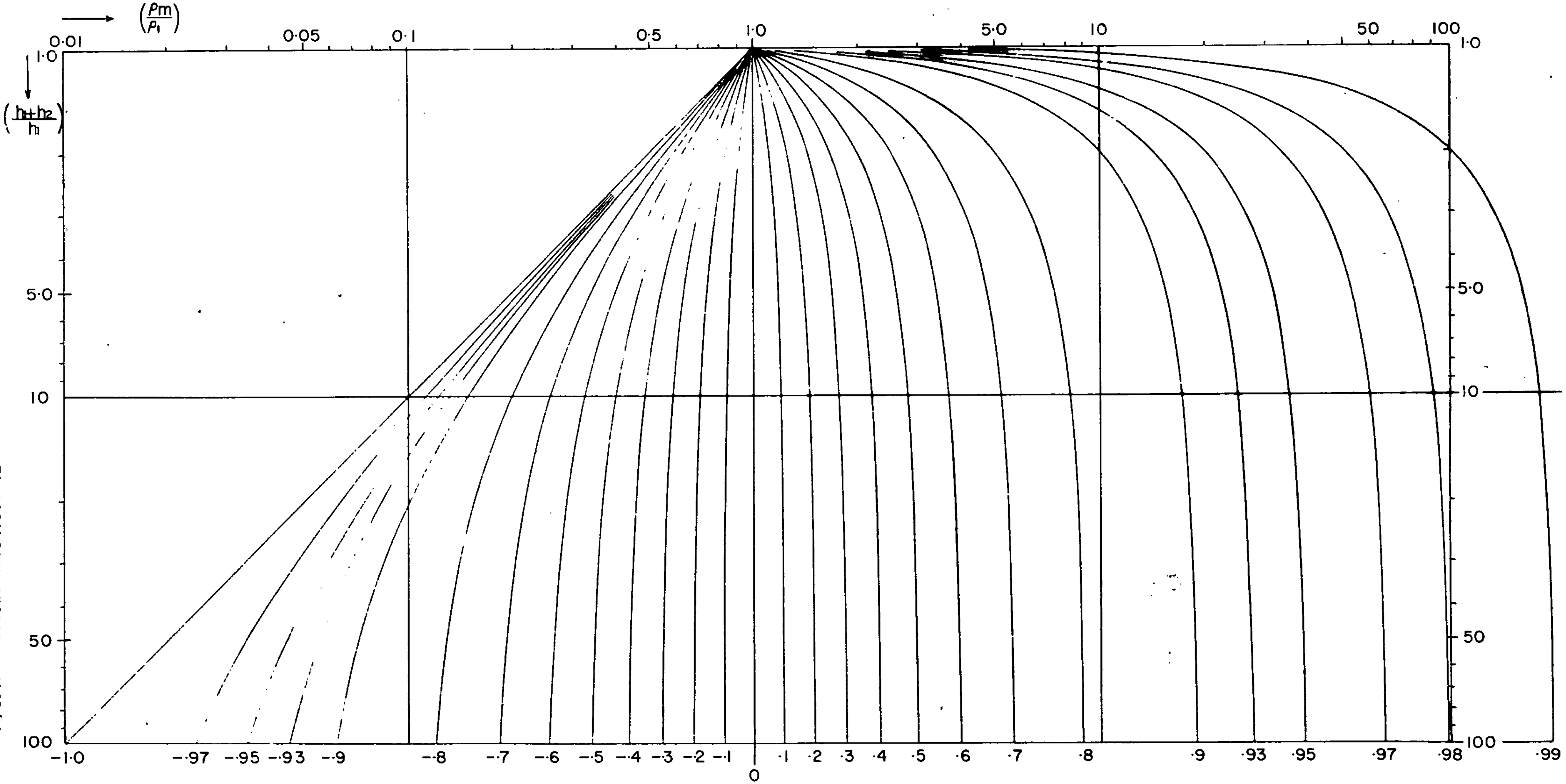


PLATE 2

Help curves for resistivity depth probe interpretation based on the Hummel relation $\frac{h_1+h_2}{\rho m} = \frac{h_1}{\rho_1} + \frac{h_2}{\rho_2}$ for varying $k = \left(\frac{\rho_2-\rho_1}{\rho_2+\rho_1}\right)$

TO ACCOMPANY RECORD No. 1965/18

G16-29



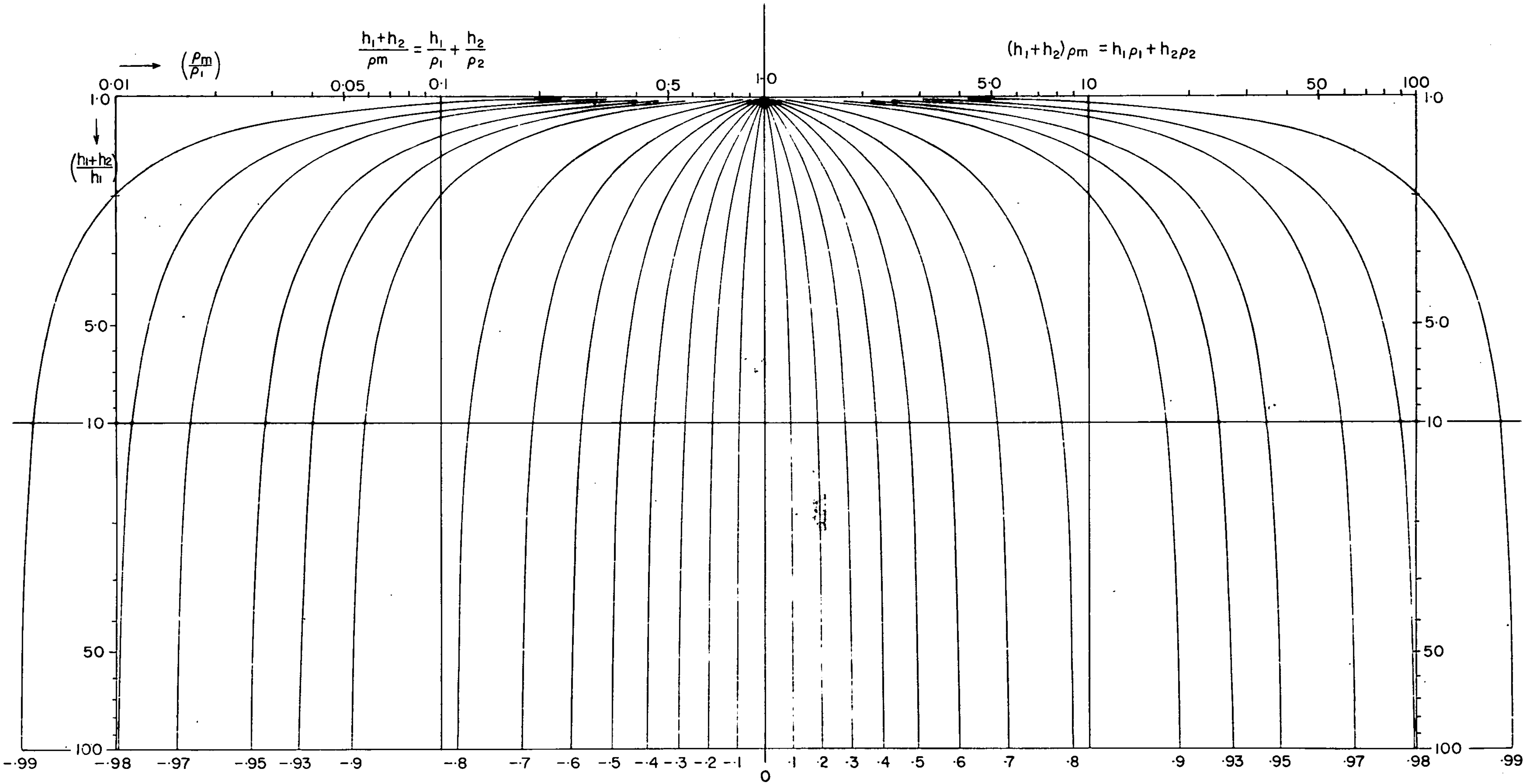
Help curves for resistivity depth probe interpretation based on the Maillet relation $(h_1+h_2)\rho m = h_1\rho_1 + h_2\rho_2$ for varying $k = \left(\frac{\rho_2-\rho_1}{\rho_2+\rho_1}\right)$

PLATE 3

TO ACCOMPANY RECORD No. 1965/18

G16-30

39



Help curves for resistivity depth probe interpretation for a combination of Hummel and Maillet relations for varying $k = \left(\frac{\rho_2 - \rho_1}{\rho_2 + \rho_1}\right)$

PLATE 4

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G16–31

40

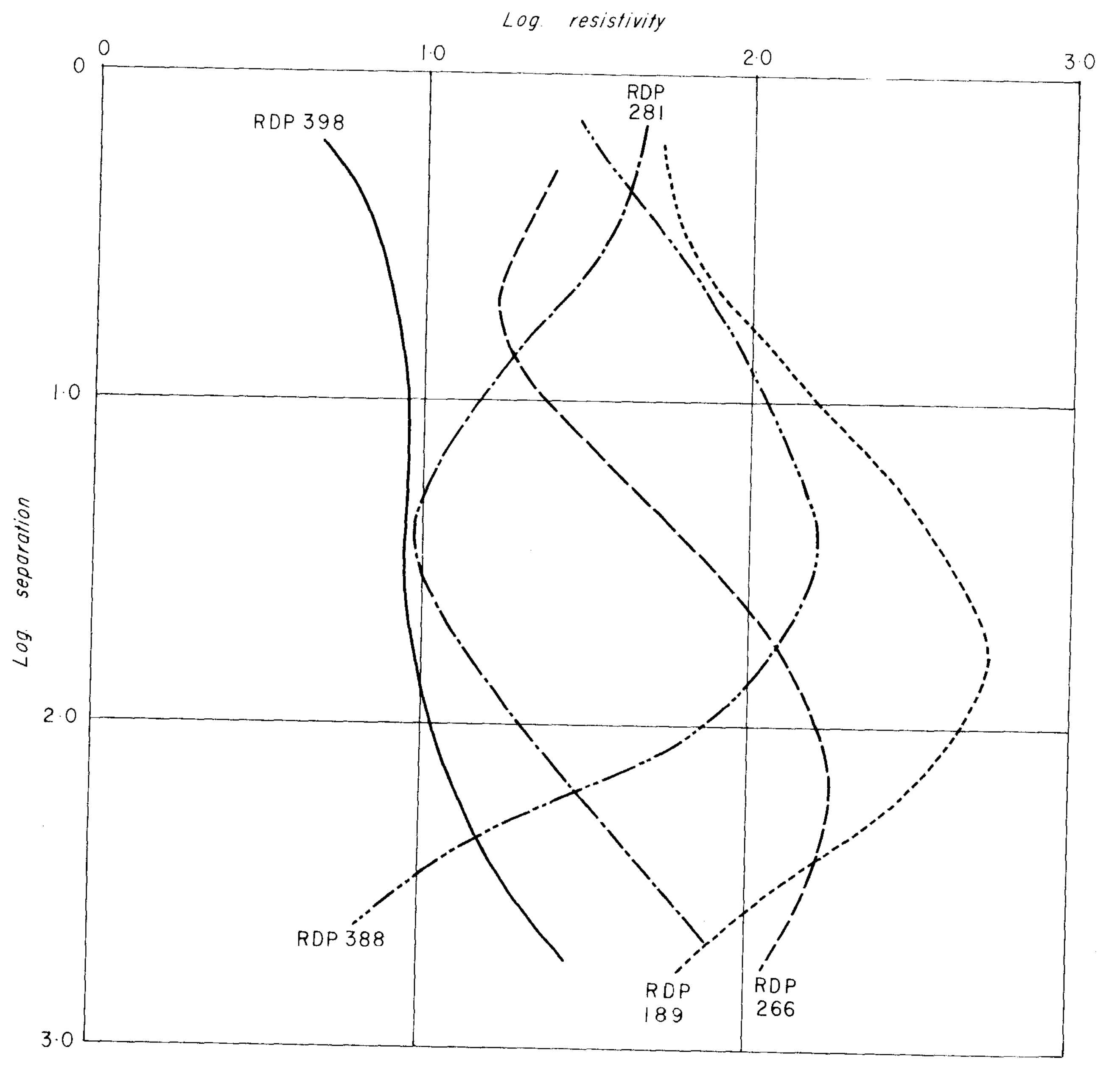


Figure 8 Typical Wenner resistivity depth probe curves

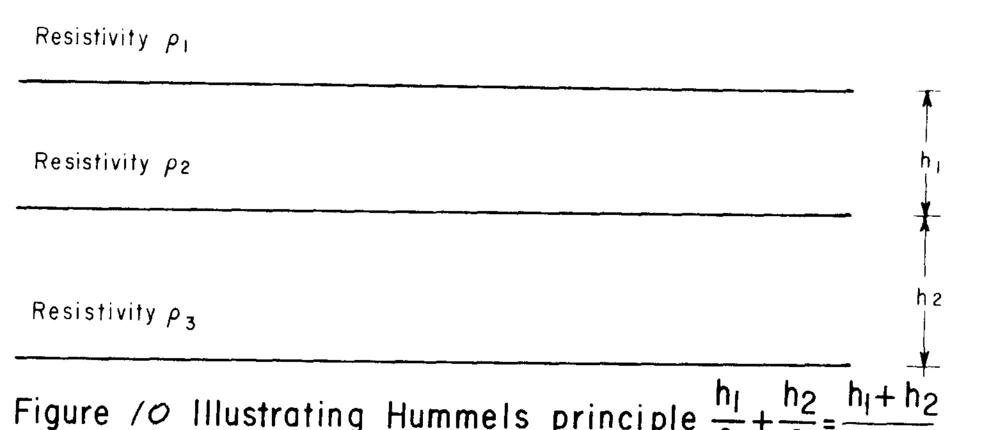


Figure 10 Illustrating Hummels principle $\frac{h_1}{\rho_1}+\frac{h_2}{\rho_2}=\frac{h_1+h_2}{\rho_a}$

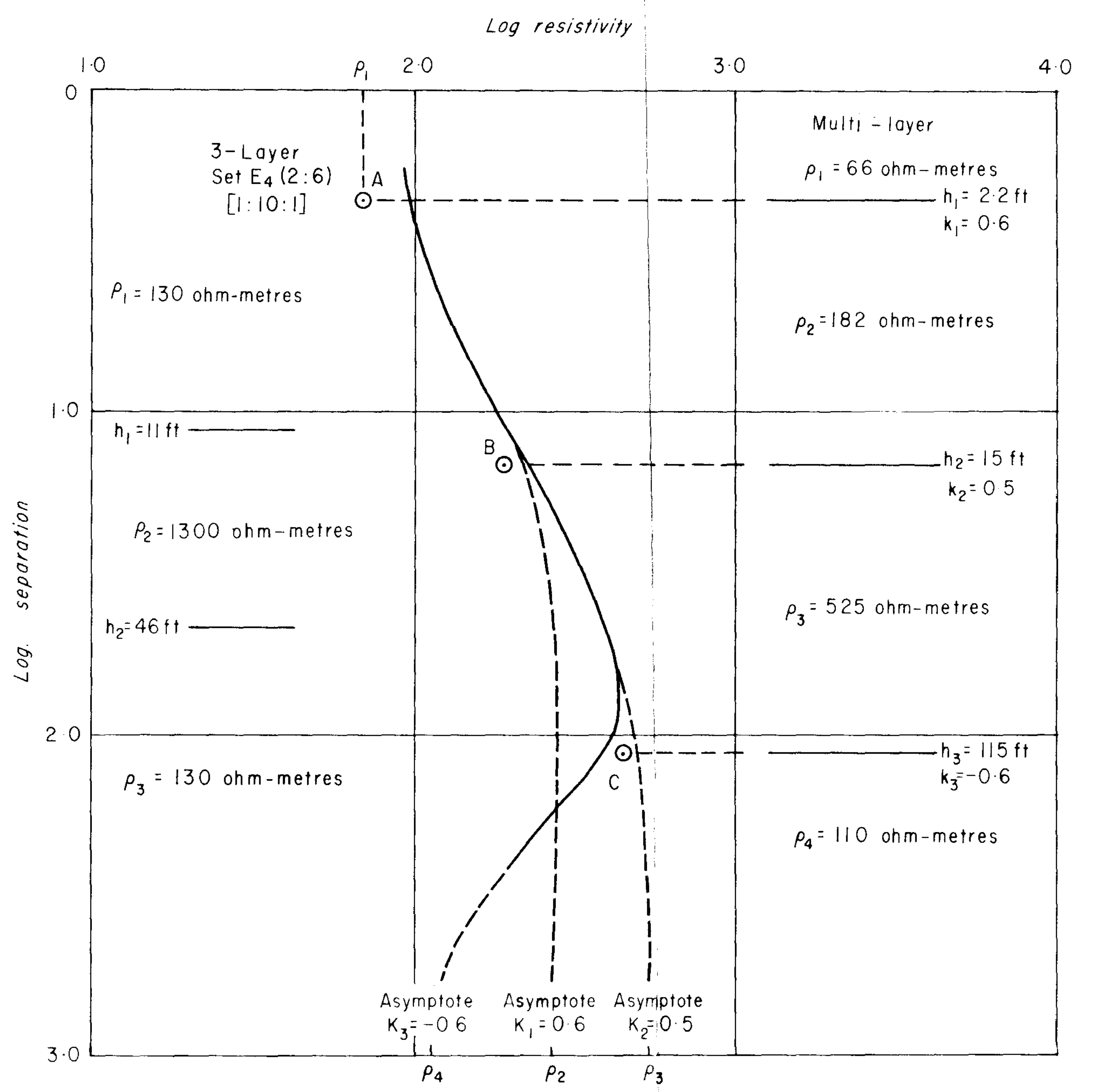
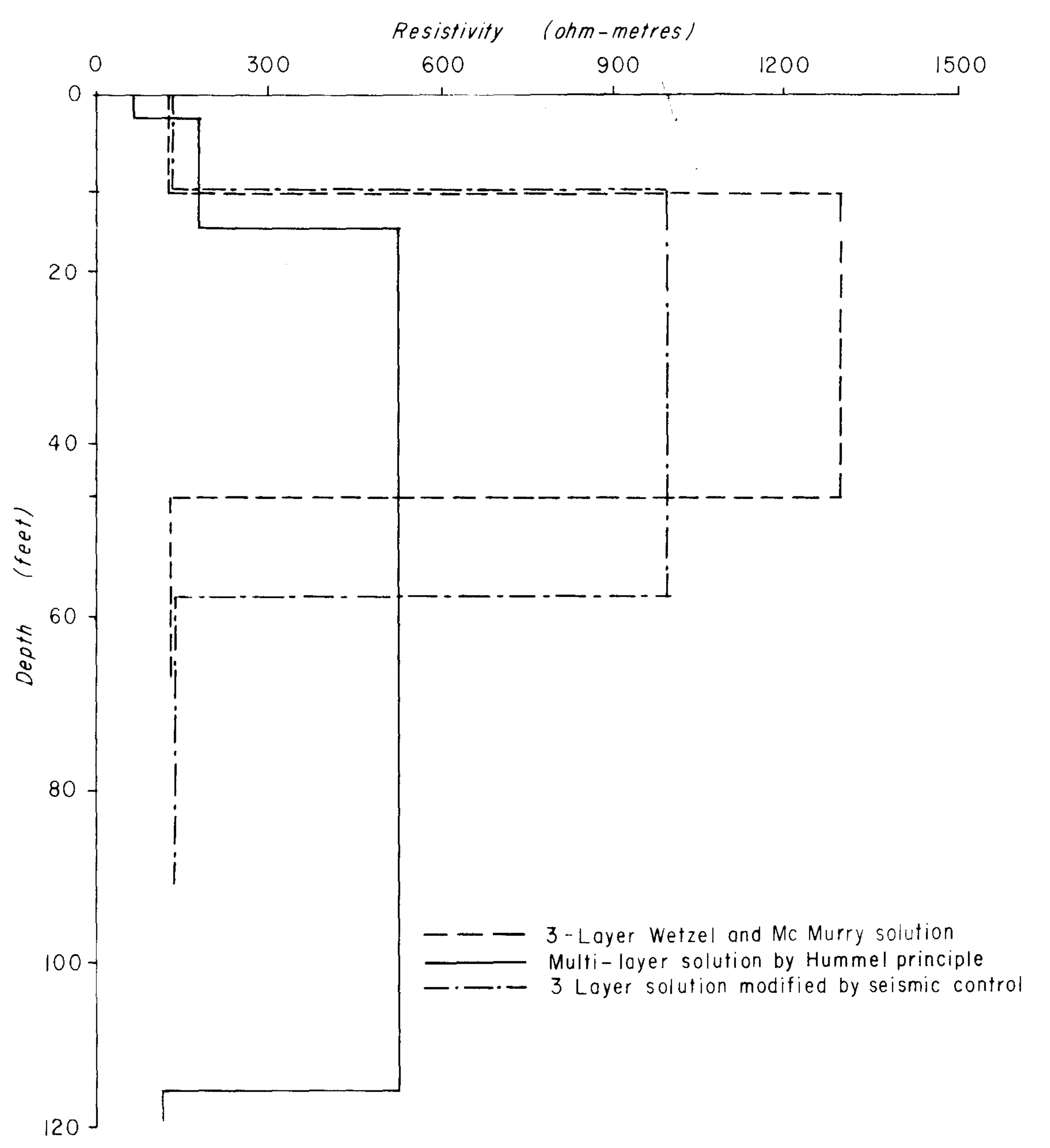
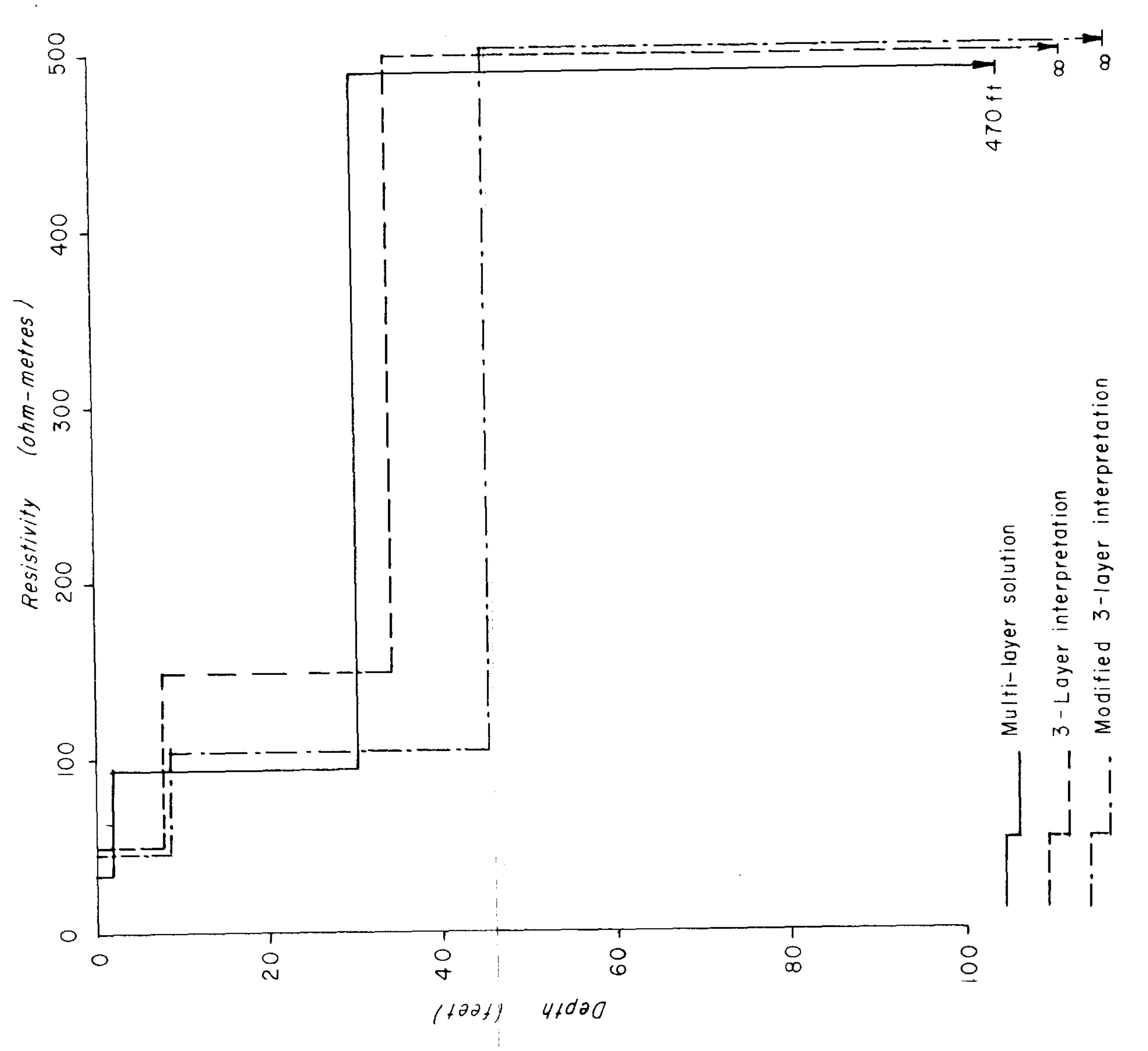


Figure 12
RDP 235 Reinterpretation of multi-layer Hummel interpretation by 3 layer method (Wetzel and Mc Murry (1936)



Interpretations of curve



Interpretations of curve

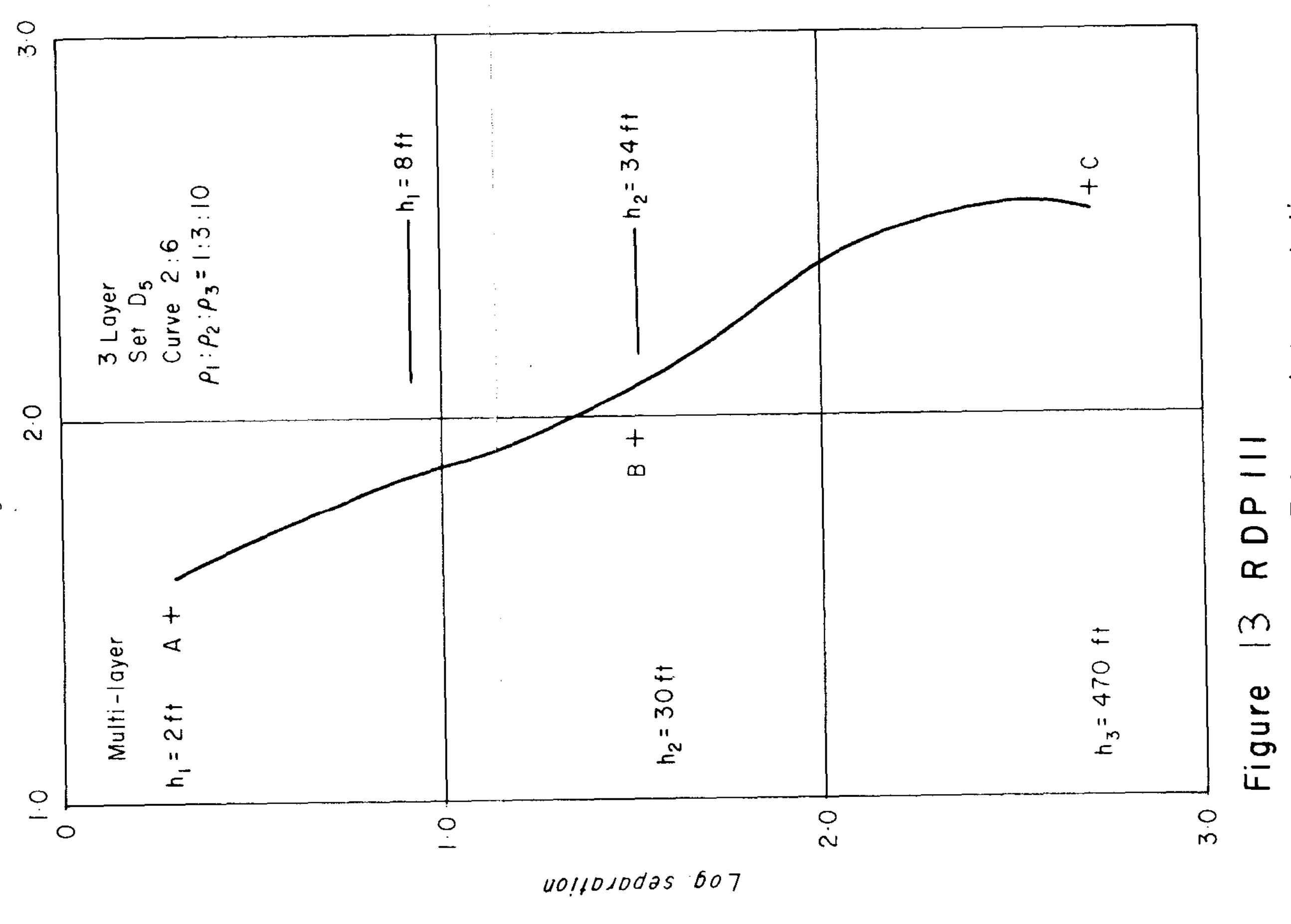
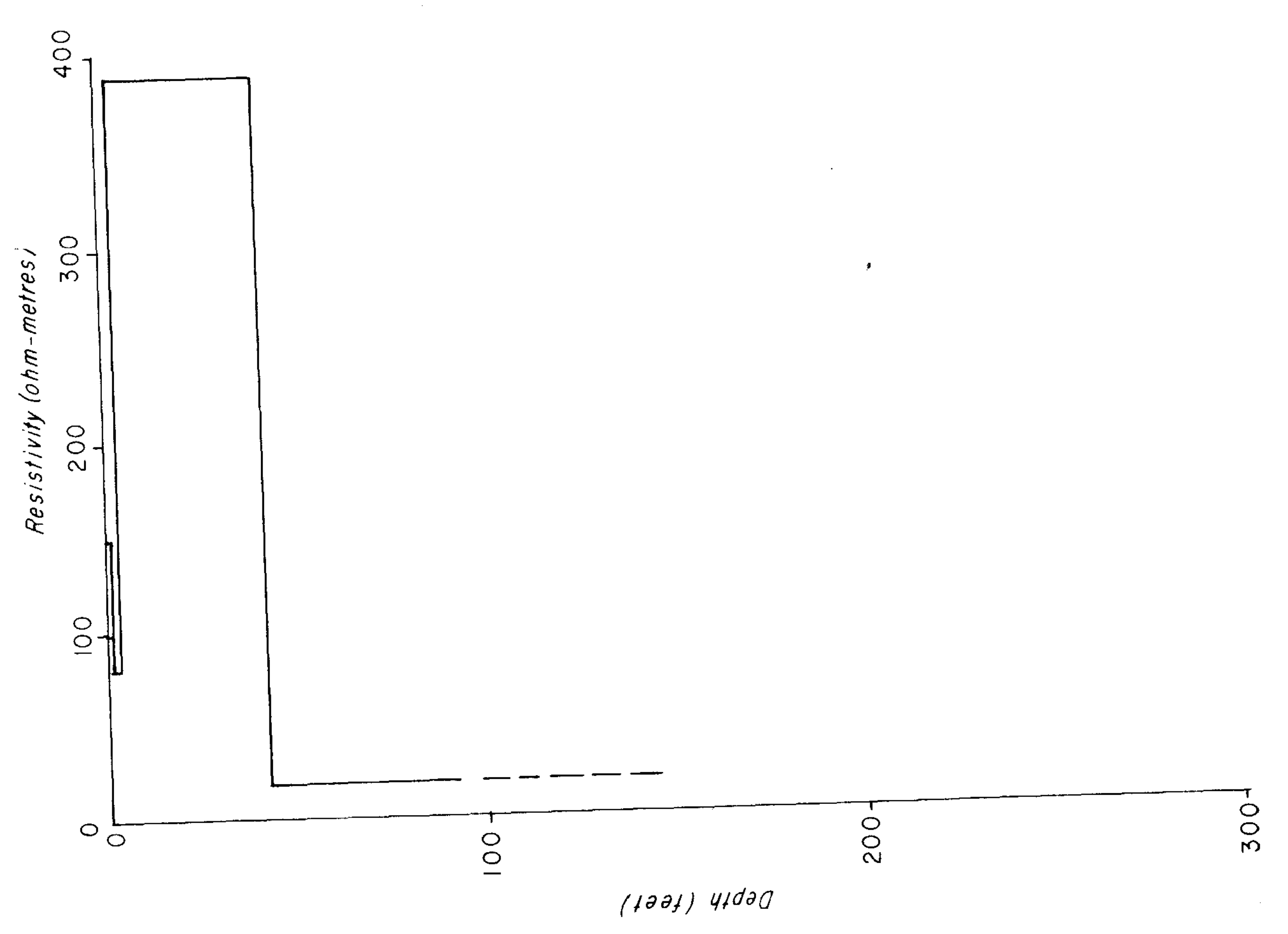


Figure 13 RDP 111
Multi-layer and 3 layer interpretations



Interpretation of curve

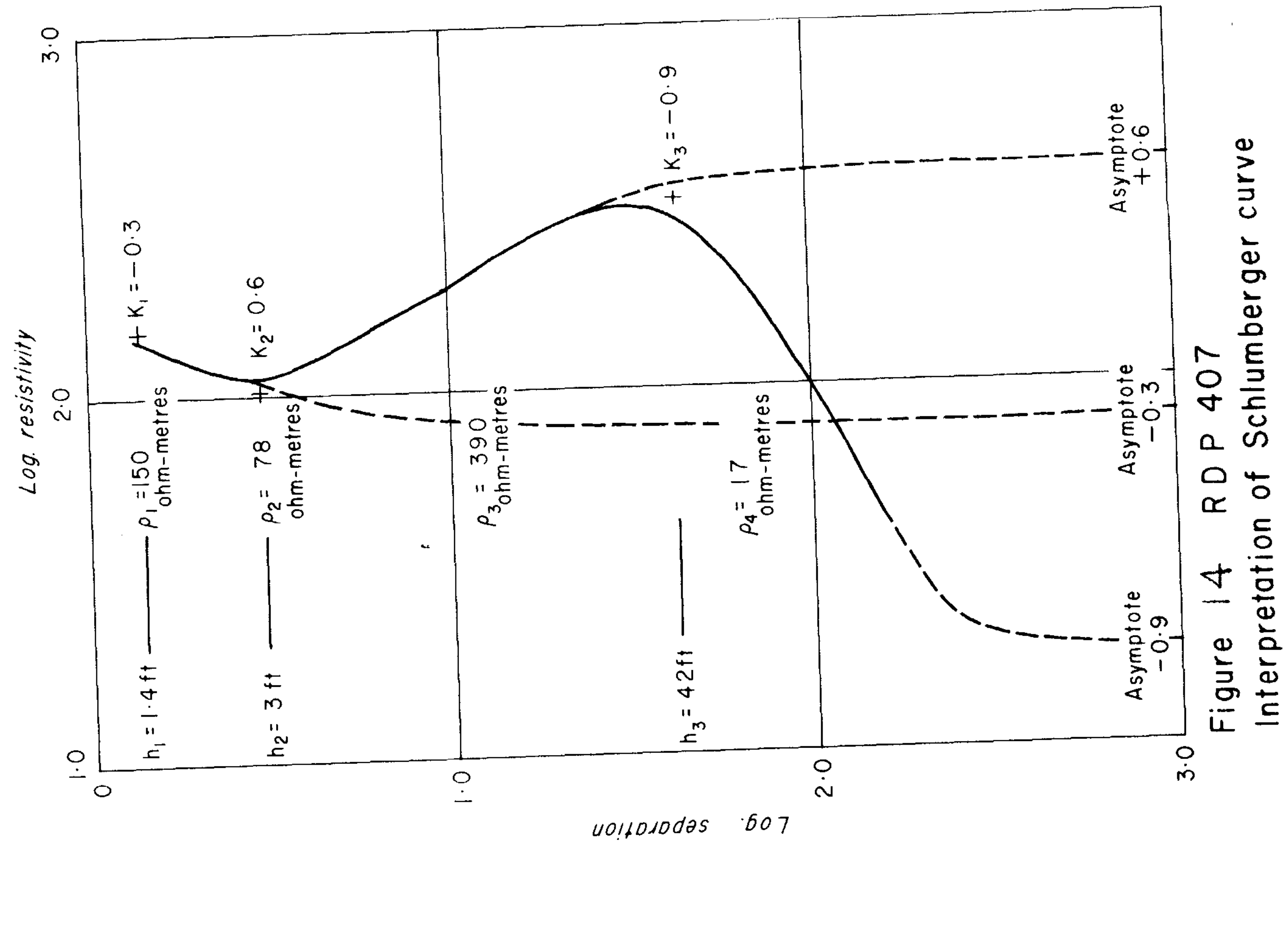


Figure 14 RDP 407
Interpretation of Schlumberger curve

LOGGING

by

E.J. Polak, Bureau of Mineral Resources, A.C.T.

In well logging the physical properties of the rocks penetrated by the drillhole are continuously recorded in terms of depth. The logs obtained give relatively detailed information on some properties of the rocks or the fluids contained both in the drillhole and in the rocks. Further more accurate cross-section of the area can be obtained by correlating the logs between drillholes.

The following properties can be measured:

Electrical resistivity of the rocks - which indicates the content, salinity and distribution of water in the rock.

Spontaneous potential - which compares the water in the drillhole with the water in the rock.

Natural gamma radiation - which is proportional to the concentration of radioactive elements in the rock.

Induced radiation - which varies with the hydrogen content of the rock.

Velocity of ground waves - through the formation, which depends upon the formation porosity.

Temperature gradient - which is related to the thermal conductivity of the rock and the fluid enclosed in the rock.

Diameter of the hole - which varies with the degree to which the rock resists erosion and solution.

In all these methods an appropriate probe (Fig. 1) is lowered into the drillhole on the end of a cable and measurements of the relevant property are taken and recorded while the probe is being winched out of the hole.

A. Electrical Logging

Electrical logs are of two types: (1) resistance or resistivity; (2) spontaneous potential. The former is obtained when an electric current is applied to a circuit containing two current electrodes A and B (Fig. 2); one of the electrodes must be placed in the drillhole. In single electrode logging (resistance log) the drop in potential between the electrode in drillhole and that on the surface is measured. Alternatively in multiple electrode logging (resistivity log) the field set up by the two current electrodes A and B is investigated by a pair of potential electrodes M and N, of which at least one must be in the drillhole.

The spontaneous potential test measures the electric current generated spontaneously in the drillhole. The electrode arrangement is the same as for the single electrode resistance log, but no external source of current is applied.

Considering the ground, including the drillhole, to be an infinite, homogeneous and isotropic medium, a formula espressing the apparent resistivity can be obtained. The formula is exactly the same as that used in the earth resistivity method (formula 1). Therefore the spacing of the electrodes gives an effective control over the radius of investigation. The generally accepted spacings used in two electode arrangements are:

micro normal	1 inch
short normal	10 or 16 inches
long normal	40 or 64 inches

in three electrode arrangements:

micro lateral	1 inch
lateral	16 ft.

The radius of investigation of the single electrode arrangement is approximately double the diameter of the probe. Therefore the effect of the drillhole fluid will be highest for the single electrode arrangement and very small for long normal or long lateral.

The resistivity of the solid material in the rock is in general very high and electric current flows through the rock largely by ionic conduction in the fluid saturating the pore spaces. The conductivity of a rock thus depends upon the amount of fluid in interconnected pore spaces and upon the conductivity of that fluid.

It has been found that the resistivity of a water saturated formation increases when

(1) the salinity of the formation water decreases

or (2) the porosity of the formation decreases, or both.

Formula 2 relates these factors.

Fig. 3 shows the resistivity of a formation as a function of porosity and type of water. The cross-hatching represents the areas of the chart corresponding to clay and to fresh water aquifers.

Two processes generate electrical potentials spontaneously in a water or mud filled drillhole. The streaming potential results from the flow of fluid from the drillhole into the rocks or vice versa. The electro chemical potential is a function of the ionic activity of the water or mud in the drillhole and the water in the adjacent formation.

The flow of current takes place in the drillhole where it intersects a contact between clay or shale and sand or sandstone (Fig. 4). The current flowing in the drillhole will produce the difference of potential. The potential opposite sandstone will be lower than that opposite clay and shale if the salinity of the formation water is higher than the salinity of the mud. When the mud is very saline the picture will be reversed.

The combined use of spontaneous potential and resistance or resistivity data makes possible the identification of the lithology of the rocks penetrated by the drillhole and also provides an accurate and detailed record of the boundaries between the different lithological units. A distinctive feature on any one of the logs provides a practical basis for correlation between drillholes and also any change in salinity of the fluid enclosed in the rock can be found and followed by an accurate determination of porosity and salinity.

B. Radiation method

Of the four forms of radiation, alpha, beta, gamma and neutron emission, only the last two are used in drillhole logging; the others are characterised by very small depth penetration.

Gamma rays are electromagnetic radiation emitted during nuclear transitions. Neutrons are produced by bombardment of light nuclei with alpha particles emitted by a naturally occurring unstable nucleus.

Most rocks and soils contain small quantities of radioactive material, mainly potassium, uranium and thorium. The radioactivity of clay and shale is several times that of the other types of common sedimentary rocks and sands. Gamma ray logs can therefore be used to distinguish clay from other types of rocks or soils. The increase in gamma ray intensity is nearly proportional to the clay content of a rock. Fig. 5 shows relative radioactivities of various sedimentary rocks. Gamma logging can be carried out in an open or a cased drillhole whether full of mud or empty.

When a source of gamma ray is placed in a hole, the radiation is scattered by the electrons in the surrounding formation. Part of the scattered gamma rays will reach a detector placed at a distance of about 1 ft above or below the source, connected to a recorder at the surface. These rays are called "gamma-gamma" to differentiate them from the gamma rays emitted by the formation. The higher the density of formation the smaller the number of "gamma-gamma" rays reach the detector.

When a source of fast neutrons is placed in a drillhole the neutrons emitted will collide with the nuclei of atoms and as a result lose energy and speed. The loss of energy due to collision with hydrogen nuclei is much greater than when the collision is with nuclei of other elements. Therefore the number of slow neutrons is propertional to the number of hydrogen atoms present close to the emission source. The measurements above the water table will give the moisture content of the formation, while the measurements below the water table give the porosity. Logging can be carried out in a cased drillhole.

C. Velocity logging

The velocity of sound waves through a formation is dependent upon the formation porosity and on whether the pores are filled with air or fluid. Fig. 6 shows a graph of velocity against formation porosity.

The relation of seismic velocity to porosity can be expressed by formulae 3.

D. Temperature logging

The interior of the earth is very hot but its surface temperature is only slightly affected by this heat. The thermal conductivity of rock is a characteristic property of the rock type and is generally very low. Igneous rocks show much higher thermal conductivity than sedimentary rocks.

The temperature in a shallow hole is equal to the mean annual air temperature; lower down the temperature increases with a temperature gradient of 1° F per 50 to 100 feet of depth depending on the thickness of sedimentary rocks above the igneous basement. Superimposed on the general gradient are small changes in temperature due to the thermal conductivity of the rock in the wall of the hole. For accurate determination of the relative thermal conductivity of the beds penetrated by the drillhole, the log must be made several days after drilling fluid circulation has been stopped and a temperature equilibrium between the drillhole fluid and the wall rock has been established. The work can also be done in a cased hole.

E. Drillhole diameter logging (caliper log)

The rocks forming the walls of the drillhole are worn away by the circulation of the drilling mud. In rock formations which are dense, hard and insoluble the diameter of the hole closely approximates bit size. Sandy or silty clay and shale are easily eroded, especially when they are hydrated by the drilling fluid. Mud penetrates into permeable formations to some extent and a mud cake, which supports the wall, forms where the drillhole intersects such formations. Where such a cake has formed the hole will be equal in size to the drill list, or smaller.

All the above mentioned methods give additional data for the lithological identification of a bed and determination of its physical parameters, and enable drillholes to be correlated.

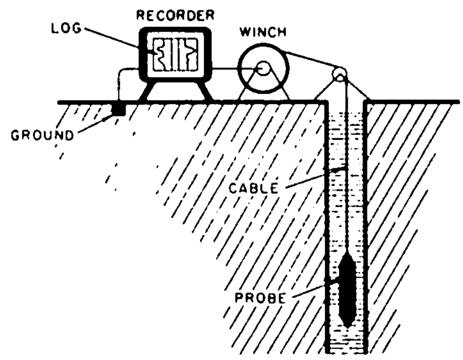


FIG. 1.—Well-Logging Setup.

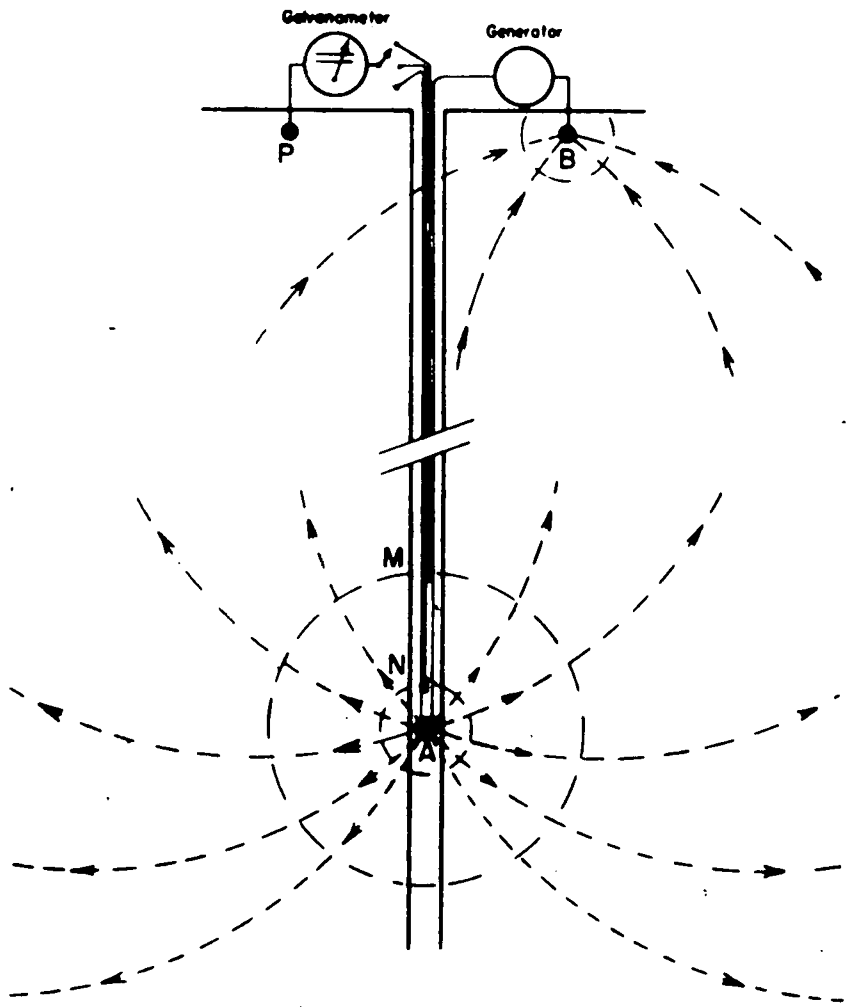


Fig. 2

$$\rho_a = \frac{V}{I} 4\pi \frac{AM \times BM}{BM - AM} \quad \text{..........(1)}$$

ρ_a = apparent resistivity in ohm-metres

V = potential difference (volts)

I = current (amps)

AM, BM = distances (as fig. 2) in metres

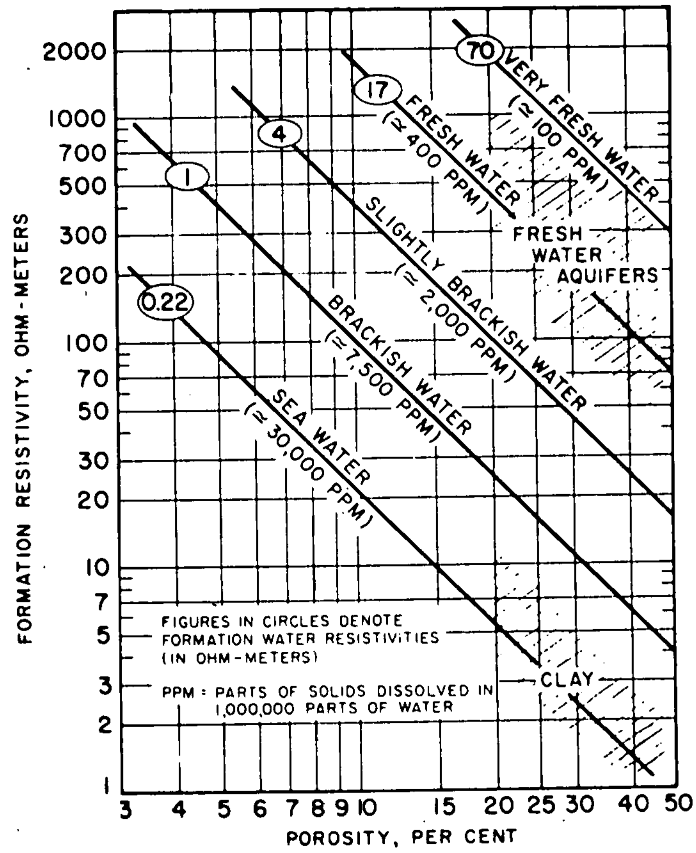


FIG.3.--Resistivity of a Formation as a Function of Porosity and of Resistivity of Formation Water

(after GUYOD)

$$Ro = F\,Rw \quad \text{.........(2)}$$

$$F = \frac{1}{p^m}$$

where

Ro = resistivity of saturated rock
Rw = resistivity of the electrolyte
F = formation factor
p = porosity
m = cementation factor
1.3 for clean sands
2.0 for consolidated sandstone

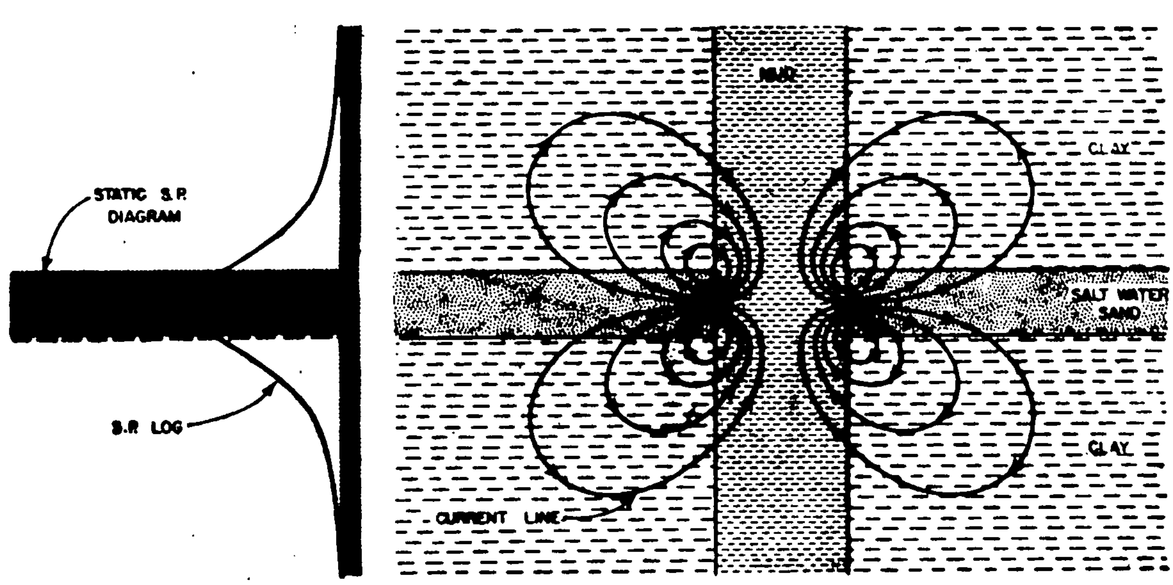


FIG. 4. **Schematic representation of potential and current distribution in and around a permeable bed**

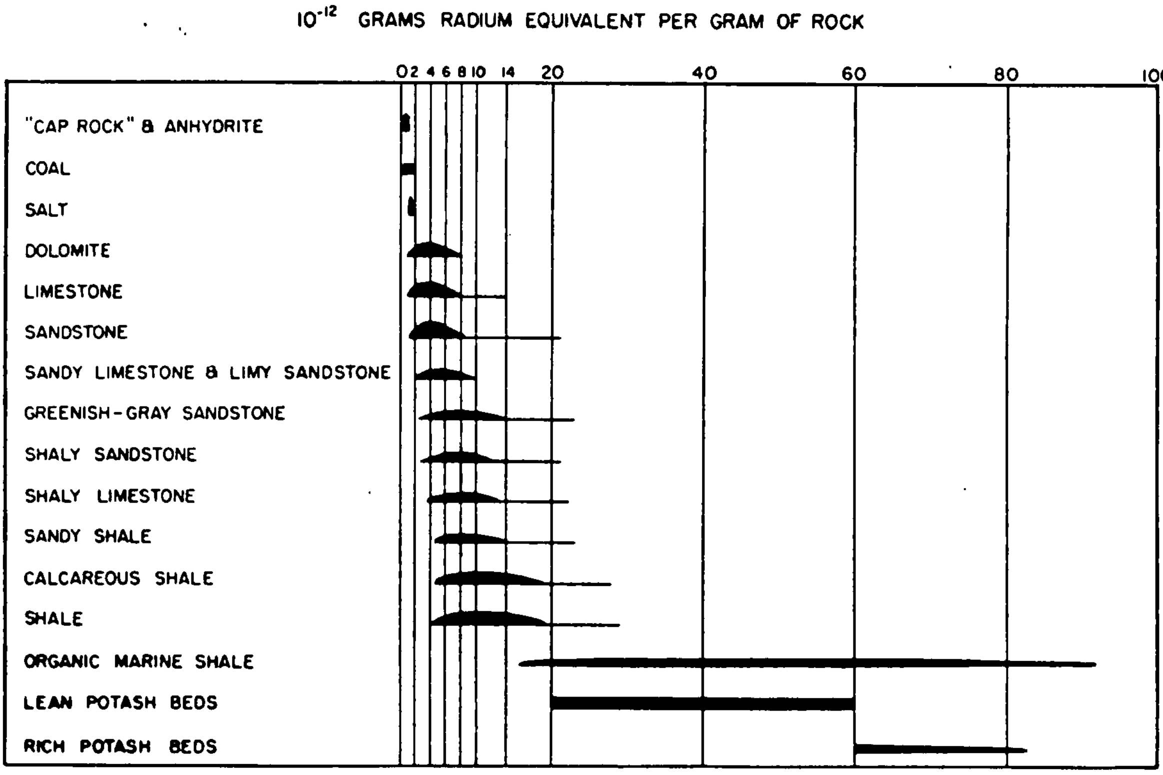


FIG. 5 Diagram showing the relative radioactivities of various sedimentary rocks

(after RUSSELL)

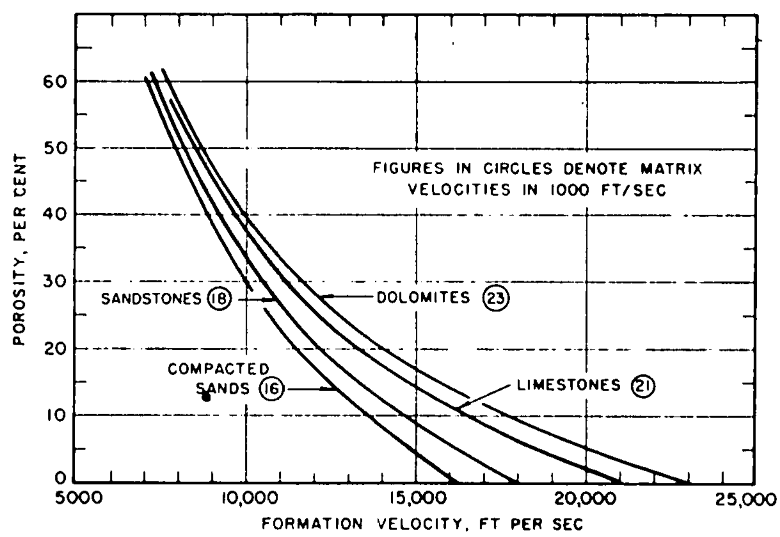


FIG. 6.--Porosity of Compacted Formations Filled with Water, as a Function of Their Acoustic Velocity.

(after GUYOD)

$$p = \frac{1/V - 1/Vm}{1/Vf - 1/Vm} \quad \ldots\ldots\ldots\ldots\ldots (3)$$

p = porosity

V = wave velocity in formation

Vf = wave velocity in the fluid

Vm = wave velocity in the solid matrix

GRAVITY METHOD

by
W. Wiebenga, Bureau of Mineral Resouces, A.C.T.

Engineers, geologists and hydrologists are frequently required to look at or judge fravity results and gravity interpretations. Often their knowledge of gravity work is limited and they do not understand what the results mean. It is hoped that this short lecture will fill this gap.

The definition of gravitational force follows from Newton's Law of Attraction.

$$F_{\text{gravity}} = G\frac{m_1 m_2}{r^2}$$

in which G is the graviatational constant, equal to 6.67×10^{-8} or, approximately, to $2/3 \times 10^{-7}$, if all quantities are expressed in c.g.s. units.

Units

If F is the force which produces an acceleration a in a mass m then:

$$F = ma$$

In the c.g.s. system F is measured in dynes and m in grams; the units of acceleration are therefore dynes/unit mass. However a more usual definition of acceleration is velocity change per unit time, with units, in the c.g.s. system, of cm/sec^2. An acceleration of 1 cm/sec^2 is termed one gal, and the milligal (=1/1000 gal) is the unit generally used in gravity surveys.

The acceleration due to gravity measured at the earth's surface is approximately 980 gals. At present the accepted International Gravity Formula, giving the theoretical gravity at a point at latitude ϕ on the earth's surface, is that published by Cassinis in 1930.

$$g_\phi = 978.049\ (1 + 0.0052884 \sin^2\phi - 0.0000059 \sin^2 2\phi)\ g_o.$$

Tables are available giving $g\phi$ for any value of ϕ.

A more recent gravity formula, derived by Heisplanen introduces a longitude term, because the earth may be represented by a triaxial spheroid.

Gravity Measurement

Variations in gravitational field may be mapped using a gravitymeter, pendulum or torsion balance. The first of these is generally used in gravity exploration. In principal the gravity meter is a very sensitive spring balance in which the elongation of the spring is proportional to the gravitational field. The relative value of the gravitational field at a number of stations is measured. The relative elevation of the stations must be known to 0.1 of a foot and the gravity readings corrected accordingly.

Elevation Correction

Any level may be chosen as datum for elevation corrections but sea level is the datum generally used.

The "free air gravity" is the gravity value reduced to sea level by use of the formula

$$go = g\,(R + h)^2 / R^2$$

i.e. $$go = g\left(1 + \frac{2h}{R}\right)$$

where h is the elevation, R the earth's radius and g the observed gravity. The "free air correction" is thus 2hg/R or approximately 0.094 mgal/ft.

The "Bouguer Correction" removes the effect of an infinite slab of material between the station and the sea level datum. It equals $-2\Pi G\sigma h$ or approximately -0.01250 mgal/ft where σ is the density of the slab.

The free air and the Bouguer correction are usually combined in a single elevation correction: $(0.094 - 0.0125\times\sigma)$ mgal/ft.

TOPOGRAPHIC CORRECTIONS

In many gravity surveys the terrain in the vicinity of the station is sufficiently flat and an elevation correction is all that is needed. If hills or valleys are close by, their effect has to be removed by a terrain correction which is positive for both hills and valleys. A topographic survey of the station surroundings is required, and the corrections are estimated by use of templates consisting of circles and radial lines.

A map correction is the same in principal as a terrain correction but applies to the area outside the tarrain correction zone. With a map correction the effect of mountain ranges or oceans is removed.

LATITUDE CORRECTION

For this correction the values of the international gravity formula are used.

Corrrections are appliediin order to isolate the gravity values which are dependent only on variations in density in the subsurface, there by making the anomalies better visible on a map.

GRAVITY MAPS

Most gravity maps used in exploration are in the form of Bouguer anomaly maps and residual anomaly maps.

The Bouguer anomaly may be defined as:

Observed gravity + free air correction - Bouguer corrections + topographic correction - theoretical gravity.

Residual anomaly may be defined as:

Bouguer anomaly - regional gravity value. Sometimes second derivative anomalies are used instead of residual anomalies.

In gravity exploration maps the contour interval generally used is one milligal or part of a milligal.

Density

For the Bouguer and terrain corrections the density of the surface layers is required. If it can be assumed that there is no relation between the topography and the density of the surface layers as statistical method of density determination may be used. A large sample of gravity stations is selected and the Bouguer anomaly of each station is computed using a series of densities $\sigma_1, \sigma_2, \ldots \sigma_n$ (3 or 4 values are usually sufficient). Then correlation factors are computed between Bouguer anomalies with these densities and the station elevations.

The correct density for corrections is the one for which the correlation factor is zero.

If the topography is related to the Bouguer anomaly, viz. if erosion was strongly influenced by geological structure, then other ways of determining or guessing the surface density should be used.

For density estimates of deeper formations either bore hole information (cores or logs) or an intelligent guess is used.

Sometimes seismic refraction work may be applied because there is an empirical correlation between seismic velocity and density.

ACCURACY

The accuracy of Bouguer anomalies is affected by random errors in observed gravity (Δg mgal), in elevation (Δh mgal) and in latitude (Δl mgal), resulting in a random error of $\sqrt{\Delta g^2 + \Delta h^2 + \Delta l^2}$
In hydrological surveys the random error in Bouguer anomalies is usually ± 0.1 to 0.2 mgal.

A systematic error is introduced by the error in density used in elevation corrections : $2\pi G h \Delta\sigma$

For $\Delta\sigma$ = 0.2, and maximum elevation difference of 100 ft this amounts to ± 0.26 mgal.

In an accurate survey with smoothly distributed errors Bouguer anomalies in excess of 0.3 or 0.4 mgal may be considered significant.

An anomaly of 0.4 mgal with a density contrast 0.5 between overburden and bedrock, corresponds to 0.4/0.006 = 67 ft overburden. However, the absolute error in this estimate may be great, say 30% or more, depending on how closely our assumptions approach the true situation.

REGIONAL AND RESIDUAL GRAVITY MAPS

Many areas have deeper seated structures causing variations in gravity at the surface which are much larger in areal extent than the features of interest.

On a gravity map the gravity pattern caused by the deep structure (named "regional") is superimposed on the detailed ("residual") gravity pattern caused by shallow features.

In shallow basins or deltas the interest is in the shallow features. To improve the "readability" of the gravity map, or to use the map for depth estimates, the regional values have to be subtracted from the computed gravity values. The remainder then gives a residual gravity map.

Of the many ways of obtaining a regional value a simple method is to average the values around a circle. The residual gravity is then the observed value at the centre of the circle minus this average. The main problem is the choice of the radius of the circle. This must be large enough so that the smaller anomalies are not filtered out but not so large as to indicate anomalies from deeper sources.

Another way of bringing out shallow features is the use of second derivative gravity maps, but these will not be discussed here.

A properly constructed residual gravity map can be used to make depth estimates as follows:

call the residual gravity where bedrock comes to the surface go, and at any other place in the basin g^1, then

$$g = g^1 - go = 0.013\ h$$

where is the density contrast and h the depth in feet.

The assumption made is that the bedrock underneath the station is relatively flat, and not located near a sharp sub surface hill, valley or large fault. As an example, for $g = 1$ mgal, $\Delta\sigma = 0.4$, $h = 200$ ft.

Application of the gravity method

(1) To determine the position of shallow sub-surface water courses, valleys, and in general, the bedrock configuration. Weathered bedrock may have a density close to that of the overburden material. Hence, bedrock, in this context, really means unweathered bedrock.

(2) The method may also be used to determine the shape and approximate depths of deeper basins.

MAGNETIC METHOD

by

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General

The magnetic method is similar to the gravity method in that both are based on potential fields. However, gravity is only concerned with positive masses, magnetism is concerned with positive (north seeking) and negative (south seeking) poles.

This makes the interpretation of magnetic data more complicated. However, magnetic methods are largely used as a qualitative, cheap reconnaissance tool.

Magnetite, by virtue of its high magnetic susceptibility and wide distribution in rock, is the most important of the ferro-magnetic minerals, and is largely responsible for the induced and remanent (or residual) magnetization of rocks.

The magnetic fields strength unit is oersted which equals one dyne per unit pole.

The field strength H due to a pole of strength P is:

$$H = \frac{P}{H + r^2}$$

where H is permeability, and r distance from pole. The unit is used in exploration is gamma (γ) = 10^{-5} Oersted.

The intensity of induced magnetization may be considered to be the induced pole per unit area along an area normal to the inducing field (H). These factors are related by the formula I = k H, in which k is a proportionality factor known as the magnetic susceptibility.

The magnetic induction B inside a material with permeability due to an external field H is:

$$B = H + 4\pi I = (1 + 4\pi k)\ H = \mu H \text{ and}$$

$$\mu = 1 + 4\pi k.$$

The magnetic moment of two equal poles of opposite polarity, distance L apart, and pole strength P, is PL.

The following data (Birch et al, 1950) indicate the magnetic susceptibilities (k) of ferro-magnetic minerals in fields of 0.5 to 1.0 oersted in e.m.c.g.s. units:

> magnetite 0.3 to 0.8, phyrhotite 0.01 to 0.03, ilmenite 0.03 to 0.04, ilmenite free of magnetite 0.0006, and specularite 0.003. The wide range in the magnetic susceptibility of rocks is shown by the fact that 80 per cent of basic rocks have values between 0.0001 and 0.004, granites and allied rocks between 0.001 and 0.004 and 75 percent of the sedimentary rocks less than 0.0001.

Remanent (or residual) magnetization can be considered as the existing pole strength per unit area (or magnetic moment per volume) due to an inducing field reduced to zero. The remanent magnetisation if igneous rocks was acquired when the rocks cooled from the Curie temperature in the prevailing earth's field. The ratios of remanent to induced magnetization usually exceeds 20 for gabbro, is 0 to 20 for dolerite, 0 to 9 for basalt and 0 to 6 for ganite porphyry. These values illustrate the importance of remanent magnetisation.

For a discussion on instruments and field techniques reference is made to current textbooks. Equipment developed after the war includes airborne magnetometers, and fluxgate and nuclear magnetometers for land use.

Field observations are generally corrected for diurnal fluctuations and temperature. Data are corrected for latitude. Sometimes a regional correction is combined with a latitude correction. Terrain corrections are rarely made : only if observations are taken close to escarpments or in V shaped valleys. Subsequently contour plans are constructed.

Interpretation

The main purpose for doing magnetic work is determination of the basin boundary and shape of the basin, to make rough depth estimates, and to disclose geological features such as subsurface valleys or hills, depressions or uplifts, anticlines, faults, escarpments, etc.

As in gravity the interest may be focused on the smaller anomalies associated with shallow basement or near-surface features. If these smaller anomalies are over shadowed by large anomalies, a regional and residual map are made. The same techniques are applied as in gravity interpretation.

An example may illustrate how susceptibility change may result in large anomalies. The vertical field above a semi-infinite slab of magnetic material is $2\pi I$. Assume a lateral change from a granite ($K = 0.003$) to an andesite ($K = 0.013$) in an earth magnetizing field of 0.5 oersted then the field change would be

$$2\pi \times 0.5 \times (0.013 - 0.003) \times 10^5 = 3000 \pm \text{gamma}$$

In addition to anomalies caused by polarization of the present earth field, many anomalies originate from remanent magnetism in rock bodies of which the field vectors do not coincide with the present field.

From the point of view of interpretation two types of basement may be distinguished.

(a) A basement with low to moderate susceptibility and anomalies of intra-basement origin (e.g. volcanic pipes, igneous intrusions dykes, etc.)

An approximate depth estimate of the magnetic anomalies will give at the same time a depth estimate of the basement. With this technique a volcanic pipe may be represented by a dipole, a dyke by a magnetised sheet and an intrusive body by an agglomeration of evenly spaced bodies.

Often the maximum magnetic gradient or the curvature of the anomaly is taken as a measure for depth.

If sufficient drill hole - seismic - or gravity control is available it is possible to make a more or less reliable statistical correlation between basement depth and the gradient - or curvature parameters as derived from the magnetic contour map. This correlation can then be used to make further depth estimates.

(b) The second type of basement consists of rock in which the susceptibility does not vary appreciably e.g. metamorphic gneiss or schist, granite, etc.

Magnetic anomalies by structural features such as faults, anticlines, synclines, basement hills, valleys or escarpments. By fitting a probable (or possible) geological structure the approximate depth to the structure can be fairly reliably estimated.

NOTES ON A TEST WITH ISOTOPES I131 AND TRITIUM AS GROUND WATER TRACERS

by
W. Wiebenga, Bureau of Mineral Resources, A.C.T.

Radio isotopes I131 and Tritium were used to determine aquifer porosities and permeabilities in a series of pumped bore tests. If reasonable values for porosity can be assumed the method can also be used to estimate the thickness of an aquifer. The direction and rate of flow of natural ground water was also determined by a free flow test. The following is the description of one of the experiments.

Isotope used was I131. Laboratory work carried out at the A.A.E.C. confirmed that there was virtually no loss of activity when percolated through the soil.

Further, the advantage of I131 is:

(1) Low energy gamma rays (0.36 Mev)

(2) Half life 8 days, long enough for project but short enough to be self disposing.

(3) Readily available.

(4) Reasonably low health hazard.

Site preparation

Three bores were drilled with a Proline drill in line with a pumping bore (to 33 ft, 12 ft to water) : at 60 ft from pump, used for dosing, and at 50 and 30 ft used as observation holes (Fig. 1). Holes tended to collapse below ground water level. A 3 ft 2" galvanized iron pipe with $\frac{1}{8}$" holes connected to a 20 ft 2" galvanized pipe were driven into the ground. Holes were cleaned with rubber piston, sand pump and compressed air. Nevertheless there was no free circulation of ground water through the observation holes.

This proved to be an advantage because this made it possible to obtain accurate gamma ray profile of the bore through the casing.

Procedure

A glass container with I131, Tritium and potassium iodide carrier was broken in the dosing bore and washed into the surrounding aquifer by adding a few gallons of water.

The pump had been started a few days before, pump output about 30,000 g.p.h. The observation bore holes were monitored for I131 by waterproof scintillation counters, consisting of a No. 1 crystal and photo multiplier, feeding pulses into a portable rate meter. The pump output was monitored by a large scintillometer. Scintillometers were calibrated at the A.A.E.C. Research Establishment at Lucas Heights, N.S.W.

All readings were corrected for background and decay. For each experiment an isotope balance was made up by comparing the total activity at the pump outlet (integrated count rate/time curve) with the amount of I131 injected in the closing bore. Recoveries varied between 70 and 100%.

Gamma ray profiles in bores 2 and 3 were taken at various times. Figures 2 and 3 show curves which were obtained by plotting the count rate at constant depth, and figures 4 and 5 show the maximum count rate against time.

<u>Porosity</u>

Using the relationship of Halevy and Nir (1962, J. of Geoph. Res., 67 (6), 2403-2409) and figure 6 to estimate the travel time (use midpoint half peaks) :

$$(\text{distance travelled})^2 = \frac{\text{Pumprate x travel time}}{\pi \text{ x porosity x aquifer thickness}}$$

$$\text{or} \quad r^2 = \frac{Q \times t}{\pi \times \underline{P} \times b}$$

gives for our test $\underline{P} = 0.32$

Differentiation of the above relation gives also the velocity at distance r from the pump.

$$2r \; dr = \frac{Q \; dt}{\pi \underline{P} \; b} \qquad \text{or}$$

$$Vr = \frac{Q}{2 \pi r \underline{P} b} = \frac{k}{r} = \frac{118}{r}$$

where the constant k has the dimension $ft^2 \; hour^{-1}$ and the interval velocity between r_2 and r_1 is :

$$Vi = \frac{k}{r_1 - r_2} \ln \frac{r_1}{r_2} = \frac{119}{r_1 - r_2} \ln \frac{r_1}{r_2}$$

With this experiment it is also possible to determine the aquifer thickness if the porosity is approximately known.

Figure 7 gives the depth of greatest activity at holes 2 and 3; the pattern for both holes is similar. The data give the interval velocities which also can be used to make estimates for k.

Figure 8 shows the interval velocities obtained from Fig. 7 plotted as a function of distance, together with the relation V = 118/r.

<u>Permeability</u>

Assuming that the streamlines for greatest activity are about parallel to the ground water profile, the data of Fig. 7 can be used to compute the permeability from the relationship.

$$\text{Permeability} = \frac{\text{cross section area x velocity flow}}{\text{gradient of flow}}$$

For cross section = 1 sq. ft, and porosity = 0.32

this reduces to

$$\text{Permeability} = \frac{0.32 \text{ x velocity flow}}{\text{gradient}}$$

The data of fig. 7 give a mean permeability of 99. cu.ft./hr per 1 ft/ft.

Under conditions of similar gradient the flow velocity will only depend on the aquifer permeability.

Fig. 9, derived from fig. 7, gives the permeability as a function of velocity for a constant gradient of 1ft/ft. Fig. 9 can be used to compute the permeabilities of points 1 to 10 on fig. 7. The mean permeability based on these figures is 77 cu.ft/hr per 1 ft/ft.

Taking the permeability as 77, the average gradient between bore 2 and 3 as 0.016 ft/ft, the distance from the pumping bore as 40 ft, and the aquifer thickness as 20 ft, the expected pump flow rate is:

Permeability x gradient x 2 r x thickness x 6.25 = 37000 g.p.h.

Free flow test (Fig. 10)

Fluorescein added to injection hole and detected in quantity in hole 6 after 42 hours.

I131 added to injection bore, 110 activity detected after 40 hours, but slight activity in hole 6 after 71 hours. Hence, rate of flow in direction of bore 6 is 5 x 12/41 = 1.5"/hour.

The injection bore became blocked and was cleared about $3\frac{1}{2}$ to 4 days later. Then the pump was started 88 hours after injection and the main activity was monitored at the pump 88 + 26 = 114 hours after injection; or 26 hours after pumping started. A small activity due to isotope going into the formation immediately after injection was recorded 10 hours after pumping started or 88 + 10 = 98 hours after injection. (See fig. 11)

$$60 = A\sqrt{26}$$

$$X = A\sqrt{10}$$

where A is a function of flow rate, porosity, aquifer thickness, and the figures beneath the square root are the times taken for the two bodies of activity to reach the pump.

$$X = 60\sqrt{\frac{10}{26}} = 38 \text{ ft.}$$

This means that the small body of activity was about 38 ft from the pump when the pump started, and in the 88 hours since injection it must have travelled more than 60 - 38 = 22 ft, or velocity of ground water is in excess of 22/88 ft/hr = 3 in/hr.

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BASIC PRINCIPLES OF WATER FLOW IN POROUS MEDIA

by
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1. Potential difference

The flow of water from place to place in a porous medium occurs in response to a difference in potential. The absolute magnitude of the potential is unimportant, because the rate of water flow depends upon the gradient of potential, which is the difference in potential, divided by the distance between the two points in question. However, it is convenient to have some arbitary datum of potential, for purposes of calculation.

As a definition of potential, we may say that the total potential of the water is comprised of three components, tht are important in real situations, namely the pressure, p', the gravitational potential, ρgz, and the osmotic potential, π'.

Therefore, $\Phi' = p' + \rho gz + \pi'$ ...(1)

and the units used in Equation (1) are absolute c.g.s. units (ergs Cm^{-3}). The vertical height, z, may be measured from any convenient datum to the point at which the potential, Φ', has to be calculated, and is positive in the upwards direction.

Differences in osmotic potential will occur only in a porous system which possesses a membrane that impedes molecular diffusion of solutes in the water. This is an important phenomenon in the absorption of water into plant roots. It can be neglected in the context of groundwater flow.

It is also convenient to change the scale and the units as expressed in Equ. (1) and write

$$\Phi = p + z \qquad ...(2)$$

in which the potential or hydraulic head has the units cm or foot.

Suppose we study the drainage of water-logged land, by tile-drains, as depicted in Figures 1 and 2.

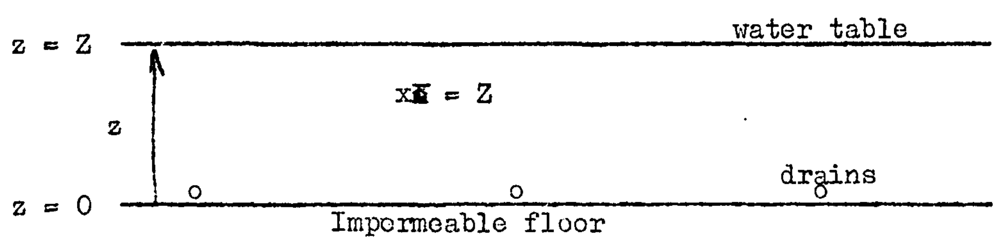

Figure 1. Flat water table.

In Figure 1, the drains have been blocked to prevent outflow, the water table is flat, and the potential is everywhere Z, the height of the water table above the impermeable floor as datum, because

$$\Phi = p + z = (Z - z) + z = Z \qquad ...(3)$$

In Figure 2 are shown the streamlines of flow towards one drain, when the drains flow freely. Flow to all other drains is the same as, or the mirror-image of this flow. As can be seen, the streamlines converge upon the drain, and the potential becomes larger against the direction of flow. These relationships can be expressed by the formulation of Darcy's Law thus,

$$\underline{v} = - K \text{ grad } \Phi \qquad ...(4)$$

in which is the volume of water flowing per second through, and in a direction normal to a unit area, K is the permeability or hydraulic conductivity and grad Φ is the gradient of the potential.

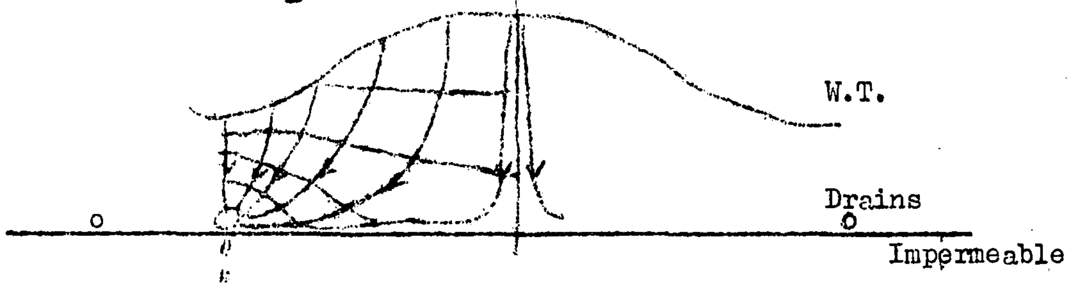

Figure 2. Flow to tile drains.

Equation (4) is just a generalisation of the expression

$$Q = KAt\,\frac{\Phi_2 - \Phi_1}{L} \qquad \ldots(5)$$

in which Q is the quantity of water which flows through a cylindrical volume of cross-section A and length L. The time of flow is t and the difference in potential from one end of the cylinder to the other end is $(\Phi_2 - \Phi_1)$. It is understood that the streamlines are all parallel to the axis of the cylinder.

Such flow is steady state flow, because the quantity of water entering at one end of the cylindrical volume equals the quantity quitting at the other end, no matter how short an interval of time may be taken. Steady state flow is described by a particular equation of continuity, which generally expresses the law of the conservation of matter. The general equation of continuity is

$$-\operatorname{div} \underline{V} + \text{sources} - \text{sinks} = \frac{de}{dt} \qquad \ldots(6)$$

For steady state flow, which implies no change in the water content, e, of the porous medium, and assuming the absence of sources or sinks for water,

Equation (5) becomes

$$-\operatorname{div} \underline{V} = 0 \qquad \ldots(7)$$

Combining Equations (4) and (6), the general equation for steady state flow is derived,

$$\nabla^2 \Phi = 0 \qquad \ldots(8)$$

or, written in full,

$$\frac{\partial^2 \Phi}{\partial x^2} + \frac{\partial^2 \Phi}{\partial y^2} + \frac{\partial^2 \Phi}{\partial z^2} = 0 \qquad \ldots(9)$$

To obtain Equation (7) or (8), it is necessary to assume that K is a constant so that

$$\frac{\partial K}{\partial x} = \frac{\partial K}{\partial y} = \frac{\partial K}{\partial z} = 0$$

Equation (7) or (8) is Laplace's Equation, which, in principle, can be solved for Φ as a function of the co-ordinate x, y and z, when values of Φ can be specified over all of the boundaries of the system.

To summarise: the concept of flow in response to a potential gradient, embodied in Darcy's Law (Equation 4), when combined with the principle of conservation of matter (Equation 6) has led to the Laplace Equation (Equation 8) as a description of steady state flow. Very many solutions of the equation are known for specified boundary conditions.

2. Permeability or hydraulic conductivity.

The permeability, K, (usually termed hydraulic conductivity in the literature of soil science) defined by Equs. (4) or (5) can vary greatly in magnitude depending upon the aquifer material. Table 1 shows representative values.

Table 1. Representative values for hydraulic conductivity.

Material	K in cm/sec	K in metre/day	K in ft/sec	K in ft/day
Clay	1×10^{-6}	8.7×10^{-4}	3.3×10^{-8}	2.8×10^{-3}
Fissured Clay	5×10^{-3}	4.3	1.6×10^{-4}	14
Structured loam	2×10^{-3}	1.7	6.6×10^{-5}	5.6
Silty loam	1×10^{-4}	0.87×10^{-1}	3.3×10^{-6}	0.28
Fine sand	1×10^{-3}	0.87	3.3×10^{-5}	2.8
Coarse sand	7×10^{-3}	6.1	2.3×10^{-4}	20
Gambier limestone	4×10^{-2}	35	1.3×10^{-3}	115
Peat	1×10^{-3}	0.87	3.3×10^{-5}	2.8

The hydraulic conductivity depends upon the pore size distribution and upon the total porosity of the material. The larger the total porosity, the greater is the amount of space available for water flow, but even more important is the size of the largest pores available for water flow. The influence of pore size can be seen qualitatively from the following argument.

By Poiseuille's Equation, the rate of flow through a narrow pipe of radius R is

$$Q = \frac{\pi R^2 . R^2}{8\eta} . \frac{\Phi_2 - \Phi_1}{L} \qquad \ldots(10)$$

where η is the viscosity of the water, and L is the length of the pipe over which the potential difference $(\Phi_2 - \Phi_1)$ is maintained. If, instead of one pipe, ten smaller pipes were substituted of such size that the cross-sectional area for flow would remain the same, i.e.

$$\pi R^2 = 10\pi r^2$$

where r is the radius of each smaller pipe, the flow rate in each small pipe would be

$$q = \frac{\pi r^2 . r^2}{8\eta} . \frac{\Phi_2 - \Phi_1}{L}$$

and the total flow would be

$$10q = \frac{10\pi r^2 . r^2}{8\eta} . \frac{\Phi_2 - \Phi_1}{L}$$

$$= \frac{\pi R^2 . r^2}{8\eta} . \frac{\Phi_2 - \Phi_1}{L}$$

The ratio of the flow rates would be

$$\frac{10q}{Q} = \frac{r^2}{R^2} \qquad \ldots(11)$$

demonstrating, by a qualitative extension of the argument, that the flow rate through a pore is proportional to the square of its mean diameter.

Childs and Collis-George (1950) initiated attempts to calculate the permeability of a porous medium from its measured pore size distribution and Marshall (1958) gave the expression

$$K = 2.7 \times 10^2 \, \mathcal{E}^2 \, n^{-2} \left\{ h_1^{-2} + 3h_2^{-2} + 5h_3^{-2} + \ldots \right.$$

$$\left. \ldots + (2\,n-1)\, h_n^{-2} \right\} \qquad \ldots(12)$$

for the hydraulic conductivity of a porous medium. K is in cm/sec when the suctions h_1, h_2, etc. are measured in cms. In deriving this equation, the total drainable porosity, $\mathcal{E}$, is divided into n equal parts by volume, each part having a pore size characterised by a suction, h, at which those pores drain of water.

Since the rate of flow of fluid through a porous medium depends upon the viscosity of the fluid, it is sometimes convenient to use the intrinsic permeability coefficient, k, which is related to the hydraulic conductivity thus

$$(\rho g/\eta)\, k = K$$

where ρ is the density of the fluid.

3. Pore size distribution, and storage coefficient.

In considering drainable porosity, it is helpful to begin with some definitions of terms.

The suction, stated without definition in Equ. (12) is related to the total potential of the water in the following way. If we consider an element of the soil or aquifer above the water table, the pressure of the water in its pores, p' of Equ. (1), is less than atmospheric pressure. The general equations (1) or (2) still hold, but Equ. (2) is often written as

$$\Phi = -h + z \qquad \ldots(13)$$

in which h, the suction, is a positive quantity, and, for an equilibrium soil water profile, is equal to the height of the element of soil above the water table.

The water content of the soil, w, is related to the suction as shown in Fig. 3, a diagram often spoken of as the moisture retention curve.

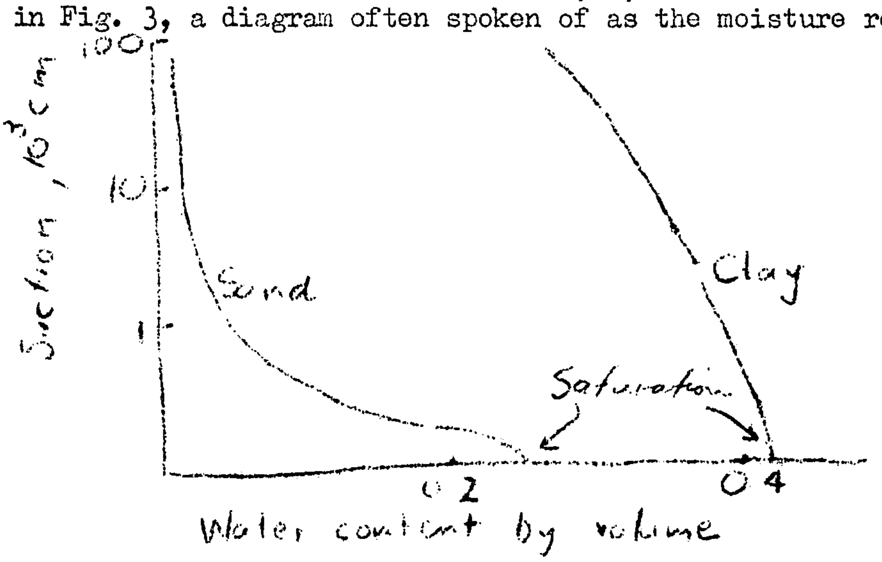

Fig. 3. Moisture retention curves, for sand and clay.

The storage coefficient of an unconfined aquifer, defined as the yield of water for unit fall in the water table height, is related to the moisture retention curve. Examining Fig. 4, which is an enlargement of conditions near saturation, and with suction plotted on a linear scale, we see that the storage coefficient is given by the area shaded, divided by the change in height, Z.

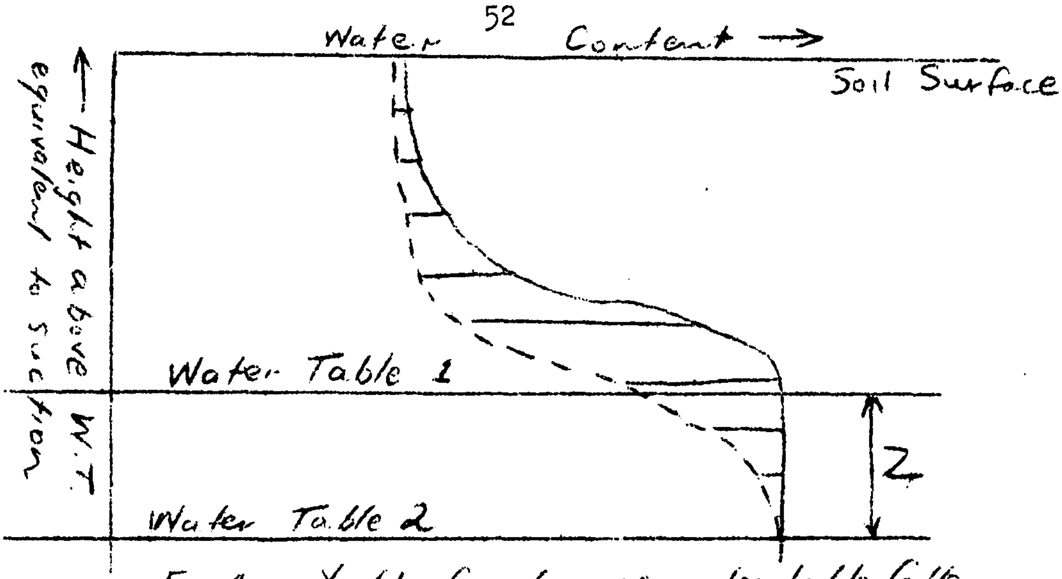

Fig. 4 Yield of water as water table falls.

Usually, for short term pumping, the yield is less than the storage coefficient which is the theoretical maximum possible yield, because a long time is required for complete drainage, to a new equilibrium, of all pores affected by the draw-down.

Representative values for storage coefficient are shown in Table 2.

Table 2. Storage coefficient, S, for a variety of aquifer materials, in unconfined conditions.

Material	Storage Coefficient
Coarse gravel	0.15
Sand	0.08
fissured calcareous clay	0.05
clay	0.001
Gambier limestone	?

When the aquifer is confined, the storage coefficient is much smaller than the relatively large values shown in Table 2. The yield of water for unit fall in the piezometric head has to come from reversible compressibility of the aquifer material and compressibility of the water itself.

4. Transmissivity

Under natural flow conditions, the hydraulic conductivity alone is not sufficient to specify the volume of water flowing under a given potential gradient. We need to know the extent of the aquifer. The transmissivity of an aquifer is defined as the product of hydraulic conductivity times the vertical thickness of the aquifer, b; thus

$$T = Kb \qquad \ldots(14)$$

If we consider the flow of groundwater down a gentle valley, the total quantity of water flowing across a section of the valley 1 metre wide would be, in cubic metres per day,

$$Q = T \frac{\Delta \Phi}{\Delta x}$$

if we employ consistent units and specify T in metres^2 per day.

5. General theory of groundwater flow

It is generally true that no aqifer or under-ground water body is in a steady state of recharge versus discharge. There is some change occurring and we are obliged to consider the general equation which can be developed from Equ. (6). It may be shown (Todd, 1959) that groundwater flow is described by the equation

$$\frac{S}{T}\frac{\partial \Phi}{\partial t} = \frac{\partial^2 \Phi}{\partial x^2} - \frac{A}{K} \qquad \ldots(15)$$

in the simple situation when flow is predominantly horizontal and only one space co-ordinate need be considered. The symbol A refers to a uniform loss of water by seepage or by transpiration from the aquifer. A more general equation than (15) was derived by Boussinesq in 1877 in a monumental study of groundwater flow.

We should consider here one solution of Equ. (15) to illustrate the information to be gained and to serve as an introduction to the lecture on factors involved in optimum development of ground-water reservoirs. When Dr. J.N. Luthin visited this country in 1958, we analysed a small and not very important aquifer near the Coorong, to investigate the mean life-time of water in it (Luthin & Holmes, 1960). The geometry of the aquifer could be simplified to that shown in Fig. 5.

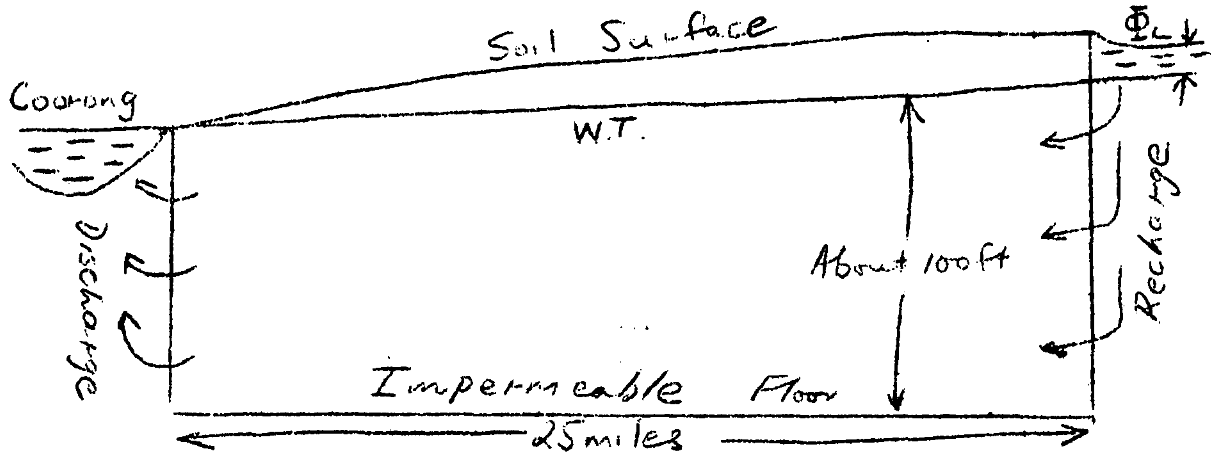

Fig. 5. Groundwater aquifer, simplified geometry.

The solution of the problem is shown graphically in Fig. 6, assuming that the leakage A is negligible.

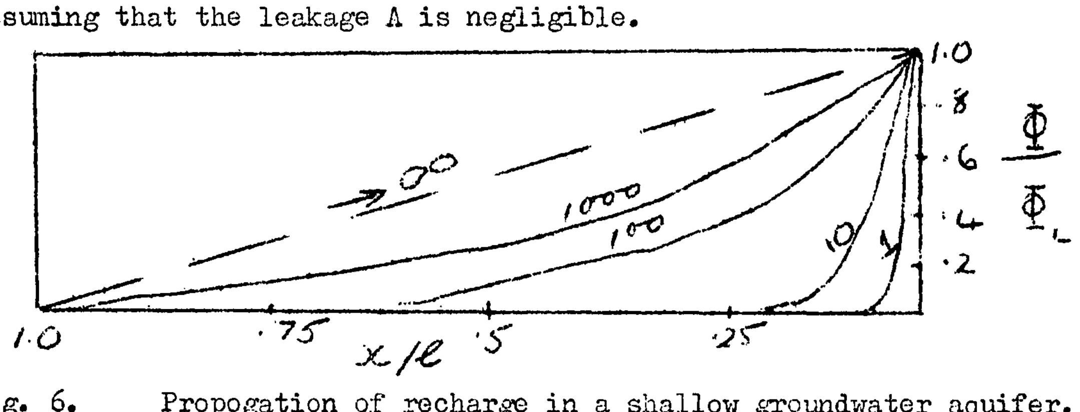

Fig. 6. Propogation of recharge in a shallow groundwater aquifer. The ordinate Φ/Φ_L is the fractional change in water level height caused by recharge. The abscissa is the fractional length of the aquifer, where 1 = 25 miles. The family of curves shows the water level after 1 year from a step recharge function, 10 years from recharge, and so on to approaching a very long time after recharge. It can be seen that the time scale is long. It is even longer for the great artesian aquifers of the world (see Werner, 1946).

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GROUND WATER SCHOOL

WELL HYDRAULICS

by

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The intention of this lecture is to discuss the derivation of the basic formulae used in the analysis of pumping tests. No claims to originality are made. It is not possible to explain all of the mathematical steps but it is possible to discuss the assumptions made. Two useful results of this discussion are that it becomes easier to convert from one set of units to another, and also that it is possible to untangle what units are implied in various formulae.

The basis of the calculationsis Darcy's law which has already been discussed in these lectures. Darcy's law relates the velocity of fluid flow to the pressure gradient which causes it. The velocity of flow is not the actual velocity of an individual water particle but is the volume of water which flows across unit area in unit time. It would be convenient for anyone using the usual English system of "Foot Pound Second" units to measure volume as cubic feet and area in square feet. The velocity of flow would then be measured as feet per second. Unfortunately, volumes are more often reckoned in gallons and it is not so easy to visualize the velocity. All derivations here will involve a consistent use of F.P.S. units unless indicated otherwise.

It is convenient and almost universal to measure pressures in feet of water. The rate of change of pressure with distance, the pressure gradient, will be a pure number. For water at rest the vertical pressure gradient will be -1 because the pressure decreases with increasing height.

According to Darcy's law, which is an experimental result, the horizontal velocity is given by

$$V_2 = -K \times \text{pressure gradient}$$

and the vertical velocity is given by

$$V_y = -K(1 + \text{pressure gradient})$$ where K is the hydraulic conductivity.

Sometimes K is called <u>permeability</u>, but strictly permeability is a different concept. The negative sign indicates that water will flow horizontally from high pressure to low pressure.

Hydraulic conductivity will be measured in feet/second in our system of units. The range of conductivity values is very wide, from about 1 ft/sec. for a very coarse gravel to about 10^{-4} ft/sec. for sands and about 10^{-8} ft/sec for silts and clays, for some impervious materials the conductivity can be about a thousand times less again.

Layered aquifers often conduct much better in a horizontal than in a vertical direction and two values of hydraulic conductivity must be specified. However, most pump tests involve very little vertical flow and horizontal conductivities are measured.

Darcy's law may not apply if linear velocities are very large. This situation is rare and can only occur close to a very strongly pumped well. There are some assumptions commonly made about "well loss" in these cases but they are empirical and will not be discussed here.

The discussion is simplified if a sharp distinction is now made between "confined" or "pressure" flow such as occurs in an ideal artesian aquifer i.e. completely filled and "unconfined" or "water table" flow in which the shape of the free water surface can vary.

Confined Flow

In discussions of confined flow it is usual to consider an aquifer of uniform thickness bounded on top and bottom by horizontal impervious beds. The aquifer is assumed to be infinitely large in all horizontal directions.

The pressure of the water at any position can be determined by driving in a small tight fitting standpipe which is open at the bottom. If there is very little vertical flow of water the height of water in the standpipe will be independent of the depth of the opening in the standpipe. The surface which passes through the water level in each pipe is called the piezometric surface.

Although the aquifer is always full of water it will hold more or less water if the pressure rises or falls. The change is partly due to compression of the water and partly due to compression of the aquifer material.

Consider a vertical column of cross section A and extending from the bottom to the top of the aquifer.

If the water pressure throughout the column increases by h feet of water, the extra volume of water which is held in the column will be SAh cubic feet. S will be a pure number and it is called the storage coefficient of the aquifer. Note that S depends upon the aquifer thickness as well as its material. Numerical values of S may vary between say, 10^{-3} and 10^{-5}.

In principle, a knowledge of Darcy's law and the storage coefficient would enable us to compute changes of pressure with time and with distance from a pumped well. Usually, there is a radial symmetry which simplifies the problem to two dimensions in space. Some old discussions neglected variations in time. However, in general, a true steady state can not be anticipated because water is being removed and nothing is being replaced. Usually, we can only expect a decreasing rate of pressure drop as the zone of influence of the pumping increases.

Dupuit Forchheimer Assumptions

It is often reasonable to assume that there is negligible vertical flow. This assumption takes slightly different forms but is generally called the "DUPUIT - FORCHHEIMER" assumption. A simple calculation using this assumption helps to explain some of the ideas and to derive a basic formula.

Consider an artesian aquifer of thickness m feet and pumped from a completely penetrating well at a rate of Q cubic feet per second. Use cylindrical coordinates, i.e. y for height and r for radial distance and let the pumped well be centred about the axis Oz in Figure 1.

We can calculate the flow of water across a cylindrical surface of radius r. The velocity towards the axis Oz will everywhere be

$K\dfrac{\partial h}{\partial r}$ where $\dfrac{\partial h}{\partial r}$ is the pressure gradient, which is written with curved differential signs to indicate that h depends not only upon r but also upon the time t. The total volume flowing across the cylindrical surface, which has an area

$2\pi r\, m$ will be $2\pi K\, m\, r\dfrac{\partial h}{\partial r}$

If two cylindrical surfaces of radii r and r + dr are considered, the net rate of gain by the annulus will be

$$2\pi K m \left(r\frac{\partial h}{\partial r}\right)_{r+dr} - \left(r\frac{\partial h}{\partial r}\right)_r$$

$$= 2\pi K m \, dr \, \frac{\partial}{\partial r}\left(r\frac{\partial h}{\partial r}\right).$$

It turns out that this "gain" is negative i.e. it is really a loss but consistent use of "gain" avoids difficulty with signs. The net gain must equal the rate of gain of water stored under pressure inside the ring, which has a cross sectional area 2 r dr

$$\therefore 2\pi K m \frac{\partial}{\partial r}\left(r\frac{\partial h}{\partial r}\right) = 2\pi r S \frac{\partial h}{\partial t}$$

which becomes

$$\frac{\partial}{\partial r}\left(r\frac{\partial h}{\partial r}\right) = \frac{rS}{T}\frac{\partial h}{\partial t}$$

where T = Km is called the <u>transmissibility</u> of the aquifer and S as already mentioned is the <u>storage coefficient</u>.

This last equation is technically called a partial differential equation. To solve the equation it is necessary to know certain "boundary conditions" which are here, that h is always finite and also that the volume of water Q is removed each second at the well.

The solution applies to a well of infinitely small radius. It is often called the Theis solution. The pressure is given by

$$ho - h = \frac{Q}{4\pi T} W(u),$$

where ho is the initial head,

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

and W(u), which is often known as the "<u>well function</u>" can be calculated for any value of u. The definition of W(u) is given as an integral,

$$W(u) = \int_u^\infty e^{-x}/x \, dx$$

where x is a "dummy" variable which is "killed" when the limits of integration are inserted.

Usually we are only interested in moderately large values of t, i.e. u is small, and W(u) can be approximated by

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

$$= -\ln 1.74 + \ln \frac{4Tt}{r^2 S}$$

where "ln" means "logarithm to the base e"

$$\therefore W(u) = \ln\left(\frac{4Tt}{r^2 S} \times \frac{1}{1.74}\right)$$

$$= \ln\left(\frac{2.25Tt}{r^2 S}\right)$$

$$= 2.30 \log_{10} \frac{2.25Tt}{r^2 S}$$

$$\text{and } ho - h = \frac{2.30\,Q}{4\ \ T} \log_{10} \frac{2.25Tt}{r^2 S} = s$$

Note that the <u>draw down, ho - h</u>, has a logarithmic shape if t is large enough to justify these approximations. Note also that the shape is always changing and never approaches a steady or equilibrium state even though the rate of change becomes steadily less.

If values of draw down are available from simultaneous observations at several observation wells it is possible to compute the transmissibility T. If the observations can be repeated at known values of t it will also be possible to calculate the storage coefficient S.

Often it will be preferred to use only one observation well because extra wells can be expensive to drill and to equip. It is still possible to calculate T and S if enough observations are made at the one well.

There is a preference for using two or more observation wells because it is thought that the inevitable variations of pumping rate have less effect upon the calculations. Also it is thought that two or more wells are representative of the portion of the aquifer between the two wells. Of course the single well methods of calculation can be performed at each well and used to assess the variation between observation sites.

Several methods of calculating S andT from observations at a single well have been suggested. For sufficiently small u, i.e. large t, Jacob plots draw down against $\log_{10} t$. A straight line should result whose slope must be

$$\frac{2.30\ Q}{4\pi T}$$

From which T can be calculated. The intercept of the extended line ($\log t = 0$) must be

$$\frac{Q \times 2.30}{4\pi T} \log_{10} \frac{2.25\ T}{r^2 S}$$

from which S can be calculated. See Fig. 2 for an example.

A more general method, ascribed to Theis, is valid for all values of u. Plot log W (u) against log 1/u on a transparent sheet. This is the "type curve". On the same scale but on a separate sheet plot log (ho - h) and log T. We have

$$\log \frac{1}{u} = \log \frac{4Tt}{r^2 S} = \log t + \log \frac{4T}{r^2 S}$$

and $\log (ho - h) = \log Q/(4\pi T) + \log W(u)$

Therefore if the two graphs are displaced sideways and upwards, but never rotated, there must be some position for which they will agree and the "type curve" should coincide with the experimental curve. Take any point on the curve, for which the values of ho - h, W(u), t and u are of course available. Calculate T from the draw down and then calculate S from the value of u. See Fig. 3.

In general T can be selected with more precision than can S. The draw down is less dependent upon S than T and the inevitable deviations from theoretical curves confuse the estimation of S.

Solutions which avoid the Dupuit - Forchheimer assumptions

It is possible to avoid the assumption that there is no vertical flow. Hantush has solved several problems using only Darcy's law and a simplifying boundary condition at the well. For the problem discussed above he finds that the answer given by the Dupuit - Forchheimer theory is valid providing $tT > 30\ rw^2 S$ where rw is the "effective well radius". This condition is satisfied for quite small values of t.

Leaky Aquifer

Hantush has also solved some problems in which horizontal flow should not be anticipated. The only problem which we can consider here is that of the leaky aquifer. In this problem the main permeable aquifer is confined between an impermeable boundary and a leaky aquifer of very low hydraulic conductivity. If the water pressure is relieved in the main aquifer, water will be released from the leaky aquifer.

It is assumed that the leaky aquifer has a very small storage coefficient, and also that the flow in the leaky aquifer is entirely vertical. Let the thickness of the main aquifer be m feet and that of the leaky aquifer be m feet. Let K and K' be the corresponding hydraulic conductivities. Introduce a new length B defined by $B^2 = K\,mm'/K'$. The formula for draw down at a distance r is

$$ho - h = (Q/4\pi T)\ W(u,\ r/B)$$

where $W(u, r/B)$ is called the "modified well function" and can be computed numerically.

Calculation of S and T follows the same form as before except that the "Type curve" is replaced by a whole series of curves, each corresponding to a definite value of r/B. The curve corresponding to $B = \infty$ is the same as earlier used for $W(u)$. See Fig. 4.

The "leaky aquifer" problem can lead to a steady state in which the leaky aquifer supplies the pumped well.

Units

It is now necessary to consider the units of measurement used in practical situations. All the discussion so far has assumed a consistent use of English "foot-pound-second" system, more properly "foot and second" system, since use of mass or weight units have been avoided. There would be no change in the formula if some other unit of time or of length were chosen but the numerical values of transmissibility, etc. would need to be altered. The conversion would be obvious from the name of the unit.

Unfortunately, all field measurements are, or claim to be, based upon the U.S. Geological Survey units which are irrational on two distinct grounds.

Firstly, all distances are measured in feet but volumes are reckoned in gallons instead of cubic feet. It should be noted that U.S. Gallons and not Imperial Gallons are meant. The conversions are 1 cubic foot = 7.481 U.S. Gallons = 6.229 Imperial Gallons. Secondly, two quite different units of time are used. Rate of pumping is usually measured in U.S. Gallons per minute, but the time of pumping and the time for measurements of hydraulic conductivity is quoted in days.

It is convenient to set out the various conversions. Denote the numerical value in the academic "F.P.S"system by Xa' and the numerical value of the corresponding field measurement in "Survey Units" by X.

Length, area, pressure and pressure gradient are unaltered

This must also apply to B in the leaky aquifer.

Pumping rate is measured in U.S. Gallons per minute

$$Q = Q' \times 7.481 \times 60$$

Time of pumping is measured in days

$$t = t' \ /(24 \times 60 \times 60)$$

Hydraulic conductivity is measured in U.S. Gallons/day/sq. ft. These are also called Meinzer units.

Transmissibility is measured in U.S. Gallons/day/ft

i.e. $T = T' \times 7.481 \times 24 \times 60 \times 60$

Storage coefficient is not altered

i.e. the volume of water gained or lost from the column of aquifer is still measured in cubic feet.

Substituting these values in the above equations

$$u = \frac{(r')^2 S'}{4 T' t'} = \frac{r^2 S}{4Tt} \times 7.481$$

$$= \frac{1.87\, r^2 S}{Tt}$$

and the draw down $ho - h =$

$$= \frac{Q'}{4\pi T'} \quad W(u)$$

$$= \frac{Q \times 7.481 \times 24 \times 60 \times 60}{7.481 \times 60 \times 4 \quad T} \quad W(u)$$

$$= \frac{114.6\, Q}{T} \times W(u)$$

$$= \frac{114.6\, Q}{T} \times W(\frac{1.87\, r^2 S}{Tt}$$

Expanding the espression obtained above for W(u), which is only valid for small u

$$s = \frac{2.30 \times 114.6\, Q}{T} \log_{10} \frac{2.25\, Tt}{7.481\, r^2 S}$$

$$= \frac{264\, Q}{T} \log_{10} \frac{0.30\, Tt}{r^2 S}$$

If the transmissibility is to be calculated from simultaneous draw down measurements at several wells the equation can be written in yet another form

$$s = \frac{264Q}{T} \log_{10} \frac{0.30Tt}{S}$$

$$- \frac{528\, Q}{T} \log_{10} r$$

i.e. if the draw down s is plotted against $\log_{10} r$ a straight line should result from whose slope T can be calculated if Q is known.

There is a further complidation that the time of pumping is sometimes given in minutes although hydraulic conductivity and transmissibility are expressed in terms of days.

The formulae now becomes

$$s = \frac{114.6\, Q}{T} \; W(u) \qquad \text{and}$$

$$u = \frac{1.87 \times 60 \times 24\, r^2 S}{Tt}$$

$$= \frac{2693\, r^2 S}{Tt}$$

The above formulae are those most often encountered because most of the references are American. If English or Imperial gallons are used it must be remembered that for the same experiment the numerical values of pumping rate and transmissibility will be approximately 5/6 of those quoted in survey units. If pumping rate is measured in Imperial gallons and the values substituted in the above formulae T will be calculated too small and S will then also be calculated too small.

If Imperial units are used consistently,

$$u = \frac{6.229}{4}\ \frac{r^2 S}{Tt} = 1.557\ \frac{r^2 S}{Tt}$$

$$\text{and } s = \frac{114.6Q}{T}\ W(u)$$

$$\approx \frac{264Q}{T}\ \log_{10}\ \frac{0.36Tt}{r^2 S}$$

providing t is measured in days. If however the time of pumping is measured in minutes

$$u = \frac{2242\ r^2 S}{Tt}$$

English engineers frequently calculate in cubic feet and seconds but English variations of Survey units are also often used.

Unconfined Flow

If either the top or the bottom of the aquifer is not limited by some impervious bed the water flow is said to be "unconfined". The mathematical discussion is greatly complicated because the position of one boundary is unknown. This discussion assumes that the aquifer is supported by an impermeable medium but that there is no confining top to the aquifer.

The water is not confined under pressure and therefore the storage coefficient as defined earlier is not significant. However, because the water table can rise or fall the volume of water absorbed or yielded up by the pores of the aquifer material is important. At first it will be assumed that this water is exchanged as soon as the water table moves. In place of the storage coefficient we consider the specific yield or drainable porosity which is a pure number and typically much larger than the storage coefficient of a confined aquifer. The symbol S will also be used for this.

It is customary to use an extension of the Dupuit -Forchheimer assumptions in problems involving unconfined flow. There are various statements of these assumptions but all involve the slope of the water table. The most general expression for the flow gives the total flow across unit length of a boundary to be $Kh\frac{dh}{ds}$ where h is the height of the water table above the impermeable boundary and $\frac{dh}{ds}$ is the slope of the water table measured at right angles to the boundary.

The calculation for a pumped well follows that for a confined aquifer. The flow across a cylindrical surface of radius r will be $2\pi rKh\frac{\partial h}{\partial r}$ where h depends upon r and also upon time t. The net gain by the annulus between r and r + dr is equated to the time rate of gain and we have

$$2\pi K\frac{\partial}{\partial r}\left(r\,h\frac{\partial h}{\partial r}\right) = 2\pi rS\,\frac{\partial h}{\partial t}$$

This equation may be rewritten

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\,\frac{\partial h}{\partial r} + \frac{1}{h}\left(\frac{\partial h}{\partial r}\right)^2 = \frac{S}{hK}\,\frac{\partial h}{\partial t}$$

which resembles the Theis equation except that the denominator on the right hand side contains h which is not constant, and also that a term $(\frac{\partial h}{\partial r})^2$ occurs on the left hand side. Both these difficulties render the equation "non linear" and enormously complicate an exact solution.

Only approximate methods of solution are available. It is usual to replace the right hand denominator by m K or T where m was the original depth of water before pumping began. This approximation will be all the worse near the pumped well.

There are two common ways to handle the second term on the left hand side. Usually this term is small and many treatments simply ignore it. This approximation also must be all the worse near the pumped well where the slope is greatest. If this term is ignored the equation becomes identical with that of Theis and the same result holds for the draw down, viz.

$$s = (Q/4\pi T) \quad W(u)$$

$$u = r^2 S/4Tt$$

It should be remembered that this solution has introduced yet another simplification near the well because it has replaced the finite well by an infinitely thin "sink". There are therefore good reasons not to trust this expression if it is evaluated very close to the well.

Another method of approximation does not neglect the square term in the differential equation. If the original equation is rewritten in terms of h^2

$$\pi K \frac{\partial}{\partial r}\left(r \frac{\partial h^2}{\partial r}\right) = \frac{\pi r S}{h} \frac{\partial h^2}{\partial t}$$

The denominator is simplified as before and we get

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

which is identical to the Theis equation and the solution is

$$m^2 - h^2 = \left(\frac{Q}{2\pi K}\right) \quad W(u)$$

The denominator on the right hand side is determined by the boundary conditions at the well, which is once again assumed to be infinitely narrow. If the left hand side is expanded we have for the draw down $= m - h$, $s(1 - \frac{s}{2m}) = \frac{Q}{4\pi Km} W(u)$ i.e. the Theis solution can be used but the draw down must be corrected by subtracting $^2/2m$. This is important whenever s becomes comparable with m.

Sometimes this correction to the draw down is recommended as though it were an allowance for delayed drainage from a dewatered zone. In fact there is very little information on delayed drainage. Some calculations made by Boulton (International Hydrological Association Publication 37, 1954) do not produce this result. There do not appear to be any means for calculating corrections for delayed drainage.

The most thorough calculations of water table situations have been made by Boulton (Institute of Civil Engineers - Proceeding Pt III Vol 3 1954 P564). Boulton avoids the Dupuit - Forchheimer assumptions and calculates from Darcy's equation and the equation of continuity. He is obliged to take simplified boundary values both at the well and at the free water surface. Also the difficulty with squares of partial derivatives arises and these terms are neglected.

Boulton concludes that providing $K\, t > 5Sm$ the Theis solution is correct (to within 5%) providing also that $0.2\, m < r < 6m$. Note that the restriction on time is expressed in any rational system. In survey units it will be $Kt > 37.4Sm$ and for the comparable English units $Kt > 31.1Sm$.

BIBLIOGRAPHY

Books here have been classified by their mathematical requirements. As a guide, this lecture is definitely "non mathematical".

Non Mathematical

D.K. TODD "Ground Water Hydrology". Wiley 1959

An attractively written book which includes much more than well hydraulics. Inclined to switch units without warning.

J.G. FERRIS, D.B. KNOWLES and R.W. STALLMAN "Theory of Aquifer Tests" - Geological Survey Water Supply Paper 1536 - E 1962.

Really a collection of papers by above authors and some others. Lacks continuity. Has some practical aspects.

R.H. BROWN et. al. Methods of determining Permeability Transmissibility and Drawdown. Geological Survey Water Supply Paper 1536 - I. 1963.

Rather similar to above. Some articles by Jacob are relevant to this lecture.

W.C. WALTON Selected Analytical Methods for Well and Aquifer Evaluation. Illinois State Water Survey. 1962.

Short account of theoretical results and considerable description of actual tests made.

W.C. WALTON Leaky Artesian Aquifer Conditions in Illinois.

Illinois State Water Survey 1960. Some field examples are discussed.

L.K. WENZEL and V.C. FISHEL Geological Survey Water Supply Paper 887, 1942.

Contains an account of equilibrium calculations as well as non equilibrium formulae. Rather old fashioned.

R. BENTALL (ed) Shortcuts and Special Problems in Aquifer Tests. Geological Survey - Water Supply Paper 1545 - C 1963.

Collection of many short papers, some of which discuss special boundary conditions.

Mathematical

M.S. HANTUSH Hydraulics of Wells in "Advances in Hydroscience" Vol. 1. 1964.

This article occupies only 150 pages. The remainder of the book is irrelevant. Very good article indeed. Much of this information is not otherwise available and all of it is relevant.

P.Ya. POLUBARINOVA - KOCHINA Theory of Ground Water Movement Translated J.M.R. de Wiest 1962.

Forbiddingly mathematical and not suited to a practical man's needs. However it has some results not otherwise available. Much of this book is more suited for studies on seepage of water under dams etc.

For Groundwater School, March 1965.
"Well Hydraulics" by J Colville

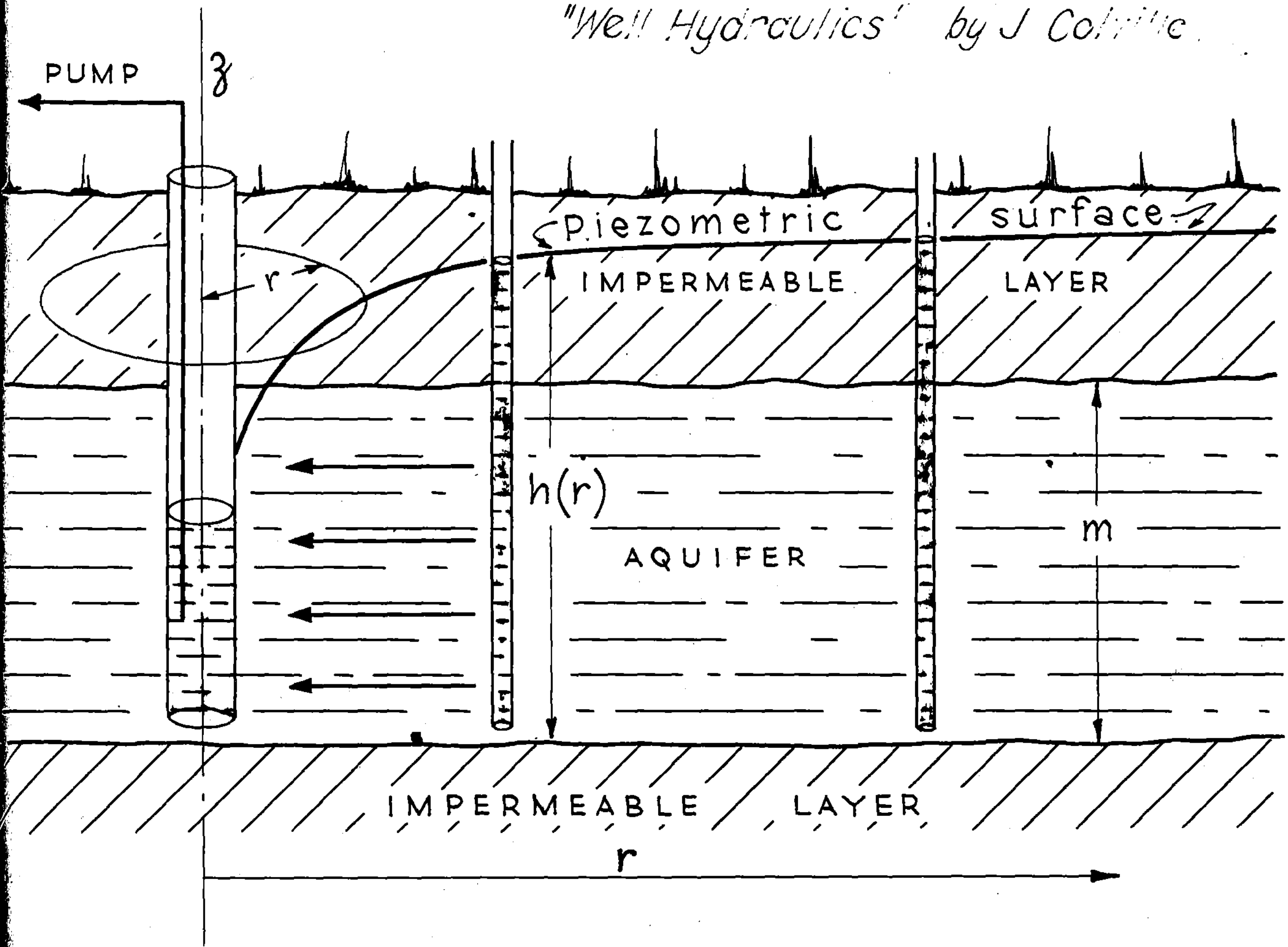


FIG. 1. Water flow towards a pumped well in a confined aquifer.

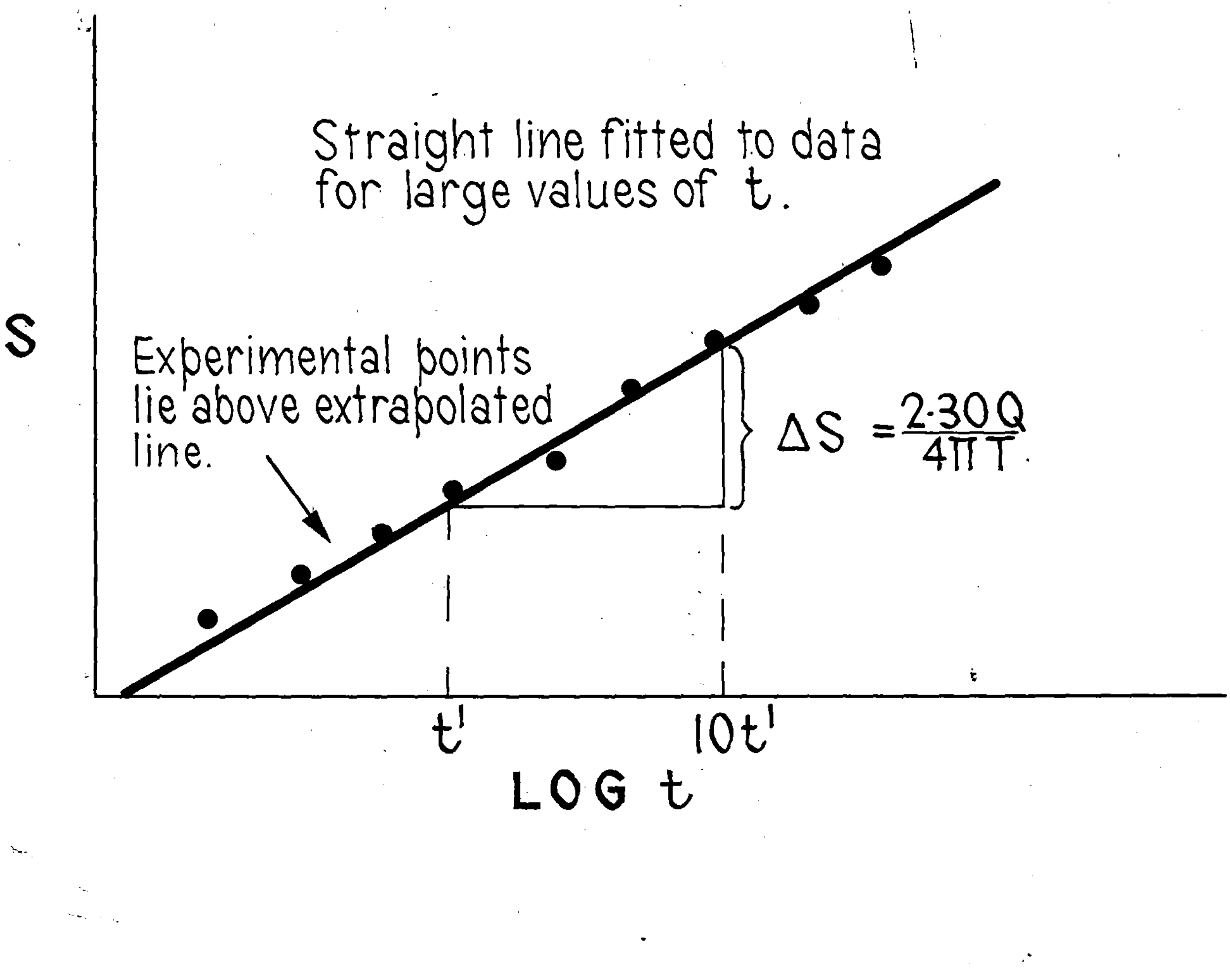


FIG. 2. Jacobs method of calculation of T and S

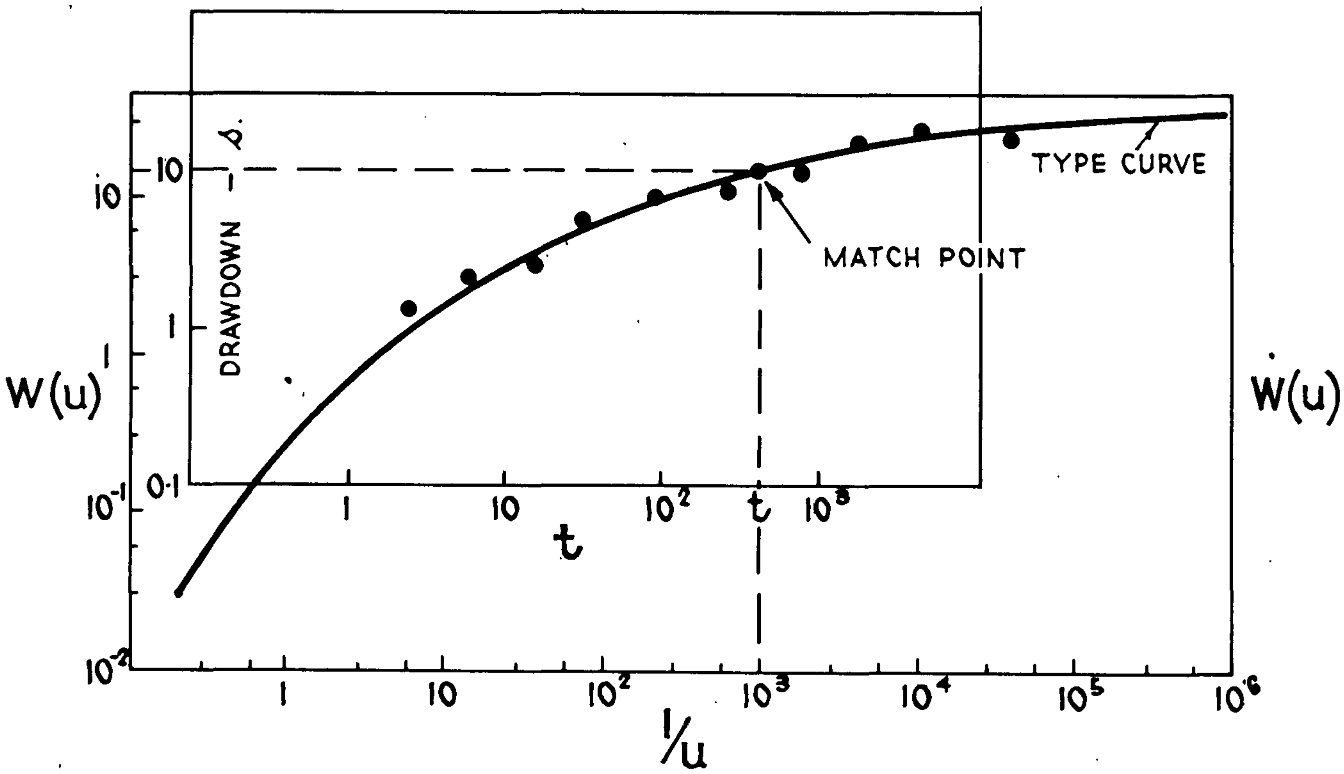


FIG. 3. Theis method for analysis of well draw down data.

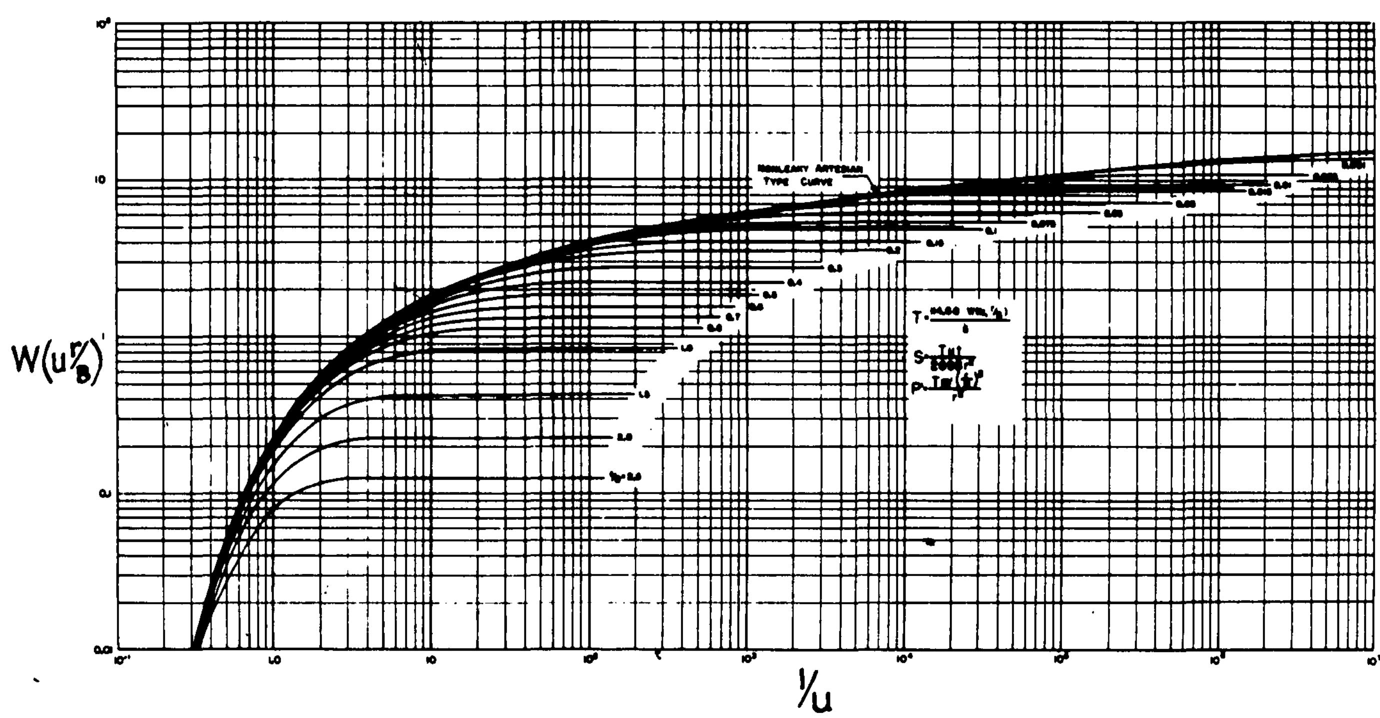


FIG. 4. Nonsteady state leaky Artesian type curves.

GROUND WATER SCHOOL

Summary of formulae in Well Hydraulics.

A. Confined or Artesian Flow

	F.P.S. Units	U.S.G.S. Units	Modified U.S.G.S.	Imperial	Modified Imper.'
Distance r	feet	feet	feet	feet.	feet.
Drawdown s	feet	feet	feet	feet.	feet.
Pumping Rate Q	Cusecs.	U.S. gallons/min.	U.S. galls/minute.	Imp. galls/min.	Imp. galls/min.
Transmissibility T	ft^2/sec	U.S. galls/day/ft.	U.S. galls/day/ft.	Imp. galls/day/ft.	Im. galls/day/ft.
Storage Coeff. S	Dimensionless & unchanged in all systems.				
Time of Pumping t	seconds	days	minutes	days	minutes
Dimensionless Variable u	$\frac{r^2 S}{4Tt}$	$\frac{1.87\, r^2 S}{Tt}$	$\frac{2693\, r^2 S}{Tt}$	$\frac{1.56\, r^2 S}{Tt}$	$\frac{2242\, r^2 S}{Tt}$
Well Function $W(u)$	$W(u) = \int_u^\infty e^{-x}/x \, dx$ in all systems of units. $\simeq 2.30 \log_{10}\left(\frac{1}{1.74u}\right)$ in all systems if $u < 10^{-2}$				
Approximation to $W(u)$	$2.30\log_{10}\left(\frac{2.25Tt}{r^2 S}\right)$	$2.30\log_{10}\left(\frac{0.30\,Tt}{r^2 S}\right)$	$2.30\log_{10}\left(\frac{2.1\times10^{-4}Tt}{r^2 S}\right)$	$2.30\log_{10}\left(\frac{0.36\,Tt}{r^2 S}\right)$	$2.30\log_{10}\left(\frac{2.5\times10^{-4}Tt}{r^2 S}\right)$
Expression for drawdown s	$\frac{Q}{4\pi T}W(u)$	$\frac{114.6\,Q}{T}W(u)$	$\frac{114.6\,Q}{T}W(u)$	$\frac{114.6\,Q}{T}W(u)$	$\frac{114.6\,Q}{T}W(u)$
Slope of draw-down - $\log_{10} t$ curve for large t.	$\Delta s = \frac{0.183\,Q}{T}$	$\Delta s = \frac{264\,Q}{T}$	$\Delta s = \frac{264\,Q}{T}$	$\Delta s = \frac{264\,Q}{T}$	$\Delta s = \frac{264\,Q}{T}$
Extrapolated time of zero drawdown t_0.	$S = \frac{2.25\,Tt_0}{r^2}$	$S = \frac{0.30\,Tt_0}{r^2}$	$S = \frac{2.1\times10^{-4}\,Tt_0}{r^2}$	$S = \frac{0.36\,Tt_0}{r^2}$	$S = \frac{2.5\times10^{-4}\,Tt_0}{r^2}$

B. Artesian Flow with Leaking Boundary

Equivalent Depth B	Feet in all systems of units.
	$B^2 = mm' K/K'$
Drawdown.	Replace $W(u)$ by $W(u, r/B)$

C. Unconfined Flow.

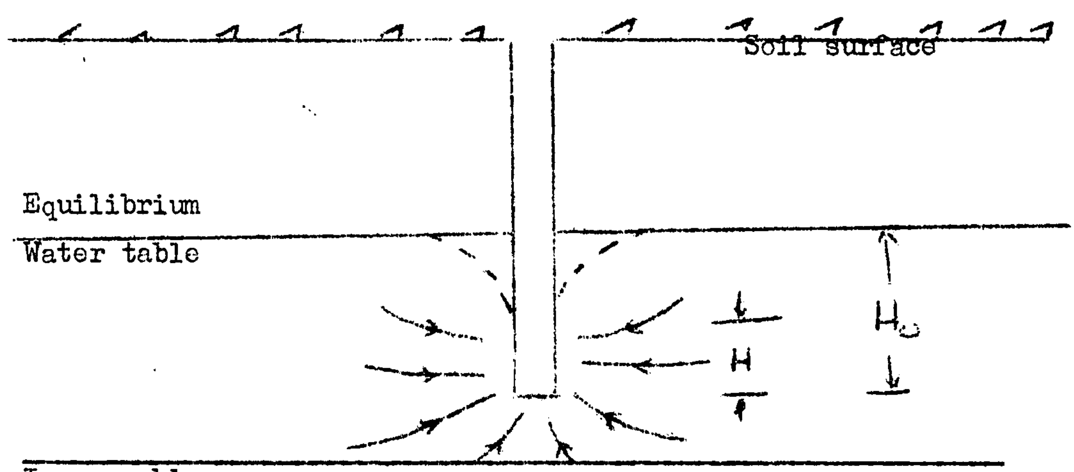
Corrected drawdown	$s' = s - \frac{s^2}{2m}$ in all systems of units.

FIELD MEASUREMENT OF HYDRAULIC CONDUCTIVITY AND WATER CONTENT

by

J.W. Holmes, Division of Soils, C.S.I.R.O., Adelaide

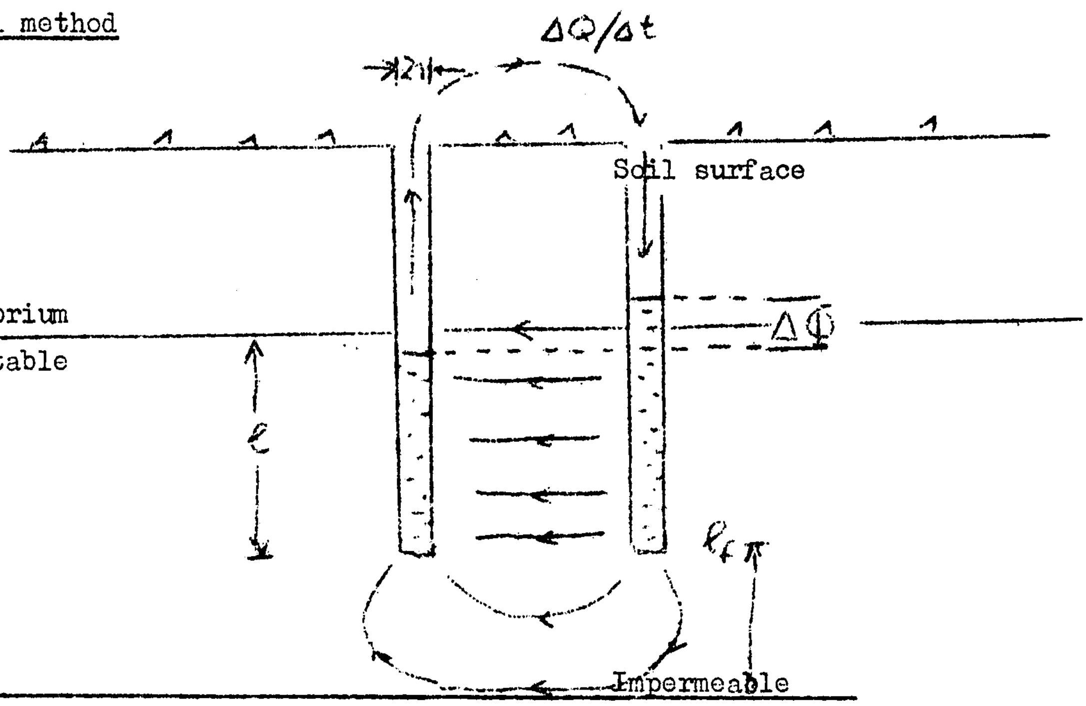
1. Auger-hole method



$$\frac{dQ}{dt} = KA(Ho - H) \quad \text{and approximately}$$

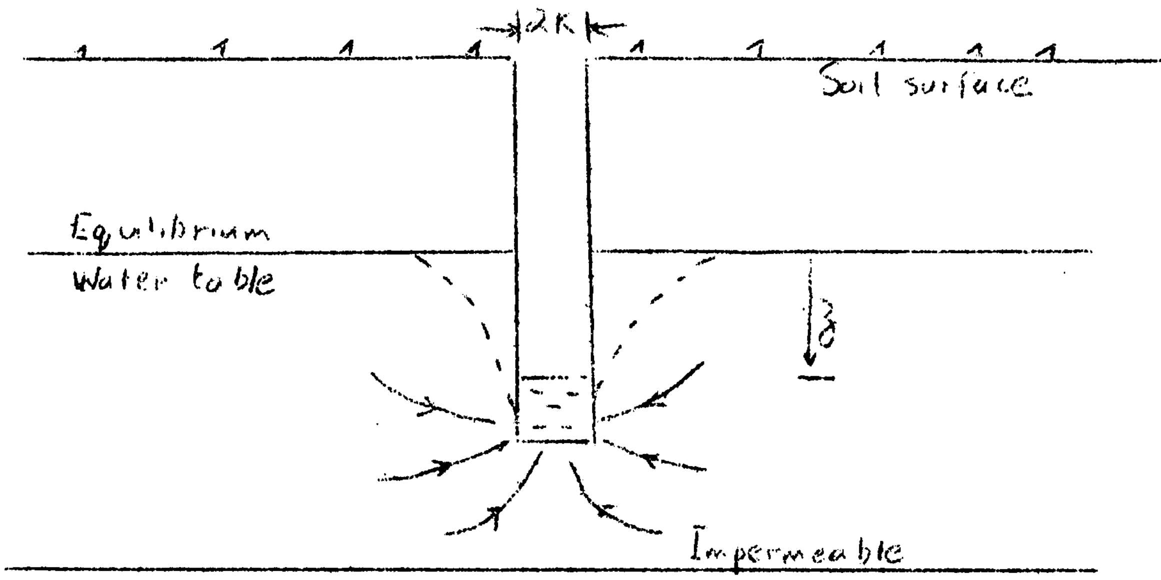
$$K = C\,\frac{dH}{dt}$$

2. Two-well method



$$K = \frac{\cosh^{-1}(d/2r)}{\pi} \cdot \frac{\Delta Q}{\Delta t} \cdot \frac{1}{(\ell + \ell_f)\,\Delta \Phi}$$

3. Kirkham-tube (piezometer) method



$$K = \frac{\pi R^2}{A} \cdot \frac{\log_e(z_1/z_2)}{t_2 - t_1}$$

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Childs, Collis-George and Holmes. - J. Soil Sci. 8:27-41 : (1957).

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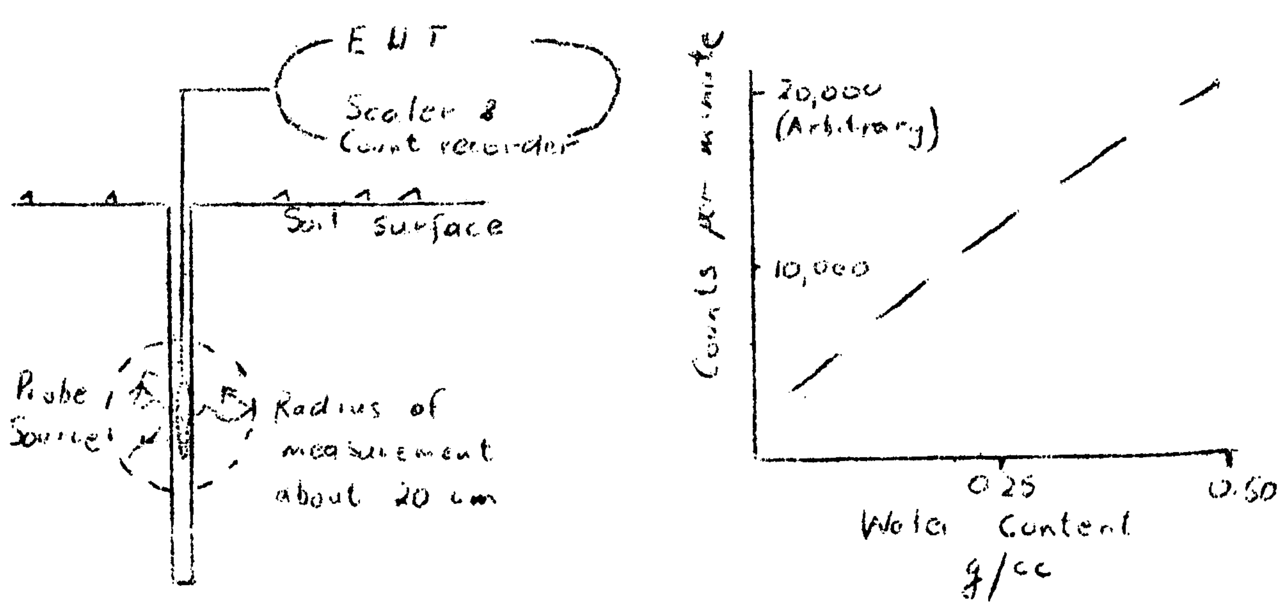
4. Water content by neutron moisture meter



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PUMPING TESTS

PART 1

PRINCIPLES & METHODS

by

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New South Wales.

INDEX

PUMPING TESTS

1. PART I: PRINCIPLES AND METHODS

PUMPING TESTS

PART I : PRINCIPLES AND METHODS

1.1 INTRODUCTION

Pumping tests are a basic tool in groundwater hydrology, and those working in this field should have a clear understanding of the principles involved and be able to correctly interpret test data. This is particularly important in those cases where large yields and long-term pumping are involved, e.g. irrigation or town-water supply bores.

A pumping test may be carried out for one or both of the following purposes:-

(a) to determine the performance and optimum yield of a well or bore;

(b) to determine the hydraulic properties of the aquifer;

(Note: Some tests involve pumping into rather than out of a bore or well. These will also be considered).

1.2 TESTS OF PERFORMANCE AND YIELD.

1.2.1 Single-rate pumping tests.

Practically all pumping tests carried out by private boring and well-sinking contractors are in this category. However, contractors commonly record no more than the static water level and the drawdown level for a given pumping rate. This is inadequate for proper interpretation because it gives no indication of the rate of drawdown. It is sometimes found, too, that contractors extrapolate such figures on the assumption of direct proportion; and this can lead to considerable error. Where a bore is relatively low yielding, e.g. the general run of stock-water bores, and the bore is to be pumped at the same rate as it was tested, The usual contractor's test may suffice, but the position is more critical and unsatisfactory where long term pumping at high rates is proposed. Consequently, contractors should be advised of the data required from pump tests to allow analysis of the results. (It may be noted that in the case of stock bores probably the majority are tested by bailing. This simulates pumping, and, if properly carried out, can give useful information, but the method is subject to limitations and misinterpretations).

1.2.1.1 Confined aquifers.

The principle of a single-rate pumping test of a bore tapping a confined aquifer is based on the non-equilibrium formula introduced by Theis (1935). The formula is

$$s = \frac{114.6Q}{T} \int_u^{\infty} \frac{e^{-u}}{u} \, du \qquad \ldots(1)$$

or, evaluating the integral,

$$s = \frac{114.6Q}{T} \left(-0.5772 - \log_e u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} \ldots \text{etc}\right) \qquad \ldots(2)$$

where $u = \dfrac{2693r^2S}{Tt}$

s = drawdown in ft. at a distance r, in feet, from a pumped bore discharging at Q gallons per minute for a time t in minutes.

T = coefficient of transmissibility in gallons per day per foot.

S = coefficient of storage.

Cooper and Jacob (1946) showed that for small values of (r^2/t) compared to the value of $(4T/S)$, u will be so small that the series following the first two terms in the series in the above equation may be neglected. They consider the approximation tolerable where u is less than 0.02. Converting to common logartithms, the equation may then be rewritten as

$$s = \frac{2.303Q}{4\pi T} \left[\log_{10} t - \log_{10} (r^2 S/2.25T)\right] \qquad \ldots(3)$$

so that when r is constant (as is the case of a bore being tested for yield), the only variables are s and t, and the equation is that of a straight line plot of s against $\log_{10} t$. The graph thus obtained is referred to as a "time-drawdown" graph, and the method as the "straight-line method" or the modified non-equilibrium method".

This method offers the most direct and simplest means of interpreting pumping test data and gives satisfactory results for all practical purposes. (The fact that it allows determination of the coefficient of transmissibility and, if an observation bore is available, coefficient of storage from the same test data is more or less incidental where the main aim is simply to establish the optimum yield of a bore under conditions of long term pumping).

Consequently, if a bore is pumped at a constant rate, and the plot of drawdown against log time gives a straight line, this line can be extrapolated to determine the drawdown for any period of continuous pumping <u>at that rate</u>. (This, ofcourse, is on the usual assumptions of an infinite, homogeneous, and perfectly elastic aquifer. The effect of deviations from these assumptions are considered in Part 3). However, difficulties arise if it is proposed to equip the bore to pump at a rate other than that used for testing, especially if it is to be a much higher-rate. This matter will be dealt with in section 1.2.2.

1.2.1.2 Unconfined aquifers.

Where the bore or well being tested taps an unconfined aquifer, i.e. "water-table" conditions, the non-equilibrium formula does not adequately describe the draw-down behaviour because it does not allow for gravity drainage. The assumptions of the formula include (a) that the coefficient of storage is constant and (b) that water is released instantaneously with a decline in head. However, in the case of an unconfined aquifer the water pumped is being essentially derived by dewatering the saturated pore spaces of the aquifer, and, depending on the permeability of the formation, this draining may take considerable time to effect. Consequently, the coefficient of storage increases as pumping proceeds, although at a diminishing rate.

In spite of this non-compliance with the assumptions of the formula, with long periods of pumping the effect of gravity drainage becomes so small that satisfactory time-drawdown graphs, similar to those for confined conditions, can be obtained.

Where only the performance and yield are to be established, pumping at the desired rate, or at a rate which preliminary tests indicate to be the order of an optimum rate, is continued until the time-drawdown graph is a straight line, thus allowing extrapolation to determine the effect of long term pumping. (For very permeable aquifers the desired time-drawdown relationship may commence in under an hour, but for thick aquifers of low permeability it may take as long as a day or more). However, if it is desired to establish values for the hydraulic properties of the aquifer, various corrections and limitations apply, and these will be dealt with in a later section. (1.3.1).

It may be noted that , if at all possible, observations of recovery of water-level should be made after all single-rate pumping tests. As will be shown in Part 3, recovery data are often very significant and considerably assist the analysis of pumping test data.

1.2.2 Multiple-stage pumping tests.

It does not follow that the drawdown-discharge relationship established by a single-rate pumping test can be directly applied to a different pumping rate. This is perhaps best appreciated by considering the various factors which, depending on the geohydrologic conditions can affect drawdown. Walton (1962) has summarized these as follows:-

$$s = s_a + s_w + s_p + s_d + s_b - s_r \qquad \ldots(4)$$

where s s = total drawdown in production bore

s_a = drawdown due to laminar flow of waterthrough the aquifer towards the well (termed "aquifer loss").

s_w = drawdown due to turbulent flow through thescreen or well face and inside the casing to the pump intake. (termed "well loss").

s_p = drawdown due to partial penetration effects.

s_d = drawdown due to dewatering portion of the aquifer.

s_b = drawdown due to barrier boundaries of the aquifer.

s_r = build-up due to recharge boundaries of the aquifer.

Of these components, S_a must occur and s_w is almost invariably evident, while the others may or may not occur. If they do, s_b and s_r are usually evident from the nature of the time-drawdown graph (see Part 3), and allowances can be made for the effects of $s_p + s_d$ (see sections 1.3.1.3 and 1.3.2.4). Where it is desired to establish performance and yield, the components s_a and s_w are normally the more critical, and these can be separated by means of themultiple-stage (or step-drawdown) pumping test described by Jacob (1946).

A multiple-stage test consists of pumping at three or four successively higher rates, the change in rate being implemented more or less abruptly at the end of each pumping period. Observations are made of drawdown against time throughout the test. (In effect, the test simulates the same number of separate pumping tests as there are stages, but avoids the necessity of awaiting for recovery of water-level to be effected between each test. It may be noted that because of the problem of drainage time, multiple-stage tests are normally not applied to unconfined aquifer conditions).

As will be shown in Part 3, from the data obtained from the test, the "aquifer loss" and "well-loss" components of the drawdown, i.e. s_a and s_w in equation (4), can be determined. These are expressed in the equation

$$s = BQ + CQ^2 \qquad \ldots(5)$$

where s = drawdown in feet at a specified period.

B = aquifer-loss constant in sec/ft^2

C = well-loss constant in sec^2/ft^5

Q = discharge in cusecs.

As shown by Bruin and Hudson (1955), equation (5) is an approximation, and, strictly speaking, B and C are variables, but the errors involved by assuming them to be constants tend to be compensatory. Furthermore, Rorabaugh (1953) has shown the equation $s = BQ + CQ^n$ to be more correct, and has found n to be usually of the order of 2.5. (It has been the experience of the author that Rorabaugh's method of determining n, involving trial and error plotting, requires very refined data and preferably more than four stages). However, for practical purposes it is usually found that equation (5) gives satisfactory results.

When the values of B and C in equation (5) have been determined, the drawdown s for the selected time period (say 100 minutes) can be calculated for any particular discharge. The rate of drawdown at that discharge can be calculated from the slope of the straight-line time-drawdown graph, and the drawdown for any period of pumping can then be calculated by extrapolation at that rate. However, other allowances may also have to be made, depending on the conditions revealed by the analysis of the test data (see Part 3) or from knowledge of such factors as seasonal water-level fluctuations.

It should be noted from equation (5) that the component of drawdown due to "well loss" varies with the square of the discharge. It is for this reason that gross errors can be made if the results of a single rate test are extrapolated too far, especially if the bore is not very efficient. It follows, too, that the efficiency of a bore can be gauged by the magnitude of the well-loss constant C. Walton (1962) states that for a properly developed and designed well, the value of C is generally less than 5 sec^2/ft^5. Values between 5 and 10 sec^2/ft^5 indicate mild clogging, and clogging is severe when C is greater than 10 sec^2/ft^5.

It will be evident from the foregoing discussions that single-rate pumping tests are of limited value in assessing bore performance, except for the particular rate used, and they may give little indication of the efficiency of the bore. Their main value is in revealing aquifer conditions, as will be discussed later. On the other hand, multiple-stage tests are of particular value in assessing bore performance and in calculating the effect of pumping at higher than those used in the test.

In order to gain as much relevant information on both the aquifer conditions and the bore performance, the Water Conservation and Irrigation Commission, New South Wales, carries out a multiple-stage test, and a twenty-four hour single-rate test, followed by recovery measurements, as standard procedure in those cases where long-term pumping at high rates is proposed, e.g. town, industrial, or irrigation supply. For stock-water bores, only a single-rate pumping test is undertaken.

1.2.3 Pumping-in Tests

Pumping-in tests have more application where it is desired to determine the hydraulic properties of an aquifer, (see section 1.3.2.5) but in some instances they are applied with the object of determining only bore or well performance. The only common example of such circumstances in the author's experience is where it is proposed to use a bore or well to dispose of effluent, usually from septic tanks, and it is desired to determine the acceptance rate. Almost invariably it has been found that contractor's tests are quite inadequate and insufficient data has been obtained to allow analysis.

The principles involved are essentially the same as those already discussed, the only difference being that the water-level is being built-up by in-flow instead of draw-down by discharge.

1.3 TESTS TO DETERMINE HYDRAULIC PROPERTIES

In quantitative groundwater hydrology studies, pumping tests are of particular value because they afford a means of establishing the hydraulic properties of an aquifer in situ. The properties usually determined are the coefficients of transmissibility, permeability, and storage. Where leakage occurs through confining beds, the coefficient of vertical permeability of the confining beds can also be determined. If barrier boundaries or recharge boundaries are present in the vicinity they, too, are evidenced in pumping test data.

The most common method of determining the hydraulic properties of an aquifer depends on the analysis of time-drawdown data. For this purpose, a bore or well is pumped at a constant discharge, and drawdown in the pumped bore or well, and in any observation bores, is measured at appropriate time intervals. (If at least one observation bore is not available an aquifer test is limited in scope, and normally only transmissibility can be determined).

It is apparent, of course, that this type of pumping test is analogous to the single-rate pumping test previously described to determine the performance and yield of a bore. However, in carrying out a test to establish hydraulic properties, the efficiency of the bore is of little consequence (provided a reasonable pumping rate is available) since the information sought from the pumped bore is the rate of drawdown, not the actual drawdown, for a given discharge. However, the significance of equation (4), which shows the various possible components of drawdown, should be kept in mind because it is often necessary to adjust drawdown data before they can be employed in analysing test results.

1.3.1 Factors affecting time-drawdown relationships.

Of the various possible components of drawdown given in equation (4), the measurementsof component s_a (drawdown due to laminar flow through the aquifer) is the main object of the test. If the aquifer reasonably conforms with the assumptions of the theory, the rate of drawdown in the pumped bore and in observation bores will be the same and a straight-line graph of drawdown against log time, as described for the modified mon-equilibrium method, will show the same gradient in each case. On the other hand, the component s_w (drawdown due to "well-loss") occurs only in the pumped bore and would not be evident in an observation bore unless it were immediately adjacent to the pumped bore.

However, a number of other factors may affect the time-drawdown relationship and it is necessary to be aware of these to properly interpret test data. They will be briefly considered in the following paragraphs.

1.3.1.1 Water level trends prior to pumping.

A factor not included in equation (4) is that of water level trends in evidence before pumping starts. These may be due to such causes as the effects of previous pumping, pumping of other facilities in the vicinity, rising or falling water table conditions (in shallow water table situations, evapotranspiration can have an effect here), or the effect of atmospheric pressure changes in the case of confined aquifer. Consequently, observations of water level should, if possible, be maintained for some time (preferably a few days) prior to pumping so that any discernible trends can be allowed for in the subsequent test. (A useful review of causes of water level fluctuations is given by Todd (1959).

1.3.1.2 Variations in atmospheric pressure.

In the case of the effect of atmospheric pressure, the barometric efficiency of a bore tapping a confined aquifer can be determined from observations prior to pumping by applying the equation:-

$$B.E. = \frac{\Delta W}{\Delta B} \times 100 \qquad \ldots(6)$$

where B.E. = barometric efficiency in %

ΔW = change in water level due to a change in atmospheric pressure, in feet

ΔB = change in atmospheric pressure, in feet head of water.

Adjustments in drawdown data can then be made for the effects of any changes in atmospheric pressure during the test. (It is evident that water level trends as described above will also affect tests to establish the performance of a bore. However, because of the economic aspects of contract boring, it is rarely practicable to maintain observations of water level variations for any appreciable time after completion of the bore and prior to testing. There is the danger, then, that if a bore has a reasonable barometric efficiency, and a significant increase in atmospheric pressure occurs fairly rapidly during the test, the increase in drawdown, which could amount to a foot or more, could be interpreted as a boundary condition).

1.3.1.3 Partial penetration of the aquifer

A further possible cause of error is the factor designated in equation (4) as s_p, i.e. the drawdown due to the effect of partial penetration. If the pumping bore or nearby observation bore(s), or both, only partially penetrate the aquifer, varying effects can be produced depending on the relative positions of the sections of the aquifer exposed to the bores. For example, if pumped and observation bores are both open to either the top or the bottom of an aquifer, the drawdown in the observation bore will be greater than for fully penetrating conditions; if the pumped bore is open to the top and the observation bore to the bottom, or vice versa, the drawdown in the observation bore will be less than for fully penetrating conditions. These effects are due to the distortion of the cone of depression because of the vertical flow component induced. However, the effect diminishes with distance, and Walton (1962) quotes the following equation from Butler to give the distance r_{pp} beyond which the effect of partial penetration is negligible:-

$$r_{pp} = 2m\sqrt{P_h/P_v} \qquad \ldots(7)$$

where m = saturated thickness of aquifer, in feet

P_h = horizontal permeability of aquifer, in g.p.d./ft^2

P_v = vertical permeability of aquifer, in g.p.d./ft^2

He also gives formulae and factors for adjusting observed drawdowns in observation bores within this distance, but points out that "at best, adjustments for partial penetration by any method can be considered only approximate because the ratio P_v/P_h is never precisely known."

1.3.1.4 Boundary conditions

In developing equations to compute hydraulic properties of aquifers it is assumed that the aquifer is infinite in extent, and it is obvious that this cannot be. Indeed, it is common for pumping tests to show evidence that a boundary condition is present within the area of influence of the bore or well.

If the boundary is a barrier (e.g. the edge of a valley, or a sudden lensing out of the aquifer), when the cone of depression reaches it it will cause an increase in the rate of drawdown. If it is a recharge boundary (e.g. a river or other surface body of water), the rate of drawdown will decrease until eventually equilibrium conditions occur, i.e. recharge balances discharge.

In the general run of aquifer tests, barrier boundaries are by far the more common. They are usually readily discernible if the modified non-equilibrium method is used, especially if the boundary is a fairly abrupt one, when it will cause a sudden doubling of the slope of the straight-line time-drawdown graph. The reason for this is best understood by considering the "image-well" theory described by Ferris (1959). In essence, this states that the effect of a barrier boundary on drawdown in a bore is the same as would occur if the aquifer were infinite and a bore having the same discharge were located on the opposite side of the boundary and at the same distance as the real bore. The same principle applies for a recharge boundary except that the image bore is assumed to be recharging instead of pumping.

Pumping test data can be used to locate the position of a barrier, but to do this it must be evident in the data from at least three observation bores.

The only other factor that will be considered as affecting time-drawdown relationships is drawdown due to dewatering portion of the aquifer, and this will be discussed in the section dealing with determining hydraulic properties of unconfined aquifers. (1.3.2.4).

1.3.2 Methods based on time-drawdown relationships

1.3.2.1 Modified non-equilibrium

Because of its simplicity of application and interpretation, the modified non-equilibrium method is the most popular method of analysing aquifer test data, especially for combined aquifers. The principles involved have been given in section 1.2.1.

Drawdown data from the pumped bore and any observation bores are first adjusted, if necessary, (see section 1.3.1) and then plotted against the logarithm of time. The adjusted drawdown data from the pumped bore or well will then consist of the drawdown components due to "well-loss" and "aquifer-loss" (i.e. s_w and s_a of equation 4). Of these, the "well-loss" is constant for a given discharge, whereas the "aquifer-loss" increases in direct proportion to the logarithm of time and thus causes the slope in the straight-line portion of the graph. In observation bores, of course, only drawdown due to "aquifer-loss" is represented. If the aquifer conforms with theory, the slopes of the straight-line graphs for the pumped bore and observation bores will be the same (see fig. 7, section 3).

The coefficient of transmissibility is calculated by using the slope of the straightline portion of the graph in the following equation:-

$$T = 264Q/\Delta s \qquad \ldots(8)$$

where T = coefficient of transmissibility, in g.p.d./ft

Q = discharge, in g.p.m.

Δs = difference in drawdown per log cycle in ft (i.e. the slope of the straight line graph).

To derive the coefficient of storage, the straight-line portion of the graph of data from an observation bore is extrapolated backwards until it intersects the zero-drawdown axis. The time given by this intersection is then used in the following equation:-

$$S = Tt_0/4790\ r^2 \qquad \ldots(9)$$

where S = coefficient of storage, as a fraction

T = coefficient of transmissibility, in g.p.d./ft

t_0 = intersection of the straight-line slope with the zero-drawdown axis, in minutes

r = distance from the pumped bore to the observation bore, in feet.

1.3.2.2 Type curve

In developing the modified non-equilibrium method considered in the previous section, Cooper and Jacob (1946) were careful to point out that "the method is not applicable in some cases and it supplements, rather than supersedes, the type curve method". Consequently, if there is any doubt as to whether or not the data lends itself to interpretation by the time-drawdown graph, the type curve method should be employed. Certainly one difficulty that often presents itself when it is desired to determine coefficient of storage by the former method is that when the slope of the straight-line is small the intersection with the zero-drawdown axis may be poorly defined.

The type curve method is based on Theis' non-equilibrium formula given in section 1.2.1. This may be written in the form

$$s = (114.6\ Q/T)\ W(u) \qquad \ldots(10)$$

$$\text{where} \quad W(u) = \int_u^{\infty} e^{-u}\, du,$$

and is termed by Wenzel (1942) as the "well function for non-leaky artesian aquifers".

Wenzel (1942) provides a type-curve giving values of W(u) against u on log graph paper. However, the method described by him to obtain a matching curve is rather tedious, since it involves plotting values of s (drawdown in an observation well) against r^2/t (where r is the distance of the observation bore from the pumped bore, and t is the time in days from when pumping began until s was measured). Walton (1962) has since provided a more convenient type curve, giving values of W(u) against 1/u on log graph paper, and to obtain a time-drawdown field data curve to superpose on the type curve it is necessary only to plot drawdown s (in feet) against time t (in minutes) on log paper of the same scale as that used for the type curve. When the time-drawdown curve is matched to the type curve, match point co-ordinates W(u), 1/u, s and t are substituted in equation (10) and in the equation

$$u = 2693\ r^2\ S/Tt \qquad \ldots(11)$$

to determine T and S. (u, r, S, s, T and t are as defined in section 1.2.1).

1.3.2.3 Type curves for leaky confined aquifers

In some pumping tests of confined aquifers, the confining formation is sufficiently permeable to allow leakage from an overlying or underlying formation into the aquifer being tested and thus affect the time-drawdown relationship. Walton (1960,a) developed a type curve method for dealing with the analysis of the pumping test data for these conditions.

The leaky confined aquifer formula is given as

$$s = (114.6\ Q/T)\ W\left(u, \tfrac{r}{B}\right) \qquad \ldots(12)$$

where $u = 2693r^2S/Tt$ as in equation (11)

s, S, Q, T and r are as defined in section 1.2.1; $W\left(u, \frac{r}{B}\right)$ is termed "well function for leaky artesian aquifers"; and

$$\frac{r}{B} = r/\sqrt{T/(P'/m')} \qquad \ldots(13)$$

P' = coefficient of vertical permeability of confining bed, in g.p.d./ft^2

m' = thickness of confining bed through which leakage occurs, in ft.

Walton (1962) presents a family of leaky artesian type curves, based on values of W (u, r/B) in terms of the practical range of u and r/B, and by plotting values of W (u, r/B) against values of 1/u on log graph paper. By plotting drawdown against time on log graph paper of the same scale and matching the time-drawdown curve thus obtained to the type curve of best fit, co-ordinates W (u, r/B), 1/u, s and t of a match point can be read off. The values of these co-ordinates are then substituted in equations 11, 12 and 13 to determine the hydraulic properties of the aquifer and the confining formation.

1.3.2.4 Non-equilibrium methods applied to unconfined aquifers

Although the non-equilibrium formula (equation 1) was derived for confined aquifers, both the modified non-equilibrium and type curve methods of analysing pumping test data, (sections 1.3.2.1 and 1.3.2.2) have application to unconfined aquifers under certain conditions. As outlined in section 1.2.1.2, after pumping has been in progress for a period sufficient to make the effect of gravity drainage small, the time-drawdown relationship for a bore or well in an unconfined aquifer (i.e. "water-table" conditions) conforms reasonably with the non-equilibrium formula.

In dealing with this problem, Boulton (1954) considers that whether or not the non-equilibrium formula can be applied depends on the distance r of the observation well from the pumped well, the hydraulic properties of the aquifer, the saturated thickness, m, of the aquifer, and a dimensionless "time-factor". He implies that if the time factor is greater than 5, and r is between 0.2m and 6m the non-equilibrium formula can be applied with sufficient accuracy for practical purposes. From this, and applying the proviso that r be between 0.2m and 6m, Walton (1962) derives the equation

$$t_{wt} = 37.4\, S_y\, m/P \qquad \ldots(14)$$

where t_{wt} = approximate time after pumping starts when application of non-leaky artesian formula to water table conditions is justified, in days

Sy = specific yield, fraction

P = coefficient of permeability, in g.p.d/ft.^{2}

m = saturated thickness of aquifer, in feet.

It should be noted that in using drawdown data to determine hydraulic properties of unconfined aquifers it is necessary to adjust the data to allow for the fact that the saturated thickness of the aquifer has been decreased by the pumping. The adjusted value of drawdown is given by the following formula, due to Jacob (1944):-

$$s' = s - (s^2/2m) \qquad \ldots(15)$$

where s' = drawdown that would occur in an equivalent non-leaky confined aquifer, in ft.

s = observed drawdown under water table conditions.

m = initial saturated thickness of aquifer, in ft.

In theory, the coefficient of storage of an unconfined aquifer is analogous to the specific yield. However, because pumping tests are mostly relatively short, generally not exceeding 24 hours, and because gravity drainage in this period is normally incomplete, the coefficient of storage derived from the test data is usually less than the specific yield. This can be a source of error in quantitative applications. Furthermore, the coefficient of storage so derived applies essentially to that portion of the aquifer dewatered during the test, and it does not follow that it can be applied to the remainder of the aquifer.

1.3.3 Methods based on distance-drawdown relationships.

1.3.3.1 Non-equilibrium methods.

Walton (1960,b) points out that distance-drawdown data complement time-drawdown data and can be used to great advantage in the interpretation of early drawdown measurements. He considers that interpretations of pumping test data based solely on time-drawdown graphs are weak, and that logarithmic and semi-logarithmic graph paper magnify the importance of early time-drawdown data, sometimes out of all proportion to their value.

Distance-drawdown relationships are inherent in the non-equilibrium formula, and can be analysed by either the modified non-equilibrium method or the type curve method in similar fashion to time-drawdown data.

In the modified non-equilibrium method, drawdowns observed at the end of a specified pumping period in two or more observation bores are plotted against the logarithm of the respective distances on semi-log graph paper. A straight-line graph should be obtained, and this line is extrapolated to intersect the zero-drawdown axis. The slope of the straight line is then used to determine the coefficient of transmissibility and the zero-drawdown intercept is used to calculate the coefficient of storage. The equation for coefficient of transmissibility is

$$T = 528\ Q/\Delta s \qquad \ldots(16)$$

where T = coefficient of transmissibility in g.p.d./ft

Q = discharge, in g.p.m.

Δs = drawdown difference per log cycle, in ft.

The coefficient of storage is given by equation (9), $S = Tt/4790\,r_o^2$.

Walton (1962) supplies type curves for matching with distance-drawdown curves for both leaky and non-leaky confined aquifers. In the type curve method, values of drawdown measured at the same time in two or more observation bores are plotted against the square of the respective distances on logarithmic paper of the same scale as that used for the type curve. The distance-drawdown curve so obtained is matched to the type curve, and match-point co-ordinates are substituted in the same equations as given for the appropriate time-drawdown type curve methods to calculate the hydraulic properties of the aquifer.

1.3.3.2 Equilibrium methods

The so-called equilibrium methods to determine permeability are, in effect, based on distance-drawdown relationships. Wenzel (1942) shows the development of the general equilibrium formula, based on Darcy's Law and the hydraulic gradient at points of the cone of depression caused by pumping. The formula for water-table conditions, after Thiem, is as follows:-

$$P = \frac{2.303\ Q \log_{10} \frac{r_2}{r_1}}{\pi (h_2+h_1)(s_1-s_2)} \qquad \ldots(17)$$

where P = coefficient of permeability,

Q = discharge of the pumped bore,

h_1 = saturated thickness of aquifer at the near observation bore at distance r_1 feet from the pumped bore, in feet,

h_2 = saturated thickness of aquifer at the far observation bore at distance r_2 feet from the pumped bore, in feet,

s_1 & s_2 = drawdown in the near and far observation bore, respectively, in feet.

For confined aquifer conditions, (h_2+h_1) is replaced by 2m, where m is the average thickness of saturated water-bearing material at the two observation points. The formula for confined aquifer conditions then becomes

$$P = \frac{2.303\ Q \log_{10} \frac{r_2}{r_1}}{2\pi m\ (s_1-s_2)} \qquad \ldots(18)$$

In applying the equilibrium method it is assumed that the aquifer is infinite and homogeneous, and that the water table (or piezometric surface, in the case of a confined aquifer) and the impervious base of the aquifer are both horizontal. In an elaborate pumping test in which 81 observation bores were used, Wenzel (1936) found that consistent results could be obtained by using the following procedure:- (i) using for the drawdown, s_1, the average of the drawdowns at distance r_1 on opposite sides of the pumped bore, preferably up-gradient and down-gradient, and similarly for the drawdown s_2 at distance r_2; (ii) using only those drawdowns that are obtained from observation bores situated on a straight line through the pumped bore; (iii) using only the drawdowns in observation bores situated within that part of the cone of depression which, by the end of the period of pumping, has reached approximate equilibrium in form; (iv) using only drawdowns in observation bores situated sufficiently far from the pumped bore that the effects of vertical groundwater movement, changes in permeability due to bore development, and, if applicable, the failure of the bore to penetrate the entire thickness of water-bearing material, are inappreciable; and (v) using drawdowns obtained at more than two distances from the pumped bore. (For water-table conditions it is also usually recommended that the drawdown in observation bores should not exceed 1/10 of the saturated thickness of the aquifer).

To provide for these factors he modified the Thiem formula and termed it the "Limiting formula". He also developed the "Gradient formula", based on Darcy's Law and flow through concentric cylindrical sections. These formulae were applied by the present author in field permeability tests of the alluvium in the Hunter valley, New South Wales (Williamson, 1958).

It will be noted that time does not enter into equilibrium formulae. However, it is nevertheless involved, since the method requires pumping to be continued until the cone of depression reaches approximate equilibrium in form, i.e. although the water level may still be falling (as is allowed for in the non-equilibrium method), the rate of fall is required to be sensibly the same at each observation point. In point of fact, Wenzel (1942) shows that for a large time of discharge the equilibrium and non-equilibrium formulae are essentially equal.

1.3.4 Methods based on time-discharge relationships

In some instances, mostly when testing a bore for yield rather than to determine aquifer properties, pumping tests are carried out by maintaining a constant drawdown. In this case, discharge decreases with time.

The usual circumstance dictating the use of this method is that it is not practicable to measure drawdown, e.g. because there is insufficient annular space between the pump column and the casing. There are more difficulties involved in conducting this type of pumping test and it is usually employed only because there is no alternative.

Jacob and Lohman (1952) developed the theory for the discharge of a bore of constant drawdown, and although they were dealing with artesian (flowing) bores which were not being pumped, the principle is the same for constant drawdown conditions created by pumping.

If it is desired to establish values for the coefficients of both transmissibility and storage, the mathematical treatment is relatively involved. However, if only the coefficient of transmissibility is required it can be obtained by a modified method which involves plotting values of $1/Q$ against $\log_{10}t$. This gives a similar graph to that obtained for time-drawdown data in the modified non-equilibrium method previously described (1.3.2.1). The coefficient of transmissibility is then given by the formula:-

$$T = 264/s_c \Delta\left(\frac{1}{Q}\right) \qquad \ldots(19)$$

where T = coefficient of transmissibility in g.p.d./ft,

s_c = constant drawdown, in feet,

$\Delta\left(\frac{1}{Q}\right)$ = difference in $\left(\frac{1}{Q}\right)$ per log cycle (i.e. the slope of the straight line graph),

Q = discharge in g.p.m.

Jacob and Lohman also found that sufficiently accurate values of T could be obtained by the recovery method. Using semi-log paper, values of residual drawdown, s, were plotted on the linear scale against corresponding values of t/t' on the log scale (t = time since discharge began; t' = time since discharge stopped), and a straight line graph obtained. The change in residual drawdown, s, per log cycle of t/t' gives the slope of the line. The normal modified non-equilibrium formula (Equation 8) is then used to determine T.

1.3.5 Pumping-in tests

It is sometimes found more practible to determine hydraulic properties of a formation by means of pumping-in tests. The usual reasons for this are that it is required to establish these properties for material which is not yet saturated (but which subsequently will be), or not saturated to a sufficient thickness to allow pumping out, or there is insufficient available drawdown (in the case of a confined aquifer).

In any event, the principles involved in analysing pumping-in test data are essentially the same as for discharge tests, the main difference being that there is a cone of build-up due to the recharge, rather than a cone of depression due to pumping. However, a factor which often leads to difficulties in such tests is that any suspended material in the water being pumped in will cause clogging of void spaces and can give rise to considerable error.

Ahrens and Barlow (1951) give details of procedures of various pumping-in tests, some of which employ only one drill hole. They point out that the one-hole method is not as reliable as when three or more observation holes are used. In the latter type of test they use the equilibrium method to analyse the test data.

PUMPING TESTS

PART 2

EQUIPMENT & PROCEDURES

by

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INDEX

PUMPING TESTS

PART 2 : EQUIPMENT AND PROCEDURES

2.1 Introduction

In essence, a pumping test consists simply of pumping a bore or well, measuring the rate of discharge and measuring the effect of the pumping on the water level in the pumped bore or well and at any observation points. Nevertheless, if satisfactory results are to be obtained, considerable care is necessary, and there are many pit-falls to be avoided.

The following notes outline the equipment required, precautions to be taken, and procedure recommended.

2.2 Equipment

2.2.1 Pumps

For low-yielding bores, e.g. stock-water bores, a draw-plunger or flush-cap type pump will usually suffice. The larger sizes of these pumps may yield, say, 5000-6000 gph. but they are not particularly satisfactory for accurate pumping tests at such rates because of the water level fluctuations caused by their reciprocating action.

For high-yielding bores, a deep-well turbine or axial flow pump is usually required, but centrifugal pumps can have application where the drawdown level is less than about 20 feet, or, for example, if the pump can be installed down a well. In any event, it is necessary that there be sufficient annulus between the pump column and the bore casing to allow measurement of the water level.

The motor must be of adequate horsepower, and direct (for electric motors) or universal shaft drive is desirable. Belt drives can cause considerable problems in maintaining constant discharge.

The pumping equipment used by the W.C. & I.C. for testing high-yielding bores consists of two 7 inch diameter Pomona pumps capable of 35,000-45,000 gph. against heads of up to 150 ft and two 5 inch Ornel pumps capable of delivering 40,000 gph. against heads of up to 130 ft. Diesel motors are as follows: 1 x 52 hp. International; 1 x 54 hp. Dorman (3 cylinders); 1 x 96 hp. Dorman (6 cylinders); and 1 x 76 hp. Lister (6 cylinders). For lower-yielding bores there are two flush-cap pumps, 5 and 4½ inches diameter respectively, capable of 5000-6000 gph., while each boring plant carries either a 2¾ or 3¼ inch draw-plunger pump fpr testing stock bores.

2.2.2 Measuring discharge

The first requirement is that there be a valve on the pump discharge pipe to allow control of discharge, but there must also be provision for accurately measuring such discharge. The most convenient, and probably the most accurate method is to use a water-meter, in which case the control valve is placed on the outlet side to ensure that the meter is always full. If an orifice plate is used, the valve is placed on the pump-side and should be at least 10 pipe-diameters from the outlet. (It is advisable that the nipple leading to the manometer tube be of smaller diameter than the tube to make for more stable water levels in the tube).

Other methods may be used to measure discharge, e.g. V-notch or rectangular weirs, the distance to a free fall of 12 inches from a horizontal pipe, and so on, and each has criteria to ensure maximum accuracy. However, in general, the methods cited are in descending order of reliability and accuracy. For low yields, timing with a drum of known volume can be satisfactory.

2.2.3 Measuring water-level

Various devices can be employed for measuring water-level. In a pumped bore the device must be small enough to fit freely into the annular space between the discharge column and the casing, but observation bores usually allow more latitude in this regard.

Electrical probes are probably the most commonly used device and can be either single or dual wire. The latter is the more satisfactory because it does not require establishing a circuit through the bore casing. (Television-aerial dual-flex wire is useful for improvising electrical probes because it is wide and flat, and can be easily marked). A milliammeter or a torch globe is incorporated in the circuit to register when the probe contacts the water. Provision should be made for sufficient shielding and separation of the probe terminals to prevent them remaining wetted.

Air-lines are often used in a pumped bore but suffer from the disadvantage of not allowing very accurate results. The air-line consists of a tube of known length down the bore, and at the surface a pressure gauge and a tyre-valve stem are fitted to a T-piece. Air is pumped through the tube, using a tyre-pump, and when pumping is stopped the pressure gauge records the pressure required to balance the column of water above the outlet of the tube. It is essential that the level of the base of the tube be known, so the tube should be fixed to the pump column as the latter is inserted. Inserting the tube later is usually not satisfactory.

Various other methods of using a weighted tape (steel), e.g. chalking the tape, using an air-bell to cause a whistle, etc. are usually not practicable when measuring in a pumped bore, but can have application in observation bores. Float type devices can also be very useful in observation bores.

2.3 Precautions

It should be ensured that the aquifer in the pumped bore is stabilized so that sand or silt do not enter during pumping. The motor should also be checked to ensure that it is operating satisfactorily. Make sure sufficient fuel is on hand.

A log of the bore should be available, as well as complete detail of casings, screens, etc. The same applies to observation bores, and it should also be ensured that these fully penetrate the aquifer, freely reflect water level fluctuations (check beforehand by adding water) and are at a sufficient distance to avoid the effect of partial penetration if the pumped bore does not fully expose the aquifer. (See equation 7, section 1.3.1.3).

Provision must be made to prevent the pumped water re-entering the system. This is usually not a problem with confined aquifer but can cause considerable difficulty in unconfined or water-table conditions, especially if the overlying formation is very permeable. In some cases it may be necessary to provide piping or impervious channelling for hundreds of feet from the bore or well. Each case must be assessed on its circumstances and conditions.

Make sure that everyone involved in the tests knows what they should do, how they should do it, and that they have adequate forms on which to record data.

2.4 Procedures

2.4.1 Water level prior to pumping

To obtain the most reliable data, observations of water level should be maintained for a day or so prior to pumping. The factors that may effect water level are mentioned in section 1.3.1.1, and if they are evident, provision will have to be made for their effect. However, more often than not, it is not practicable to maintain prior observations for any appreciable period, especially if the bore is being constructed under contract for production rather than investigation purposes.

If other bores or wells are being operated in the vicinity, and it is not practicable to have them stopped for a day or so prior to the test, they should, if possible, be kept operating at a constant rate prior to and during the test. Water-level observations prior to the test are then virtually essential so that provision can be made for the effect of this pumping.

2.4.2 Single-rate tests

On the day prior to the test, short pumping runs of ¼ to ½ hour should be made at various rates to assess the general performance of the bore. Water level measurements during these runs will give some indication of the likely time-drawdown relationship. From the information so gained, and taking into account any other factors involved, the appropriate rate for the main test is decided. The requisite motor speed and discharge valve settings are also determined from these tests.

On the day of the test, water level is taken prior to commencing pumping and should show complete or almost complete recovery from the effect of the previous pumping. The times of all measurements and of commencement or cessation of pumping must, of course, be recorded.

Once pumping commences, it is a good general principle to measure water level as frequently as is practicable while the level is changing rapidly. As the rate of fall diminishes, the time interval between readings can be lengthened. Bear in mind, too, that in analysing the data the scale of time will be logarithmic. Particular attention should be given to obtaining early time-drawdown data, especially if it is proposed to use a type-curve method of analysis. Suitable time intervals would be as follows:- 1, 2, 3, 4, 5, 6, 8, 10, 12, 15, 20, 25, 30, 40, 50, 60, 75, 90, 105, 120, 150, 180, 210, 240, 270, 300, and thence hourly, assuming a 24 hour test.

No doubt the biggest "bug-bear" in conducting pumping tests is maintaining constant discharge, particularly in cases where the transmissibility is relatively low, with consequent high rates of drawdown. The problem is that as the drawdown increases, the pumping lift increases and the discharge decreases. And, of course, as the discharge decreases the drawdown decreases! An example of this is shown at Figure 4 (Section 3.3) and it can lead to considerable error in interpretation and analysis of data. (This example also illustrates the importance and value of recovery data.)

During pumping, frequent measurements should be made of the discharge rate. If practicable, adjustments should be made to maintain the rate sensibly constant, preferably by means of the discharge valve rather than attempting to alter the speed of the motor. All meter readings or other measurements of pumping rate should be recorded, as well as any variations or adjustments in pumping rate.

There is need for an adjustable device to maintain the desired constant discharge for this work. The only one known to the author is the "Flo-stat" unit, but the largest currently available is rated to deal only with discharges of up to 15,000 gph. Any suggestions or information in this regard would be appreciated.

If at all possible, recovery measurements should be taken after a single rate pumping test. A similar time interval between readings as suggested above for drawdown measurements is desirable, although after say 8 hours the interval could be increased to 2 hours.

It is important to note that the recovery data used in analysis is not simply the difference between the drawdown-level at the cessation of pumpings and the water-level recorded at a certain time. The theory of recovery is that the water level behaves as if the bore continued to discharge at the constant rate, but that at the time pumping actually ceased a flow equivalent to the pumping rate was introduced into the bore. Consequently, an adjustment, equivalent to the additional drawdown that would have been caused for the particular time had pumping been continued, must be added to the recorded recovery in water level at that time. The adjustment is readily determined graphically by extrapolating the straight line time-drawdown graph beyond the time at which pumping ceased and reading off the additional drawdown that would have occurred for the respective times of the recovery measurements. In Figure 5 (Section 3.3.) both recorded and adjusted recovery have been shown to illustrate the necessity of the adjustment.

During pumping, when a "breathing-space" becomes available between measurement times, it is a good practice to plot the time-drawdown data on semi-log graph paper and then plot each new measurement. This will allow the water level behaviour during the test to be assessed far more satisfactorily than is possible by scrutinizing recorded data.

2.4.2. Multiple-stage tests

If a multiple-stage test is proposed it should precede a single rate test, since it will give a more reliable indication of the appropriate pumping rate for a single-rate test than can be gained from short preliminary test runs. Also, since a multiple-stage test may consist of 3 or 4 stages of only two hours each, recovery should be effected in time to run a single rate test the following day, if required.

Short test runs should be carried out on the day preceding a multiple stage test to indicate the likely maximum yield. The figure decided on for the latter should then be divided into the same number of approximately equal amounts as the number of stages proposed for the test. It is not essential that these be strictly adhered to in the test, they are merely a desirable distribution of pumping rates. Motor speed and discharge valve settings appropriate to these rates should be ascertained, if practicable.

In conducting the test, the procedure is essentially the same as for a single rate test during the first stage, and the same time interval of measurement is desirable. At the end of the period selected as being appropriate per stage, say 2 hours, the discharge is abruptly increased to the next rate. Any adjustment in the discharge rate should be done within the first few minutes, since this data is not normally usable anyway. The same interval of measurements is repeated as for the first stage except that readings taken at less than say 5 minutes nedd not be as frequent, in fact in most cases they are not necessary.

In carrying out multiple stage tests it is important to keep track of the water level behaviour by maintaining a semi-log time-drawdown graph. If the aquifer is confined (multiple stage tests have but limited application in unconfined aquifers) and the conditions allow application of the modified non-equilibrium method, time-drawdown data from the first stage should give a straight line graph. The slope of this line is directly proportional to the discharge, so that the slope of the line for subsequent discharges can be calculated. By thus knowing the slope of the line that time-drawdown data should conform to for the discharge of subsequent stages, it can readily be determined whether pumping has been maintained for a sufficient period during a particular stage. This is important because as the length of the log-time scale becomes compressed with increasing time, it is often difficult to be sure whether the water-level behaviour has settled down after the abrupt increase in discharge.

This procedure is repeated for subsequent stages until the end of the test. Recovery measurements after multiple stage tests do not lend themselves to analysis, so need not be taken.

PUMPING TESTS

PART 3

ANALYSIS AND INTERPRETATIONS

by

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INDEX

PUMPING TESTS

PART 3 : ANALYSIS AND INTERPRETATION

3.1 Introduction

In the following sections, examples of analyses of pumping test data carried out by the author in recent years are presented. They have been selected for their instructive value in interpreting data.

3.2 Determining Performance and Yield

The following account is of the analysis of data from Bore No. 14976, a 10 inch diameter screened and developed bore in the alluvium of the Lachlan River Valley, near Engowra, N.S.W. The bore was constructed by the W.C. & I.C. to a depth of 350 ft and the main aquifer consisted of sands and fine to medium gravels from 197–344 ft. The bore was backfilled to 343 ft and a total of 32 ft 6 ins of 8 inch diameter screen inserted in three selected zones. 8 inch diameter casing blanks were used between the screens, and the top of the top screen is 253 ft from the surface. The static water level was 20 ft at the time of testing, thus allowing 233 ft available drawdown.

The pumping tests consisted of the following:- (a) a four stage test at rates of 11,400 gph., 19,900 gph., 29,000 gph. and 36,600 gph. respectively, (each stage was of two hours duration); and (b) on the following day, a 16 hour single rate test at 36,600 gph. followed by observation of recovery levels. (Recovery levels are not taken after multiple stage tests because they do not lend themselves to analysis).

The modified non-equilibrium method of analysis was used to analyse the data obtained. Figure 1 shows the graph of drawdown against log time for the multiple stage test, and Figure 2, the graph for the single-rate test and the recovery data.

From the multiple-stage test data, the drawdown at 100 minutes (a period selected for convenience) for each pumping rate is calculated. The 100 minute drawdown for 11,400 gph. is given directly from Figure 1 as 3.83 ft but to obtain the 100 minute drawdown for 19,900 gph. it is necessary to allow for the effect of the previous pumping. The straight line graph through the drawdown time points for the first stage is extrapolated, and the difference in drawdown between the extrapolated line and the data for 19,900 gph. is determined for a time 100 minutes from the time of commencement of pumping at this rate. This difference Δs (2.96 ft) is the 100 minute incremental drawdown, and when added to the drawdown of 100 minutes for the first stage (3.83 ft) gives the 100 minute drawdown for the pumping rate of the second stage (6.79 ft).

Since the slope of the straight line graph is governed by the transmissibility of the formation, once it is determined for the first pumping rate the slope for other rates is given by direct proportion between this and the new pumping rate. (This also permits a check on the validity of the trend of the time-drawdown curves given by the data in subsequent stages, and in some cases may indicate the desirability of longer pumping periods per stage.) The calculated slope is drawn in to fit the data towards the end of the pumping period and then extrapolated beyond the pumping period in order to allow the incremental drawdown for the next stage to be determined.

The above procedure is carried out for each stage, and the data so obtained are tabulated as follows:

Discharge in gph.	Discharge in cusecs	100 min. Δs in ft.	Drawdown s, in ft. at 100 mins.	$\frac{s}{Q}$ (cusecs)
11,400	0.510	3.83	3.83	7.55
19,900	0.888	2.96	6.79	7.63
29,000	1.295	3.43	10.22	7.88
36,600	1.632	3.10	13.32	8.15
36,600	1.625	12.46	12.46	7.67

(The data in the last line are from the single rate test. It will be noted too, that in each case the discharge, Q, has been converted from gallons per hour to cusecs. This is done in order to obtain more convenient factors in the equation which will be derived from this data.)

Although not essential, it is well worthwhile to plot the 100 minute drawdown against discharge, on linear scales, as shown in Figure 3 a. This gives a good check on the conformity of the data and will sometimes indicate the possible source of error in any anomalies. In the case of confined aquifers, the more efficient the bore, the closer will the drawdown discharge curve approach a straight line. For bores of low efficiency, the slope of the tangent to the curve will increase rapidly with increasing discharge.

Values of $\frac{s}{Q}$ from the tabulated data are then plotted against the respective values of Q, as shown in Figure 3b, to derive the equation relating drawdown at 100 minutes to "aquifer loss" and "well loss", i.e. in the form $s = BQ + CQ^2$ (Equation 5, Section 1.2.2). The straight line of best fit is drawn to this plotted data. The intercept of this line with the s axis then gives the factor B, (7.27) and the slope of the line gives the factor C, (0.52). The drawdown s, at 100 minutes for <u>any</u> discharge Q, in cusecs, can then be calculated from the equation.

$$s = 7.27Q + 0.52Q^2$$

The single rate pumping test is carried out to give information on aquifer conditions. The semi-log plot of the data from this test is shown at Figure 2, and the drawdown data indicates that a boundary condition to have come into effect at about 110 minutes. This is evident by the change in slope of the straight line of best fit from 1.96 ft per log cycle to 3.59 ft per log cycle at this time. On the other hand, the recovery data do not conform with this, and show only the slope of 1.96 ft per log cycle. Unfortunately there is only one recovery measurement beyond 105 minutes so the evidence is not conclusive. However, on the principles that a boundary condition is not unexpected, and that, in any event, if an error is to be made it should be on the side of safety, the rate of drawdown of 3.6 ft per log cycle was adopted as a basis for calculating the rate for higher discharges when extrapolating beyond 100 minutes.

For irrigation bores, the practice adopted by the author in making recommendations of safe pumping rates is to allow for the effect of 10^5 minutes (approximately 70 days) continuous pumping. (For town water supply bores, 10^6 minutes or approximately 700 days, is more appropriate). The drawdown, s, at 100 minutes for a given discharge is calculated from the equation derived above, and the drawdown at 10^5 minutes is given by adding to s the drawdown for three additional log cycles. The amount per log cycle is given from the selected base figure by direct proportion of the respective discharges, e.g. for 50,000 gph., the drawdown per log cycle will be $(3.6 \times \frac{50,000}{36,600})$ ft $= 4.92$ ft.

By way of example the calculation for the drawdown at 70 days at a discharge of 50,000 gph. (2.23 cusecs) is as follows:-

$$\begin{aligned} s_{100\ min} &= 7.27\ (2.23) + 0.52\ (2.23)^2 \\ &= 16.2 + 2.6 \\ &= 18.8 \text{ ft.} \end{aligned}$$

then $s_{10^5 mins}$ = 18.8 + 3 x 4.92
= 18.8 + 14.8
= 33.6 ft.

In making recommendations of pump-intake settings for various pumping rates, and in estimating the pumping lift (e.g. for irrigation design purposes), a fair degree of judgement is still normally required. For example, the drawdown level at 70 days in the case cited is given by adding the static water level to the drawdown, i.e. (20 + 33.6) ft or 53.6 ft. However, the static water level may vary considerably with seasonal conditions and allowance will have to be made accordingly. For the most part, there is insufficient information to be specific in this regard, and an estimate must be made on the basis of local knowledge.

Another factor is the possibility that boundary conditions additional to any evident during the pumping test may occur. This suggests the desirability of allowing an additional margin for such unknown factors, and the extent of this margin must be assessed from consideration of the circumstances. As a general guide it is desirable to have the pump intake set at least 10 feet below the lowest anticipated drawdown-level, but this "safetly margin" should obviously be varied to suit the particular conditions.

As a further general guide, it is considered that the estimated drawdown-level at 10^4 minutes continuous pumping (i.e. about 7 days) is a reasonable figure for the pumping lift for design purposes, for the average irrigation bore. However, this, too, may be varied, depending on the proposed schedule of pumping.

In the case considered above, it was known that the owner of the bore wished to pump at a rate of the order of 80,000 - 100,000 gph., and since the maxium discharge achieved in the pumping test was only 36,600 gph. extrapolation to the required pumping rate puts considerable strain on the analysis. Consequently, in arriving at recommendations, it was allowed that the regional static water level could fall 10 ft., i.e. from 20 ft. to 30 ft., and that a further boundary condition could occur, thus causing the amount of drawdown per log cycle to double. The pumping lift for desigh purposes was based on the seven-day drawdown from the recorded static water level. Extracts from the estimated drawdown and drawdown-levels for various discharges are as follows:

Discharge g.p.h.	Drawdown in feet at 7 days	Drawdown in feet at 70 days	Drawdown level in feet at 70 days, assuming SWL of 30 ft.
60,000	46.8	58.6	88.6
80,000	64.2	80.0	110.0
100,000	82.3	102.0	132.0

Extracts from the final recommendations to the landholder are as follows:

Discharge g.p.h.	Design pumping Lift, ft. to surface	Pump intake settings, ft. below surface
60,000	77	90
80,000	94	110
100,000	112	140

The importance of recovery data

In Part I (Section 1.2.1) it was stressed that if, at all possible, recovery of water level should be observed after a single-rate pumping test. There does not seem to be sufficient emphasis on the value of recovery measurements in the literature on pumping tests, and the following examples are presented to show how significant recovery data can be.

(a) Figure 4 shows the semi-log graph of the drawdown and recovery data from a 24 hour pumping test on an 8 inch diameter screened bore, Bore No. 21041. The average pumping rate, based on the last 10 hours of the test, was 24,320 gph.

It will be seen that although there is a fair degree of scatter in the time-drawdown data, it does not appear to be an inordinate amount. If recovery data were not available, it would seem reasonable to draw a line of best fit through the plotted points and ascribe the scatter to minor water level fluctuations which are not uncommon where large drawdowns are being caused. Such a line would give a slope of the order of 4.5 ft. per log cycle.

On the other hand, the recovery data plot very well, and fall on a straight line of slope 7.1 ft/log cycle. When this is drawn in, it becomes apparent that a number of segments of the time-drawdown data also conform to this slope, as shown in the figure. It is evident, then, that the drawdown data are influenced by variations and adjustments in the pumping rate, and that the recovery data give the more correct basis on which to determine the increase of drawdown with time. If the former slope were used to extrapolate to 10^5 minutes from the 100 minute drawdown, it would underestimate the drawdown by about 8 feet. If it were used as a basis of calculation to determine the rate of increase in drawdown for higher discharges, it would lead to greater errors.

(b) The semi-log graphs shown at Figure 5 are based on drawdown and recovery data supplied by a landholder. He had sought advice, by mail, on establishing the safe yield of Bore No. 17152 and had been informed of the data required in order to allow analysis of the results of a pump test. Although the data supplied are not as complete as was requested, they are nevertheless instructive.

The bore is 150 feet deep, and aquifers are recorded in sandstones at 95-108 and 111-126 feet. The static water level was 60 feet. Pumping for about 12 hours at 18000 gph. caused less than 10 feet of drawdown, and the limited data conform reasonably to a straight line of slope 2.6 feet per log cycle. From this, it could be inferred that there would be little difficulty in maintaining a discharge of 18,000 gph. or even more.

However, in spite of this promising performance, the recovery data showed a pronounced retardation in recovery of water level suggesting that the aquifer had but limited storage which was being markedly depleted. (For purposes of illustration, the recovery from the drawdown-level is shown, as well as the adjusted recovery (see Part 2).

The owner was advised that the data were insufficient for satisfactory analysis but indicated that, because of the above factors, a discharge of 18,000 gph. could not be maintained. He was also advised that he should pump at as low a rate as was practicable, in order to conserve the supply.

Some months later, he wrote that he had continued pumping but the water level kept failing to recover and the bore would not then maintain a discharge of 7,000 gph.

(c) Figure 6 shows semi-log graphs of drawdown and recovery data from a pumping test of Bore No. 21047. This is a 6 inch diameter bore in the alluvium of Billabong Creek, near Holbrook, in N.S.W., and records an aquifer consisting of silty fine to medium sand from 206-245 feet. About 32 feet of 0.020 aperture-screen was installed from 243 feet upwards and the static water level was 18 feet at the time of testing.

The drawdown data plot on lines of very steep slope, initially 18.2 ft. per log cycle and finally 32.0 ft. per log cycle, and could be interpreted as indicating an aquifer of low transmissibility with a boundary condition coming into effect at about 120 minutes. However, the recovery data certainly do not conform to this interpretation. They show an initial rapid recovery, and fall on a line of slope only 2.0 ft. per log cycle, which indicates a high transmissibility.

The above situation, in conjunction with evidence that unstable clayey silt from above the aquifer had been induced through the screen, and other evidence of aquifer instability, was assessed as indicating that the drawdown behaviour and the poor performance of the bore were due to mechanical problems in development, and that the recovery data indicated a much better bore performance should be obtainable. On these grounds it was recommended that an 8 inch diameter production bore be constructed in the vicinity of the existing bore, rather than move to a new and unknown area. (Difficulty had been experienced in locating suitable aquifers in this valley).

The 8 inch diameter bore is under construction at time of this writing.

Determining hydraulic properties of an aquifer

Figure 7 shows the semi-log graphs of drawdown and recovery for a production bore, Bore No. 14725, and an observation bore at a distance of 50 ft. from it. The bores are in the alluvium of the Belubula River valley, downstream of Eugowra, N.S.W. Bore No. 14725 is of 8 inch diameter and was constructed to a depth of 67 ft. It encountered the valley basement rock at 63 ft. A sandy fine to medium gravel aquifer occurred from 46 to 63 ft. and was screened from 50 to 62 ft. with 7 inch diameter screen of 0.100 aperture. The pumping rate was 14,300 gph. and the observation bore was of six-inch diameter, with a five inch slotted liner in the aquifer.

Drawdown and recovery data conform well with each other in each case, and the slope of the straight line of best fit is 1.33 ft. per log cycle for both bores.

From equation 8 (Section 1.3.2.1.), the transmissibility, T, is given by

$T = (264Q / \Delta s)$ gpd/ft, where is the discharge in gpm, and Δs is the difference in drawdown per log cycle, in feet.

From the above data,

$$T = \left(264 + \frac{14,300}{60} \times \frac{1}{1.33}\right) \text{ gpd/ft.}$$

$$= 47,300 \text{ gpd/ft.}$$

The permeability,

$P = \frac{T}{m}$ gpd/ft^2, where m is the thickness of the aquifer, in feet then $\quad P = \frac{(47,300)}{17}$ gpd.ft.^{2}

$= 2,780$ gpd/ft.^{2}

From equation 9 (Section 1.3.2.1), the coefficient of storage, S, is given by

$$S = T\ t_o/4790\ r^2$$

where t_o = intercept of the straight-line time-drawdown graph with the zero-drawdown axis in minutes

r = distance of the observation bore from the pumped bore, in feet

then $$S = \frac{47300 \times 0.005}{4790 \times 50 \times 50}$$

$$= 1.97 \times 10^{-5}$$

The very small value of S is typical of confined aquifers

Leaky Aquifer conditions

Where leaky aquifer conditions are encountered, i.e. when the confining formation is sufficiently permeable to allow leakage from an overlying or underlying formation into the aquifer being tested, the effect on the time-drawdown relationship is fairly characteristic. When a semi-log plot of time-drawdown data is prepared, it is found that the data points form a curve for a much longer period thanis usually the case before conditions conform to the requirements of the modified non-equilibrium method. The curve gradually flattens out to more or less a straight line, often of very gentle slope, and the danger here is that if this line were used to determine transmissibility it would indicate values far too high. (By way of example, in some pumping tests carried out in the Hunter Valley alluvium (Williamson, 1958) leaky conditions were encountered. It was found that application of the modified non-equilibrium or straight-line method gave permeability values of up to 7,000,000 ft/yr. and it was shown that these could not be valid, and equilibrium methods gave values of the order of 280,000 ft/yr. Unfortunately, type curves for allowing analysis of leaky conditions were not then available.)

It will be evident, too, that if early time-drawdown data is lacking, or if there is much scatter of the points on a semi-log plot, and particularly if leakage is not pronounced, misleading interpretations can easily be made.

An example of the type of semi-log graph of time-drawdown data from a leaky confined aquifer is given at Figure 8 for Bore No. 18474. This bore had been constructed by a private contractor for town water supply purposes at Gunnedah, N.S.W. and only a generalized driller's log is available. It records soil and clay to 16 ft and sandy gravel to 71 ft but it is unlikely that the latter is all water-bearing. Screens were installed at 37-48 ft and 57-67 ft and the static water level was 22 ft when tested. The pumping rate was 15,100 gph.

When the time-drawdown data at Figure 8 is replotted on log paper of the same scale as that used for the leaky artesian type curves provided by Walton (1962), a good match is given with the type curve having an r/B value of 2.0, as shown at Figure 9. By taking match points at the intersection of major W(u, r/B) and 1/u axes, values for s and t are obtained, e.g. for 0.1 and 1.0, respectively, the corresponding values for s and t are 6.5 ft and 3.7 minutes, respectively. The coefficient of transmissibility is given by substitution in the equation

$$s = \left(\frac{114.6Q}{T}\right) W \ (u,\ r/B) \text{ i.e. equation 12 (Section 1.3.2.3)}$$

so that $$T = \frac{114.6 \times 15{,}100}{6.5 \times 60} \times 0.1$$

$$= 444 \text{ gpd/ft.}$$

(By way of contrast, if the semi-log graph at Figure 8 were accepted, and the modified non-equilibrium formula $T = 264Q/\Delta S$ applied, taking ΔS as 0.32 ft/log cycle, (Fig. 8), then the value obtained for T is 208,000 gpd/ft. !!)

In the above type curve analysis, if data were available from an observation bore, the coefficient of storage could be determined from equation 13 (Section 1.3.2.3). Furthermore, if the thickness of the confining foundation were known, the coefficient of permeability of the confining bed would be given from equation 14 (Section 1.3.2.3)

It is apparent from the above test data that the system was approaching a steady state because of the effect of recharge from leakage. In fact, in view of the relatively low transmissibility of the aquifer, leakage is no doubt providing the major proportion of the discharge. From the town water supply viewpoint, the critical aspect is just how much water is stored in the formation supplying the leakage, and there is insufficient data to assess this.

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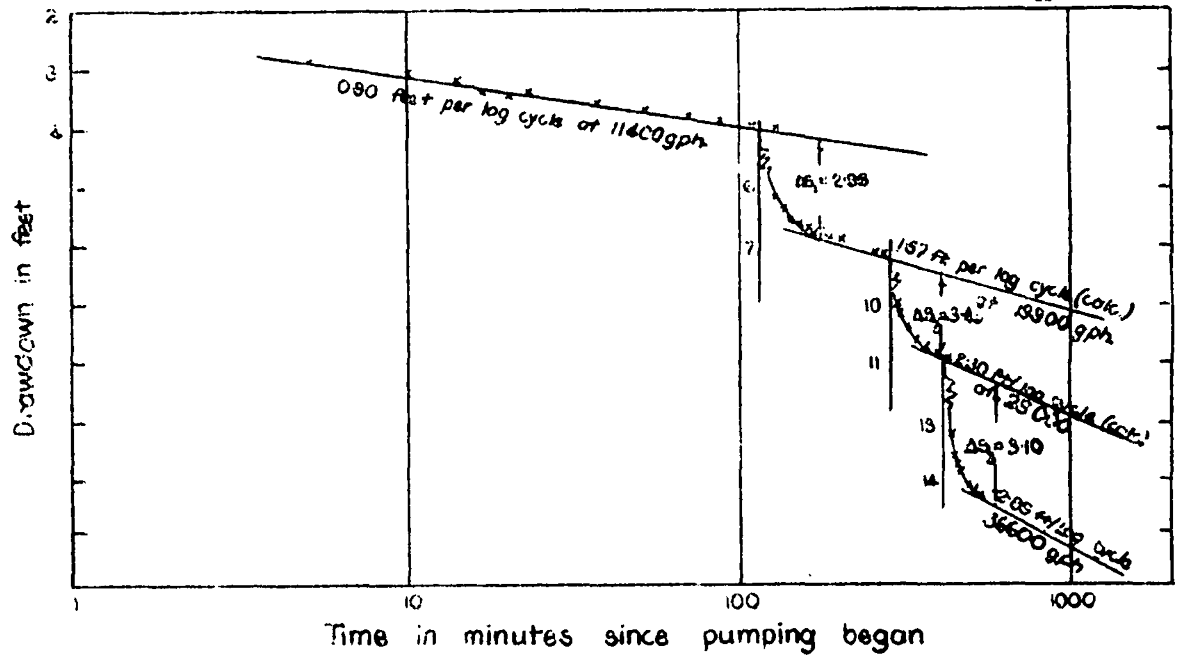
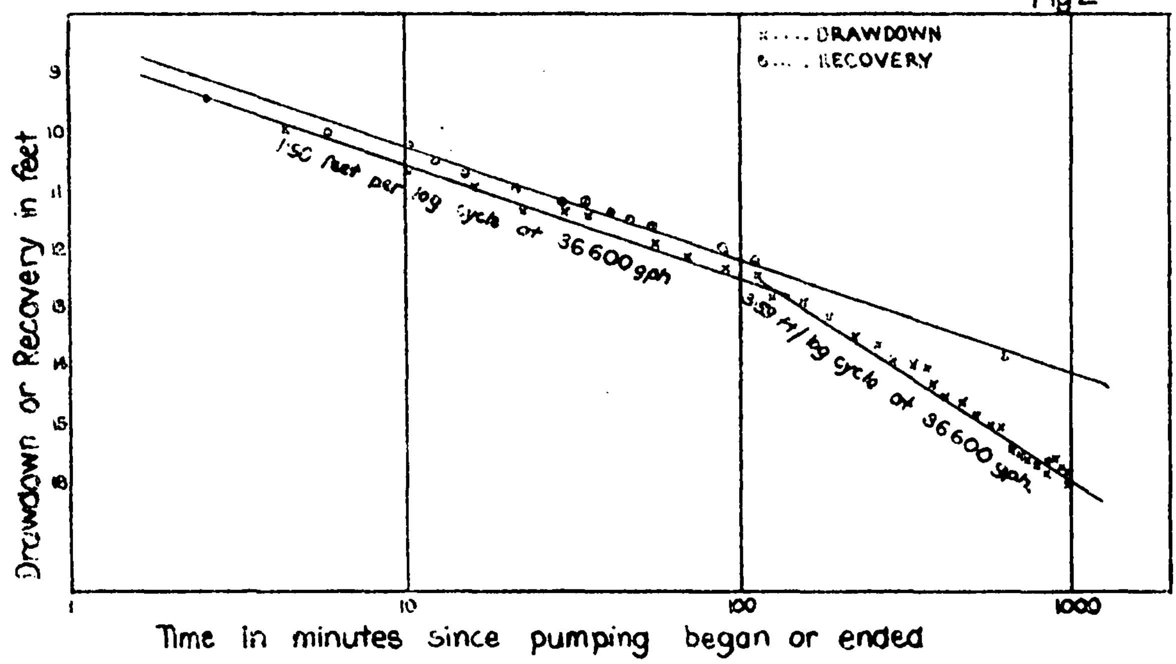
Fig. 1



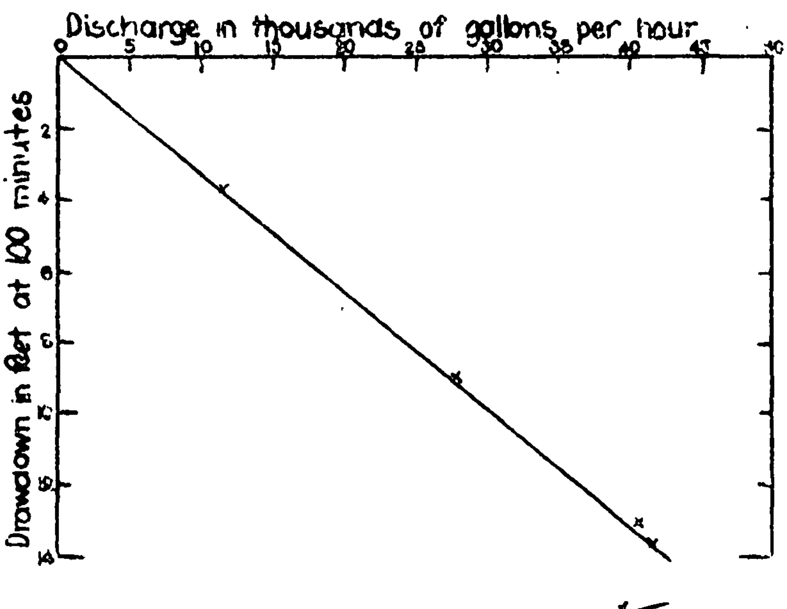
MULTIPLE-STAGE TEST BORE No. 14976

Fig 2



PUMPING TEST BORE No. 14976

Fig. 3a



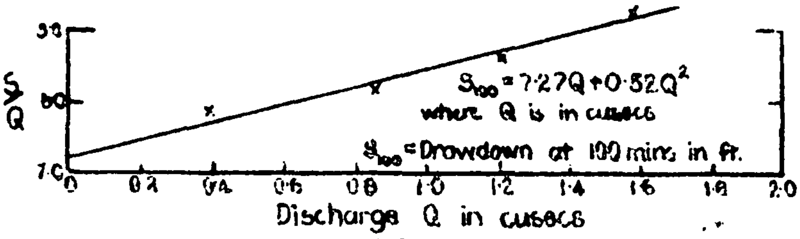
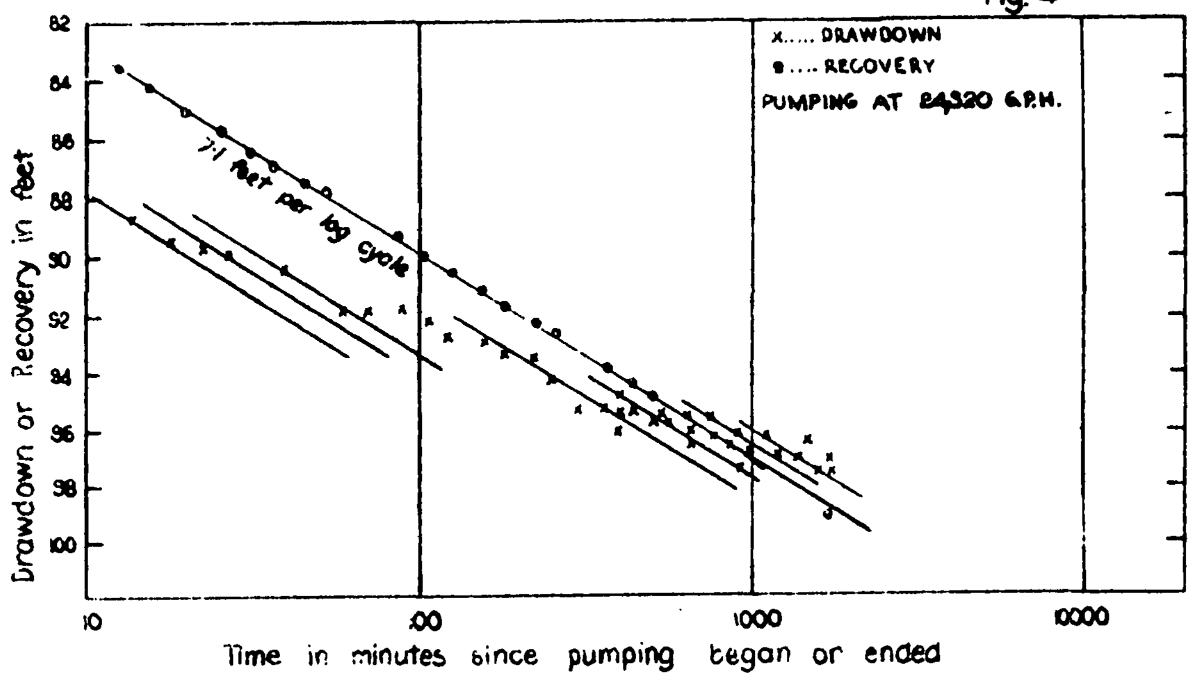


Fig. 3b

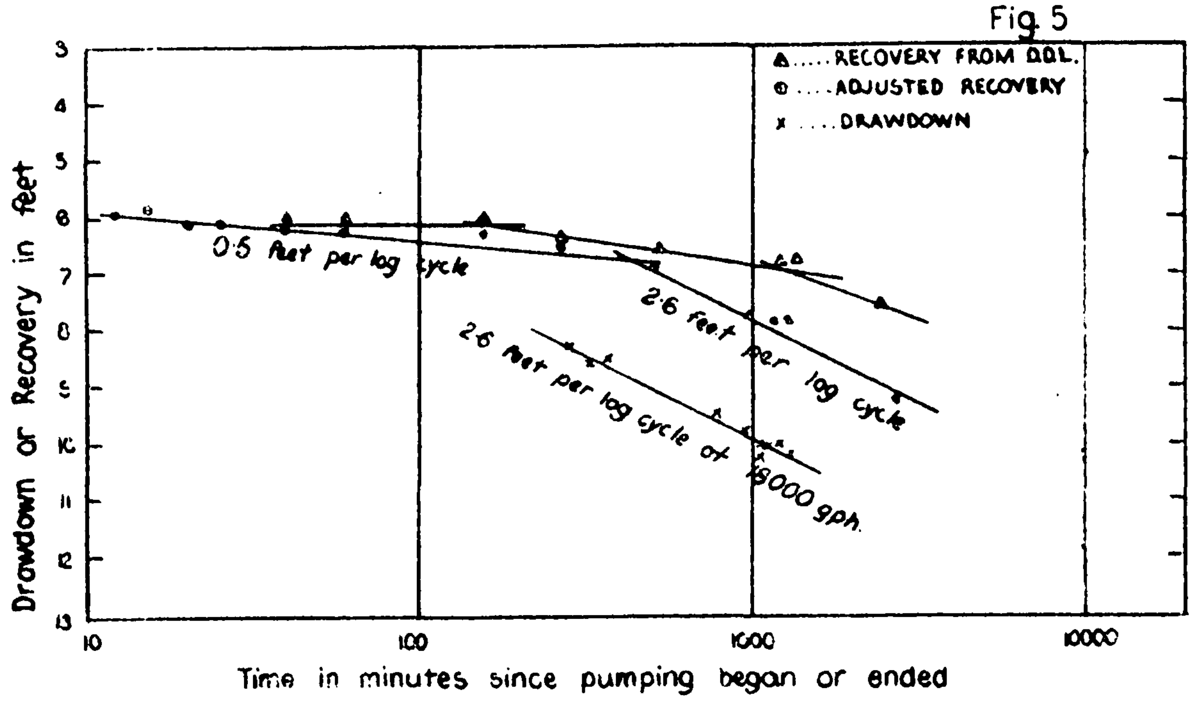
BORE No. 14976

Fig. 4



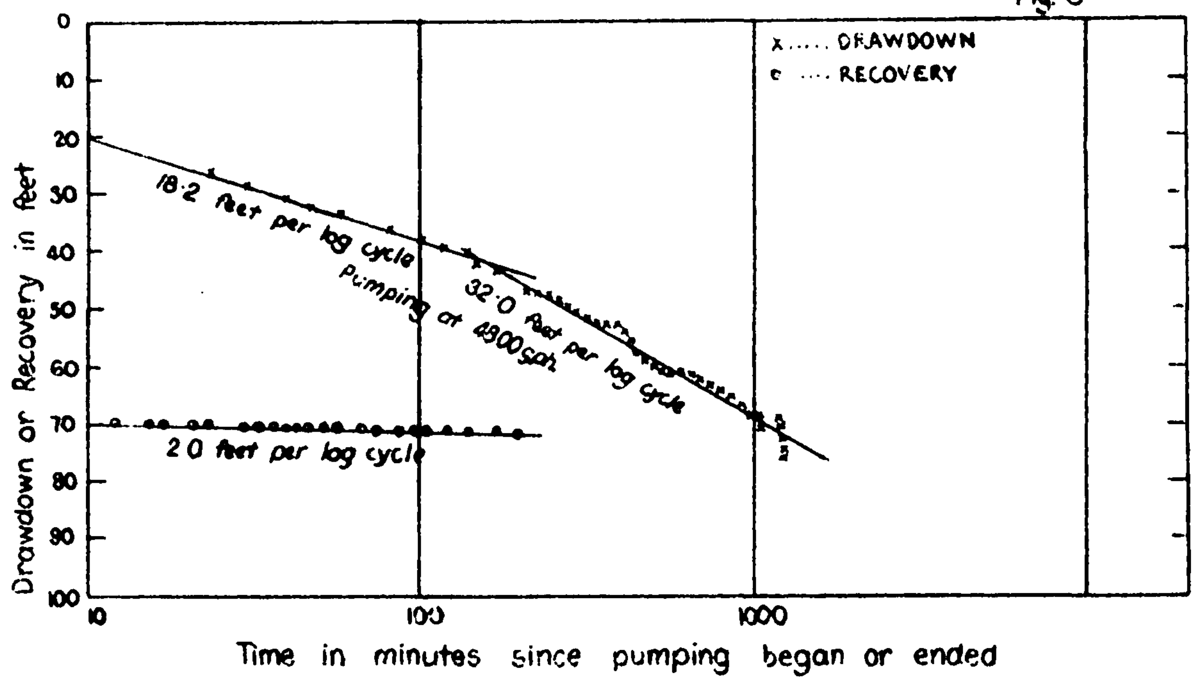
PUMPING TEST DATA BORE No. 21041

Fig. 5



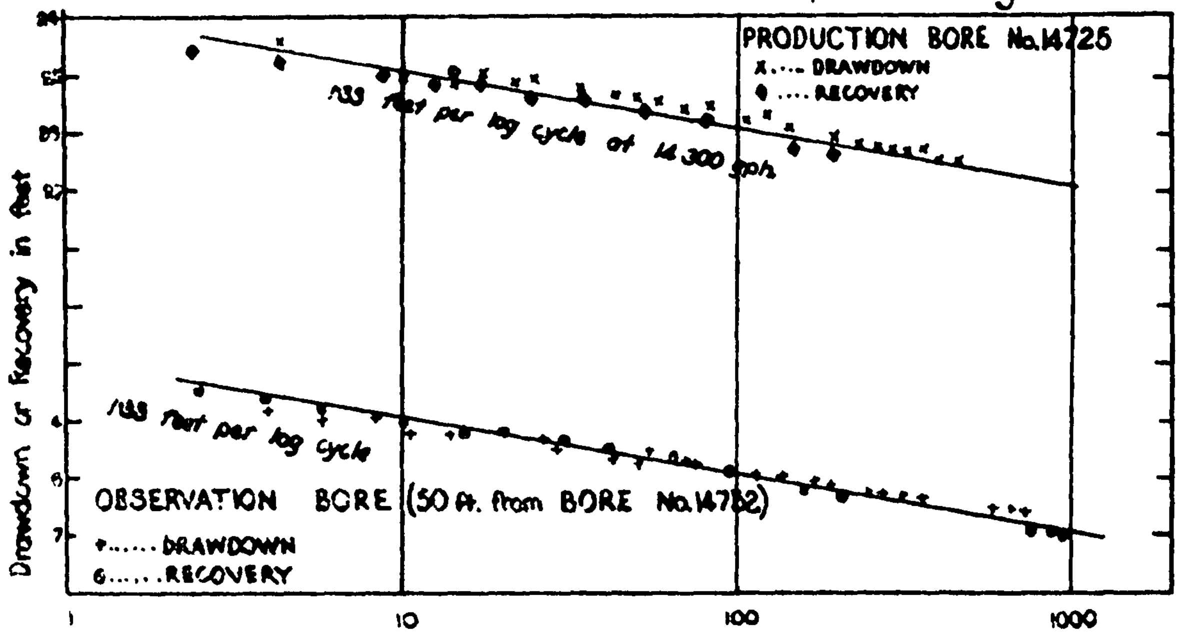
PUMPING TEST DATA BORE No. 17152

Fig. 6



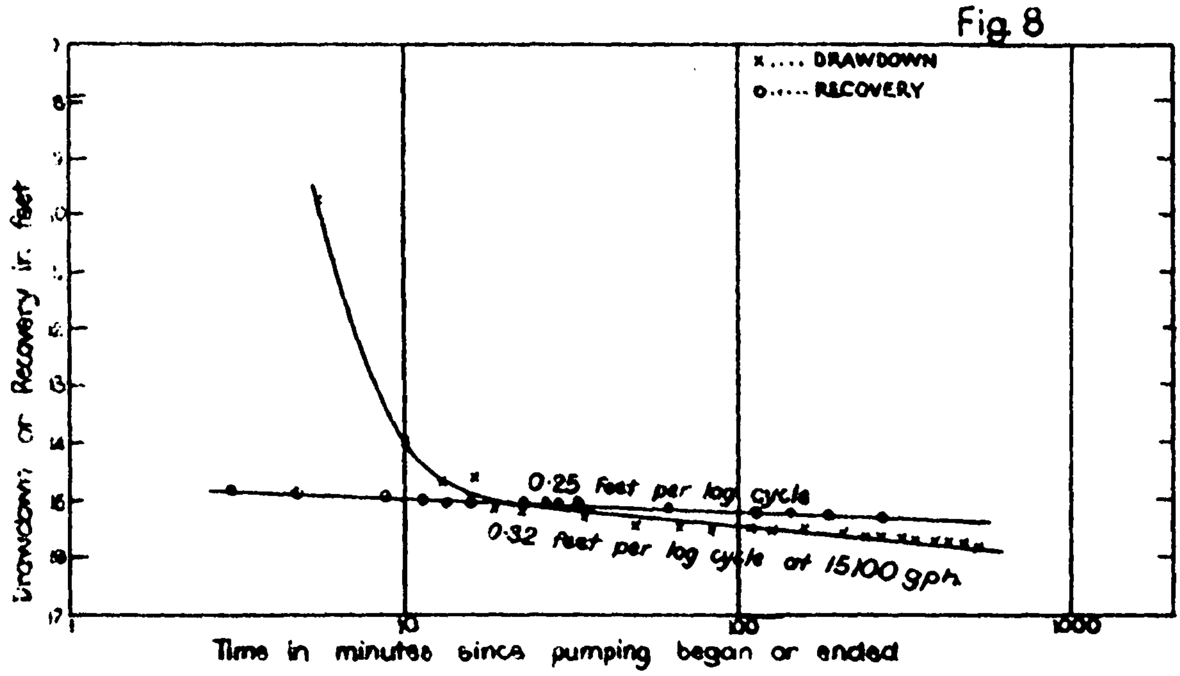
PUMPING TEST DATA BORE No. 21047

Fig. 7



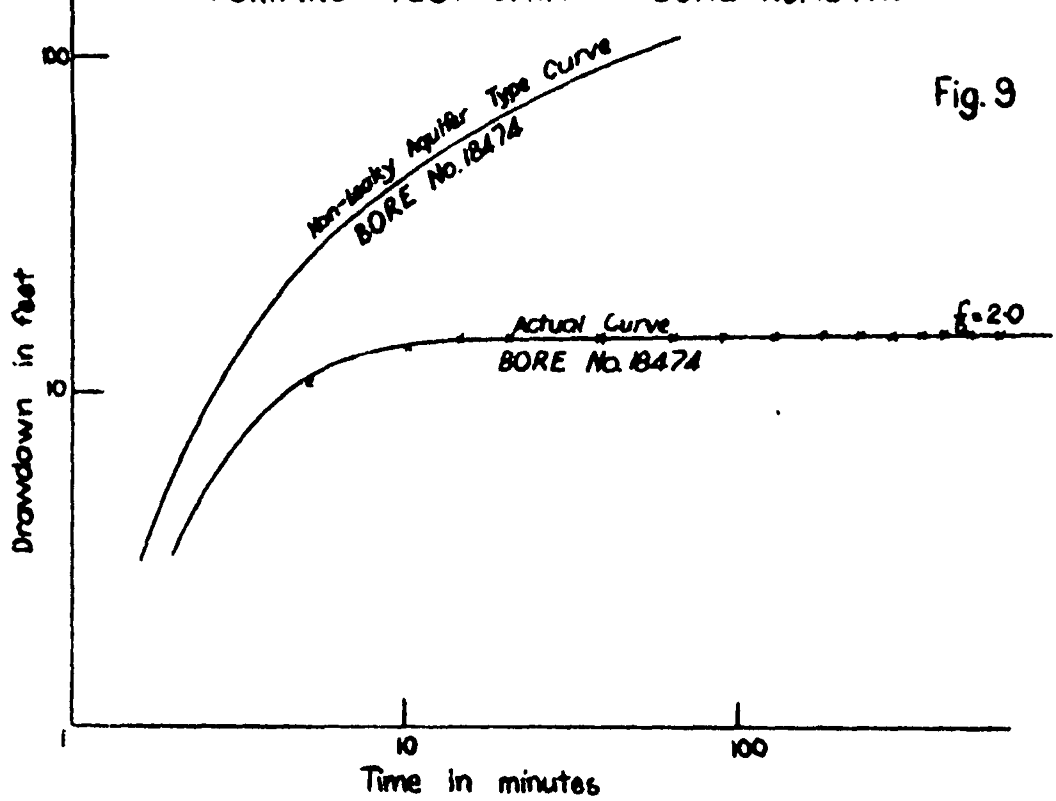
PUMPING TEST DATA BORE No. 14752

Fig. 8



PUMPING TEST DATA BORE No. 18474

Fig. 9



LEAKY AQUIFER CONDITIONS – TYPE CURVE METHOD

PLATE 2.

WAVE HILL
NORTHERN TERRITORY

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