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Hydraulic behaviour of an unconfined aquifer

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Research Project No. 75/85

HYDRAULIC BEHAVIOUR OF AN UNCONFINED AQUIFER

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1. INTRODUCTION

The analysis of pumping test data for unconfined aquifers, particularly at early times, is complicated by the existence of vertical flow components and the delayed drainage of the unsaturated material above the falling water table. Semi-empirical methods are available for including these conditions separately in any analysis but the results are often unsatisfactory. Although comprehensive numerical studies are being initiated by some research workers in this field the current position is still a developing one requiring detailed experimental investigation involving the measurement of the hydraulic parameters in the cone of depression.

The measurement of the hydraulic parameters for flow in unsaturated porous materials at depth is a difficult task and has been rarely documented. However, previous studies by the authors [Webb and Watson (1977a, 1977b)] have provided the instrumentation capability for such measurements. An existing bore owned by The University of New South Wales and situated in a reasonably homogeneous unconfined aquifer was also available for research purposes. With these facilities to hand the Australian Water Resources Council approved AWRC Research Project 75/85, "Hydraulic Behaviour of an Unconfined Aquifer" with the objective of studying the hydraulic behaviour of a typical unconfined aquifer by instrumenting the cone of depression formed during a pumping test with a rapid-response pressure measuring system.

The following report gives details of this project covering such aspects as a description of the Botany Basin Aquifer, the instrumentation of the field site and the field investigations and results. In addition, an annotated bibliography is included on water movement in unconfined aquifers. Although the instrumentation and experimental phases of the project were fully satisfactory the original objectives as relating to a *homogeneous* aquifer were not achieved due to the presence in the sand profile of silt lenses. These caused the aquifer to behave in a semi-confined manner and thus minimized the flow contribution due to such effects as the delayed yield from storage. However, the study remains significant in its own right in that a detailed analysis, based on reliable experimental data, has been made of the hydraulic behaviour of an unconfined, albeit non-homogeneous, aquifer.

2. THE BOTANY BASIN AQUIFER

2.1 Introduction

The field site for this study was located within the region underlain by the Botany Basin aquifer. This is largely an unconfined aquifer with some localised areas of partial confinement due to the presence of lenticular clay beds. It has been studied in some detail by several investigators, and since the region and its geology are relevant to this study, the important details will be briefly summarised in this chapter.

The area underlain by the Botany Basin aquifer is shown in Figure 2.1, together with the location of the field site where this study was carried out. Recharge to the aquifer takes place in a zone stretching from Botany Bay through Kingsford, terminating on the northern boundary of the aquifer. The main pumpage from the aquifer (currently 45 000 m^3d^{-1}) is in the Botany area just north of Botany Bay. Most of this water is used for industrial purposes.

Probably the major reference on the geology of the Botany Basin is Griffin (1963), and the findings in his report are summarised in the following section. Environmental factors affecting the Botany Basin aquifer are summarised in Section 2.3.

2.2 Regional Geology of the Botany Basin

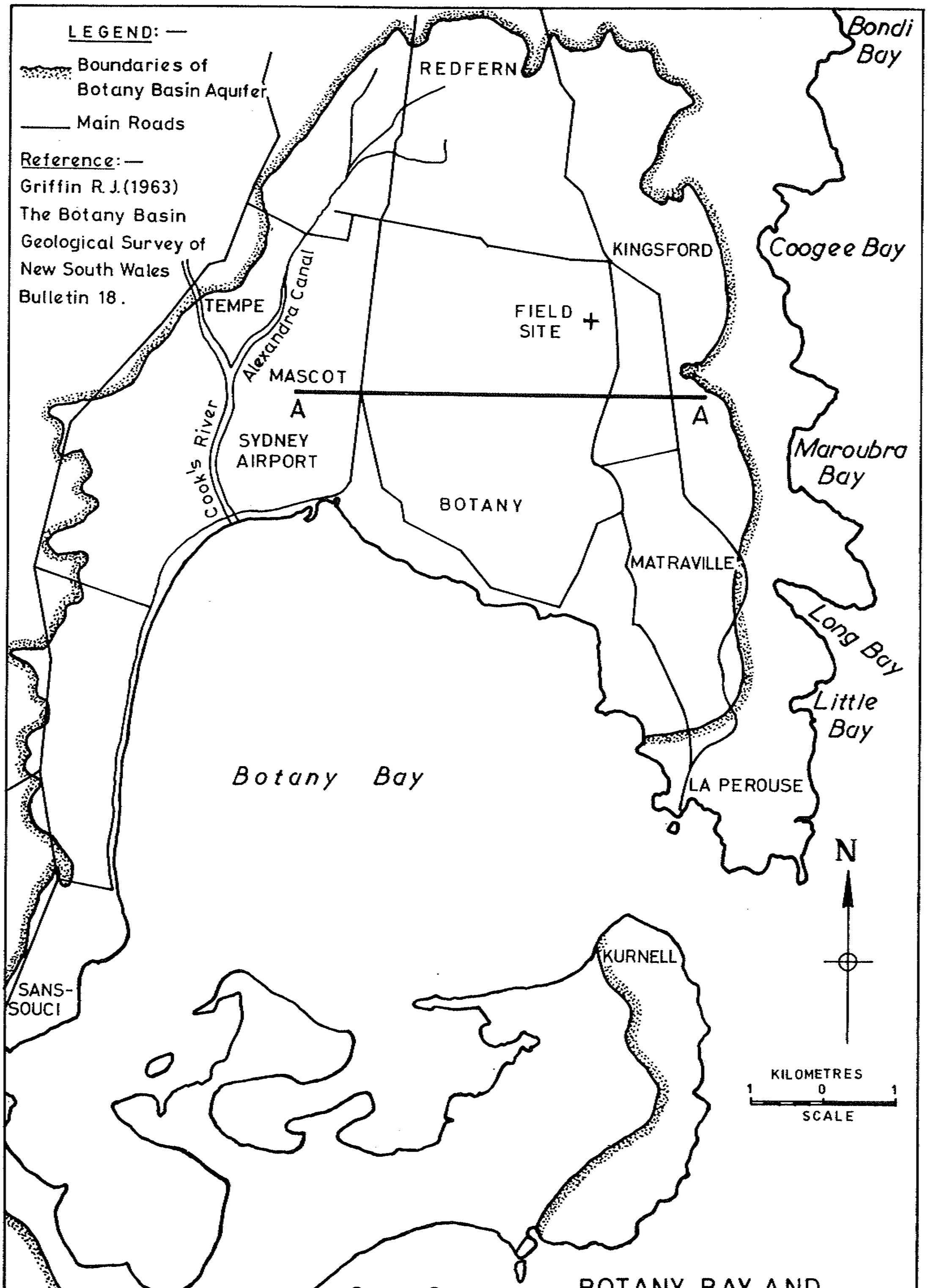
Griffin (1963) describes the geology and geohydrology of the Botany Basin region. The extent of the aquifer sands and alluvium is delineated in Figure 2.1. The zone near Sydney Airport and extending along the western boundary of the aquifer consists of Quaternary alluvial material. In the Kurnell zone and throughout the eastern and northern parts of the aquifer the deposits consist of Quaternary aeolian dune sands. There are also Quaternary tidal swamp areas in the Kurnell zone fringing Botany Bay.

The Botany Basin aquifer is encompassed by Botany Bay in the south, Triassic Hawkesbury Sandstone in the east, Triassic Hawkesbury Sandstone in the west and north-east and by Triassic Wianamatta Group Ashfield Shale in the north-west. A map in Griffin (1963) delineates these areas. The fresh water aquifer also extends under Botany Bay at depth with an overlying zone of saline water.

The Botany Basin aquifer lies in a small tectonic depression in the Hawkesbury Sandstone. Sediment deposits in the Basin range in depth from 2 m to approximately 80 m under the Georges River. The bedrock depressions correspond approximately to the present surface topography depressions in the present Georges River and Cooks River—O'Sheas Creek system.

The Quaternary deposits in the Botany Basin are composed of fine-grained quartzose sands with isolated lenses of peat and clay and some cemented ferruginous sand known locally as "Waterloo Rock". Figure 2.2 shows a cross-section (A-A of Figure 2.1) reproduced from Griffin (1963); the Pagewood location on this section is close to the field site used in this study. Bore logs taken during the original installation of the pumping bore and piezometers at the field site are given in Figure 2.3. It should be noted that there are layers of clay and peat present in the profile which occur at similar elevations in all the bore holes, indicating that these layers could be continuous over the zone affected by pumping. In fact it was found that there was evidence of this layering causing partial confinement of the aquifer in the vicinity of the pumping bore.

It is clear that the aquifer is inhomogeneous and that layers of peat and clay can



BOTANY BAY AND GEORGES RIVER BASIN

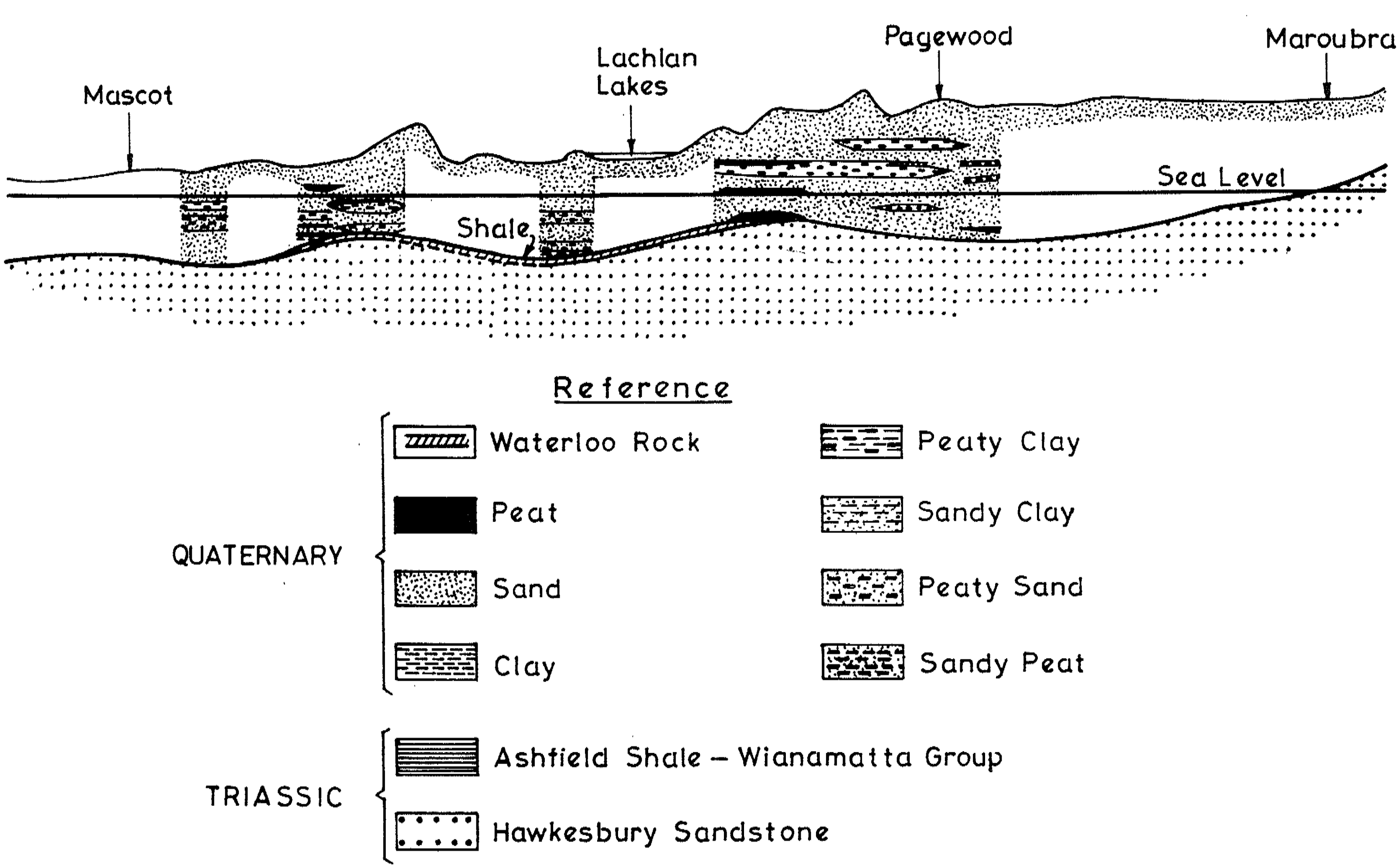


FIGURE 2.2: CROSS-SECTION A-A THROUGH THE BOTANY BASIN AQUIFER (see Figure 2.1)

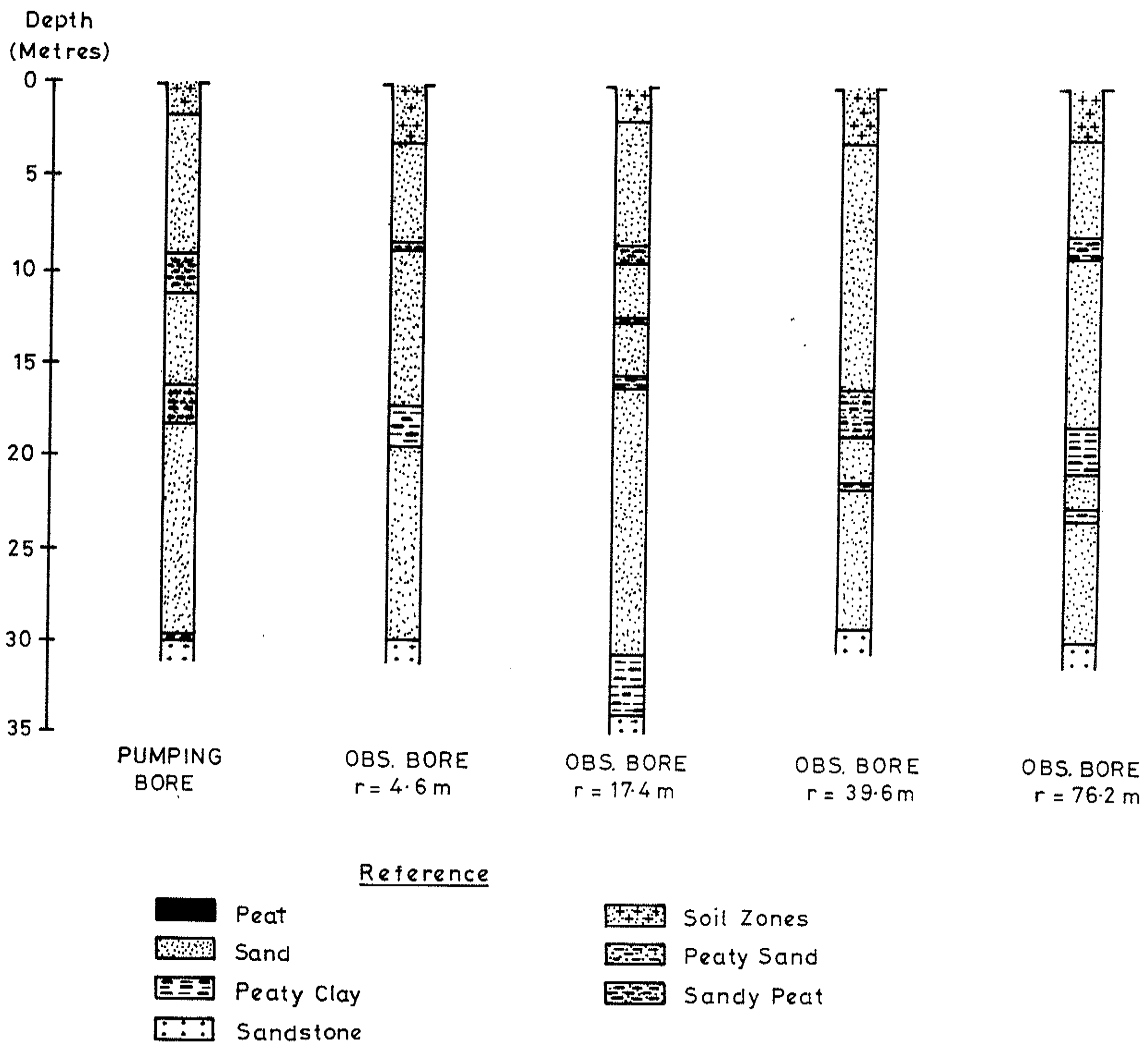


FIGURE 2.3: BORE LOG PROFILES TAKEN AT THE FIELD SITE

occur over significant areas. It is believed that the peaty layers resulted from swampy conditions and that the "Waterloo Rock" layers were caused by water level variations during swampy conditions. Griffin (1963), Philip (1973), Hawke (1973), and the Background Report of the Botany Basin and Georges River Environmental Study (1973) discuss the history and hydrogeology of the Botany Basin in greater detail.

2.3 Environmental Factors Affecting the Botany Basin Aquifer

There are two main factors threatening the environmental balance of the Botany Basin aquifer region. The first of these is pollution from industrial wastes and the consequent degradation of water quality, and the second is the possibility of an overdraft situation developing in the aquifer causing salt water intrusion. Pollution of the aquifer beds has been caused mainly by industrial wastes in the Botany-Banksmeadow area and possibly also by salt water intrusion caused by heavy pumpage in the same area. Information on the long term "safe yield" of the aquifer is still not definitive although the present pumpage is probably approaching this value. However, in some heavily pumped areas there is a danger of localised overdraft situations developing.

Several investigators have examined the pollution and general degradation of water quality in the Botany Basin sand beds. Griffin (1963) produced a water quality map for the aquifer showing heavy pollution in the Botany-Banksmeadow area and to a lesser extent on the western side of Botany Road. He concluded that the pollution was mainly due to dumping of industrial and other wastes for filling and reclamation works in the Basin. Pollutants included debris, wastes, detergents, weed killers and insecticides.

Hawke (1973) summarised the present knowledge of the distribution of pollutants in the Botany area. He drew on work by Wallis (1967) and Smart (1968) of the Geological Survey of New South Wales. Wallis reported that the intensity of pollution had increased by 70% from the time Griffin's data were collected in 1961. He found that the bores previously giving very high readings were considerably less affected but the pollutants had spread over a much wider area, extending up into the centre of Matraville. Smart (1968) showed that the pollutants had spread in a north-westerly direction from their previous area of concentration. Hawke (1973) concluded, as did Griffin (1963), that more geohydrological studies were needed including more water level information and the location of the salt water/fresh water interface. Only by examining such data could the "safe yield" of the Botany Basin aquifer be determined.

The work of Griffin (1963), Smart (1968) and Hawke (1973) is summarised in the Botany Basin and Georges River Basin-Environmental Study Background Report (1973). A series of four groundwater quality contours of the Botany-Matraville area are presented for the years 1961, 1966, 1968 and 1973. These clearly show the progressive extension of the polluted area. By 1973 the pollutants were not far from the field site shown in Figure 2.1.

It is believed that the extension of the polluted area has been caused by heavy pumpage drawing the pollutants in a north-westerly direction away from their source area. Pumpage in the heavily polluted area itself has been reduced in recent years as industry is now obtaining its water from bores located away from the area. It is again concluded in the Environmental Study that the prime cause of the pollution is industrial wastes and not salt water intrusion. However, there is a danger of this occurring because of the apparent slope of the piezometric surface away from the Botany area towards the centre of the Basin. If this situation continues, irreversible pollution by salt water intrusion could occur. The Geological Survey of New South Wales has detected both organic and inorganic pollutants in the aquifer. Inorganic pollutants consist mainly of chloride and sulphate ions, whilst organic pollutants include phthalate plasticiser, aromatic and aliphatic compounds and hydrocarbon residues.

The Environmental Study concludes that a programme of basin management is required, a similar conclusion to that reached by Griffin in 1963 and still not implemented. To devise a suitable management programme for the Botany Basin aquifer the Environmental Study recommends the collection of basic hydrogeological data. These include:—

1. A survey of existing bores in the Basin
2. Establishment of a network of observation bores to monitor water levels and quality
3. Estimation of aquifer coefficients throughout the Basin
4. Estimation of recharge to the aquifer
5. Estimation of runoff from paved areas
6. Estimation of pumpage from the aquifer.

It is recommended that the estimation of Basin "safe yield" be given a high priority.

A factor which has not been given much consideration in these studies, and which appears to be very important, concerns recharge to the aquifer from the main recharge areas in the Lachlan Lakes and Centennial Park; this recharge in itself could be a source of pollution. A study by Cordery (1976a, 1976b) has shown that runoff within the Basin contains a similar level of pollutants as secondary sewage effluent. Two of the small catchments he studied lie within the Botany Basin aquifer. Since it appears that surface runoff such as this is the prime source of recharge to the aquifer, consideration should be given to treatment of the water before it begins to cause a serious degradation in the water quality of the aquifer.

The delineation of the main recharge zones should therefore have a high priority both for the determination of the "safe yield", and to see whether recharge from these zones is in itself a source of additional pollution.

3. INSTRUMENTATION OF FIELD SITE

3.1 Introduction

The field site (see Figure 2.1) was located at the David Phillips Sports Field at Daceyville on land owned by The University of New South Wales. A 0.20 m bore had already been installed at the field site together with ten piezometers. The installation had been completed using a grant provided by the Water Resources Commission of New South Wales. The bore is equipped with a submersible turbine pump, with the suction inlet positioned at an elevation of 19.8 m below the ground surface. The pump is capable of pumping 2 400 m^3d^{-1} when the inlet valve is fully open. From the ground surface to a depth of 20.4 m the bore consists of 0.20 m mild steel casing, which is connected to a 0.19 m diameter stainless steel screen (1.0 mm opening) from 20.4 m to 29.6 m. The final 1.5 m of casing reaching to the basement of the formation forms a sand trap. The grading of the natural aquifer sand is such that 40% is retained on a 0.3 mm screen and 90% is retained on a 0.2 mm screen. Sand pack material (40% is retained on a 1.6 mm screen and 90% is retained on a 1.1 mm screen) was distributed evenly around the bore for a thickness of 100 mm.

A considerable amount of data was already available from previous investigations at the site. Pump tests were carried out at this site soon after the bore was installed and the results of these tests were presented in an internal report by Swan (1971). Although there were inadequate data at short times to determine the aquifer characteristics accurately, these tests, in retrospect, could have been interpreted as indicating that the aquifer in this area behaved in a leaky or partially confined manner. The storage coefficient varied from 0.4×10^{-3} to 2.2×10^{-3} in different observation bores and the transmissivity was found to be of the order of 230 m^2d^{-1}.

Since one of the main objectives of this study was to study the behaviour of the aquifer in the cone of depression it was necessary to install additional pressure sensors in the upper zone of the aquifer near the pumping bore. All the available piezometers were located deep in the aquifer and at distances ranging from 4.6 m to 76.2 m from the bore. Additional pressure sensors were installed progressively as more became known about the behaviour of the aquifer. When the installation was completed, comprehensive pump tests were carried out and these are described in later chapters.

3.2 Design of Pressure Sensors

Two types of pressure sensor were used to measure the hydraulic conditions at the David Phillips Field site. The first sensor used was a tensiometer-pressure transducer unit designed for the Burdekin Artificial Groundwater Recharge Study and described in Webb and Watson (1977a). This was an adaptation of a field tensiometer unit described by Watson (1967). It can be used with both saturated and unsaturated porous materials. The second type of pressure sensor was used for the automatic monitoring of the water levels in the piezometer holes; these constituted the bulk of the pressure measuring points.

3.2.1 Tensiometer-pressure transducer unit

Three tensiometer-pressure transducer units were installed at the field site. The technique used for installing them is described in Section 3.3.1. Initially it was assumed that most of the pressure measuring points would require this type of unit as the water table was expected to fall rapidly in the vicinity of the pumping bore, causing dewatering and negative soil water pressures. Since this did not eventuate (indeed in most of the profile the soil water pressure remained positive) pressure transducers in a specially constructed probe housing were used to measure the fall of the water level in the cased piezometer holes.

A cross-section of a typical tensiometer-pressure transducer unit is shown in Figure 3.1. The lower part of the sensor containing the ceramic was fabricated from stainless steel bar. High-flow one bar ceramic was cemented with epoxy resin in a recess on the lower face of the unit. The ceramic provided the necessary continuity for unsaturated conditions between the water inside the unit in contact with the pressure transducer diaphragm and the soil water in the unsaturated porous material. The electrical disconnect, pressure transducer, lock washer, gasket and two transducer assembly parts were purchased from Statham Instruments Inc., Oxnard, California.

The Statham transducer used in the units was the PA856-25 absolute pressure transducer which is a thin film vacuum deposited strain gauge transducer. Deflection of the diaphragm changes the strain in four strain gauges mounted in a Wheatstone Bridge circuit inside the transducer. By supplying 10 V DC current across two arms of the bridge the response of the diaphragm to pressure changes can be measured by reading the output across the other two arms of the bridge with a digital voltmeter. The relationship between mV output and pressure is very close to linear over the range 0 to +173 kPa absolute for this transducer. Temperature compensation is provided over the range -54°C to +121°C.

If the DC power supply is stabilized and a 1 μV digital voltmeter is used to measure the mV output from the transducer, the resolution of the pressure reading is approximately 0.5 mm of water, and the accuracy in measuring the relative difference between two water levels is approximately ±3 mm.

The tensiometer-pressure transducer units consisted of the end housing described above, together with 25 mm internal diameter water pipe which was used to extend the assembly to the ground surface. The pipe was fabricated in 1.5 m and 3 m lengths with stainless steel connectors (incorporating O-ring seals) being used to join several sections together to give the required overall length. The five-core shielded cable attached to the pressure transducer (2 output leads, 2 input leads, one spare lead) was brought up inside the pipe and passed through an O-ring seal in the end cap. A small bag of silica gel was hung inside the top of the pipe to absorb the water vapour enclosed inside the pipe. Special care was taken to seal all joints against the entry of water because of the danger of damage to the pressure transducer.

The last length of pipe was cut to an appropriate length so that the end cap stood approximately 0.3 m above the soil surface. A length of 50 mm I.D. pipe was placed over the top of the end cap and pushed 0.2 m into the soil to protect the end cap and cable. The cable was taken underground to an instrument hut located nearby and connected to the data acquisition equipment used to monitor and record the output signal.

3.2.2 Transducer probe used in piezometer holes

The ten piezometers installed at the same time as the pumping bore, were fabricated from 32 mm I.D. galvanised water pipe with a 0.9 m screen at the bottom end. Eleven new piezometers were installed using the techniques described later in Section 3.3.2; these were constructed from class 12 polythene tubing of 32 mm O.D. cut to the appropriate length for each installation. To this was attached a 0.45 m length of copper tubing which had been perforated over the central 0.3 m, and covered with 100 mesh brass woven wire. The wire was

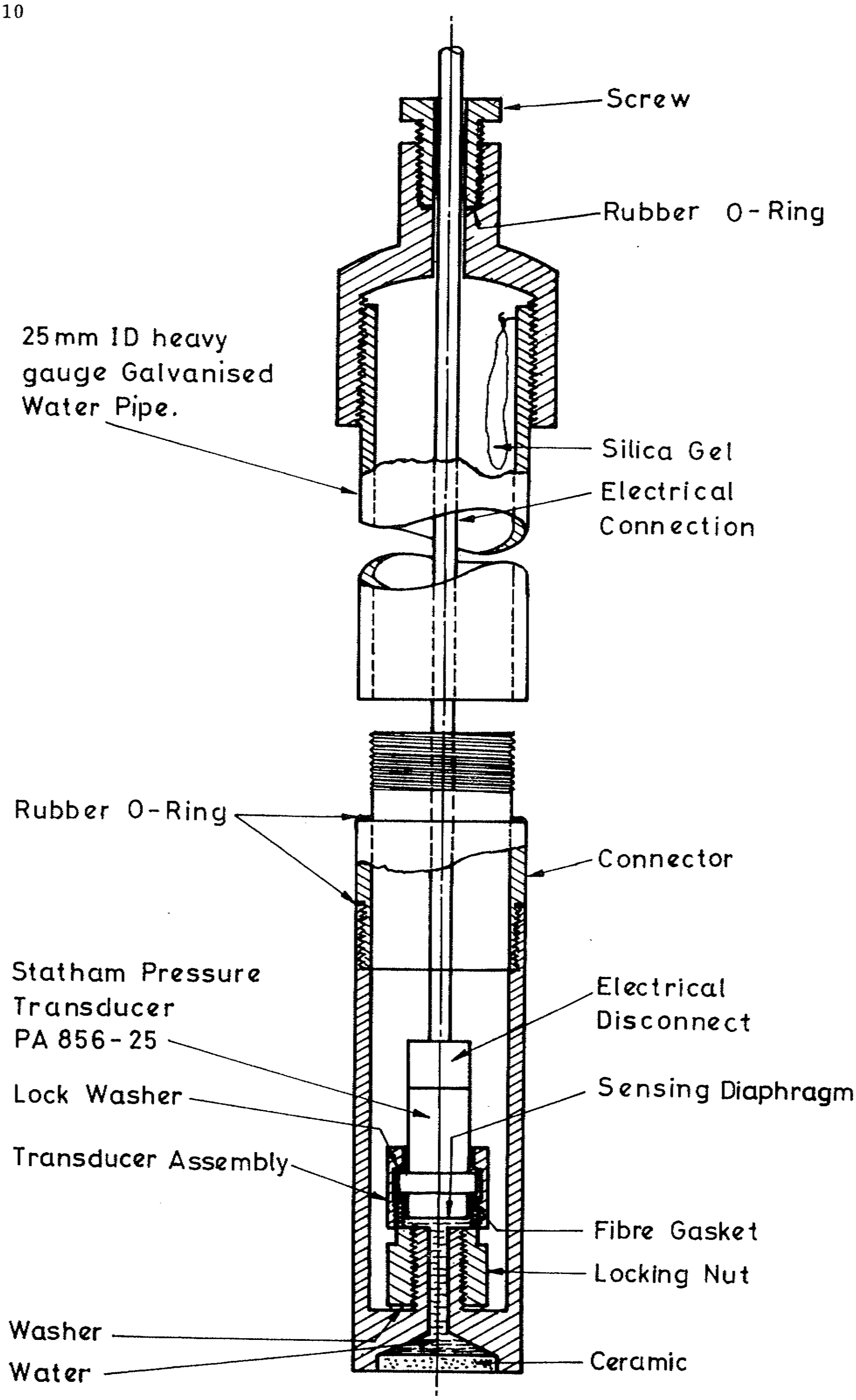


FIGURE 3.1: CROSS-SECTION OF TENSIOMETER-PRESSURE TRANSDUCER UNIT

soldered to the tubing and a plug was soldered on one end of the tubing. The outside diameter of the copper tubing (25 mm) was slightly larger than the inside diameter of the polythene pipe, so that by heating the polythene tubing it was possible to force the open end of the copper tubing into the polythene making a firm connection. These piezometers were found to be highly effective. Little or no sand could enter the tubing through the 100 mesh screen and reduce the efficiency of the piezometer.

Previously all the piezometer type installations were monitored manually during pump tests by lowering a water level detector down the piezometer pipe until water was encountered. Contact with water completed an electric circuit causing a globe to light at the ground surface. This technique required one observer and detector at each measuring point and someone to log the readings. With the installation of an additional eleven piezometers for this study it became prohibitive by this method to try and monitor more than a few of these points in any given pump test. In addition to this problem it was found that it was extremely difficult to obtain accurate readings by this technique in the critical early stages of a pump test, and during the first part of the recovery period, due to the rapid movement of the water surface in the piezometer holes.

Because of these problems it was decided to devise a simple probe unit containing a pressure transducer, which could be taken to the field and lowered down a piezometer hole for the duration of a pump test. Ten of these units were fabricated and a typical cross-section of one of them is given in Figure 3.2. The unit was made entirely of stainless steel. The prime consideration in designing these units was to protect the transducer terminals from the entry of water. Initially this proved to be a difficult problem but was eventually solved by the use of thread sealer. The probe was designed so that it could easily fit down the inside of the old piezometer pipes and also the new ones installed for this study. Statham PA856-25 pressure transducers were again used as the sensing unit in these water level probes.

The probes were separately calibrated for each pump test since they had to be removed from the field site on the completion of a test for security purposes. The calibration procedure consisted of the following sequence of events. Firstly the ten water level probes were suspended above the water level in the various piezometer holes. They were kept in this position until the data acquisition equipment indicated that the readings had stabilised. Several scans were then recorded of the outputs of these pressure transducers and also an additional transducer open to the atmosphere. The probes were then lowered below water level in each piezometer hole by a known amount to ensure that they were below the expected drawdown limit for each hole. This ranged from about 0.5 m to 5 m below the water level for different piezometer holes. After the readings had stabilised at their new level several more scans were taken. The difference between the second set of readings and the first gave the depth of water over each transducer. The pump test was then commenced.

A separate transducer was always used to monitor any atmospheric pressure changes that may have occurred during the test. Since all the pressure transducers were absolute pressure units allowance had to be made for atmospheric pressure changes to obtain valid water pressure readings. Initially the transducer for monitoring atmospheric pressure was kept in the instrument hut but it was found that small pressure changes caused by a fan, and opening and closing of the door etc., caused fluctuations in the readings. The atmospheric transducer was subsequently suspended above the water level in an unused piezometer hole, so that it

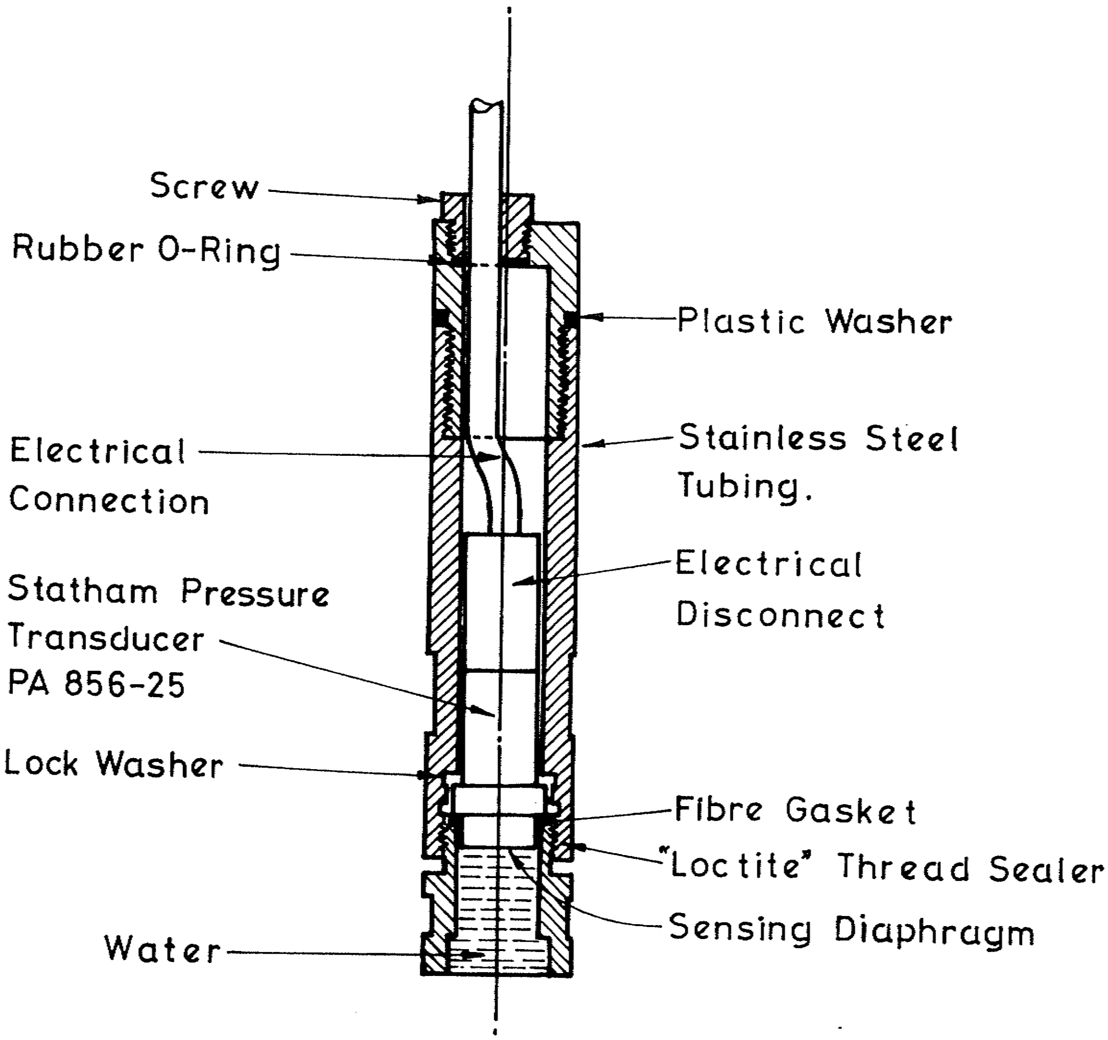


FIGURE 3.2: CROSS-SECTION OF TRANSDUCER PROBE UNIT

was in an environment similar to the other pressure transducers.

The probes proved to be accurate and reliable and greatly increased the amount of data available from each pump test, particularly at the important early times. Details of the data acquisition equipment and the collection and analysis of the data, are described in Section 3.4.

3.3 Installation of Pressure Sensors

The piezometer holes available at the beginning of this study are numbered 11, 12, 13, 18, 19, 20, 21, 22, 23 and 24 in Figure 3.3. They are all located well below the water table and, unfortunately, provided little information on the behaviour of the cone of depression in the vicinity of the pumping bore. The first objective of the field work therefore concerned the placing of piezometer holes in the upper part of the profile near the bore so that detailed information would be available on the movement of the water table.

3.3.1 Tensiometer installation

Three tensiometer-pressure transducer units (see Section 3.2.1 for details) were installed at the field site. These are numbered 1, 2 and 4 in Figure 3.3. Two were placed at a radius of 3 m from the bore and the other at 4.6 m. Tensiometers 1 and 4 were placed at approximately the same elevation.

The installation technique used in this study was similar to that used in the Burdekin Study but some changes had to be made to cope with the below water table conditions. The first step in the installation procedure was the drilling of a hole to the water table. A two-man post-hole digger was used for this purpose with a 40 mm diameter coal auger being used as the drilling tool. It was fabricated in 1.5 m lengths and additional lengths were added as drilling progressed.

Plate 3.1 shows the drilling equipment laid out in preparation for drilling a tensiometer hole. The auger bits and wrenches used in joining lengths of auger together are shown in the centre; the rigid PVC pipes used for casing the hole can be seen in the background. Plate 3.2 shows a view of the equipment being used at the Daceyville field site. The hole made by the auger was free-standing until the water table was encountered. At this point 40 mm PVC piping was installed down the hole to act as casing. This was fabricated in 1.5 m lengths with male and female sockets machined on either end. Joints were fastened using quick-drying cement.

The hole was continued beyond the water table using one of two techniques. The first technique involved drilling through the plastic casing with the auger, while preventing the casing from rotating using a pipe clamp. This device is illustrated in Plate 3.3. This approach had limitations for the auger tended to act as a "sand pump" with resultant jamming of the auger inside the casing. The slurry of sand brought up by the auger is clearly visible in Plate 3.3.

The second technique used a sampling tube on the end of a long rod fabricated in 3 m lengths. This sampling tube was moved up and down quickly at the same time as the casing was pushed downwards using the pipe holder. When the sampling tube became full it was removed and cleaned. This technique was reasonably effective but was slow and laborious over any significant depth.

Once the casing had been sunk to the required depth the tensiometer-pressure transducer

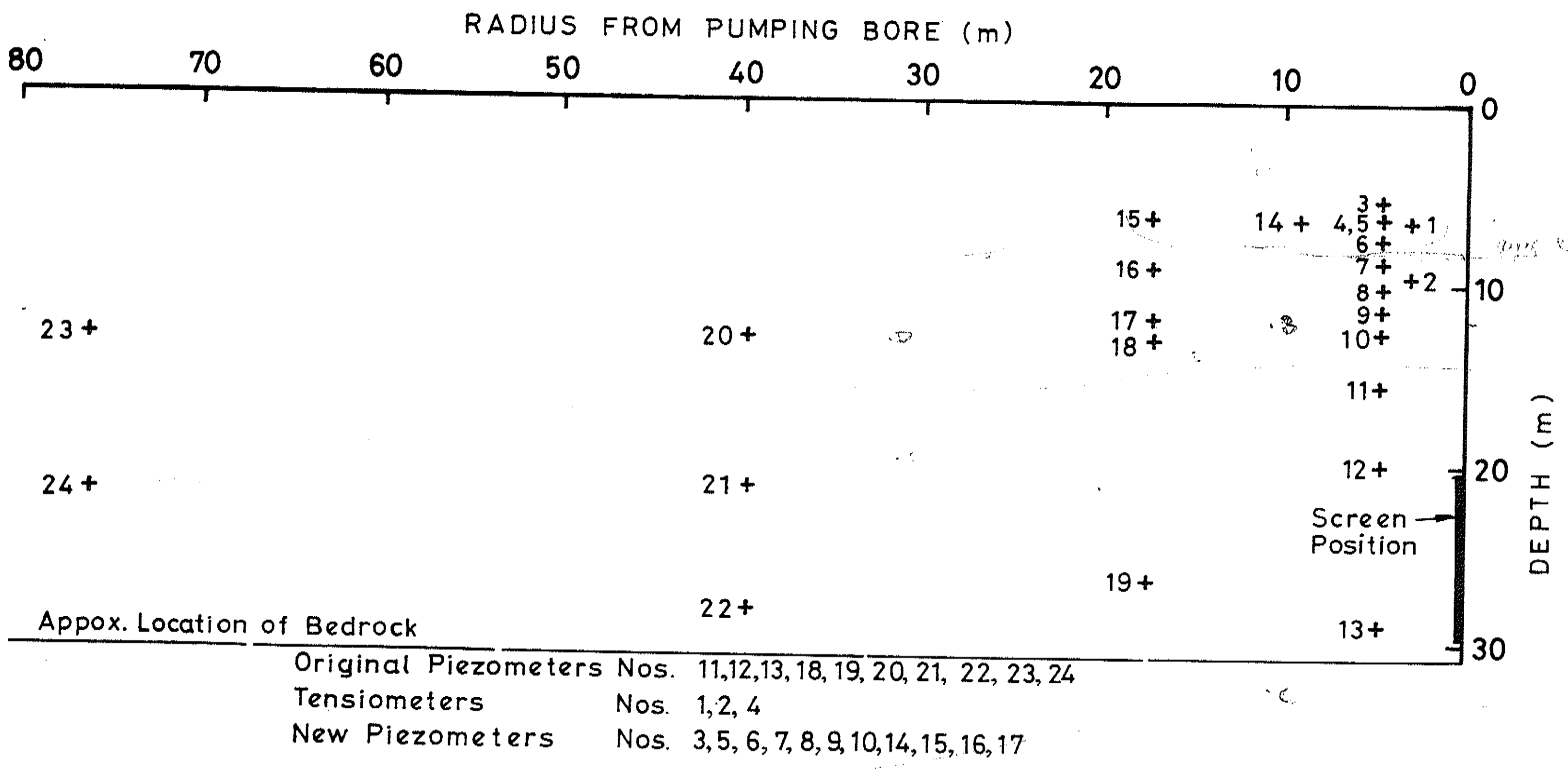


FIGURE 3.3: CROSS-SECTION OF PIEZOMETER INSTALLATIONS

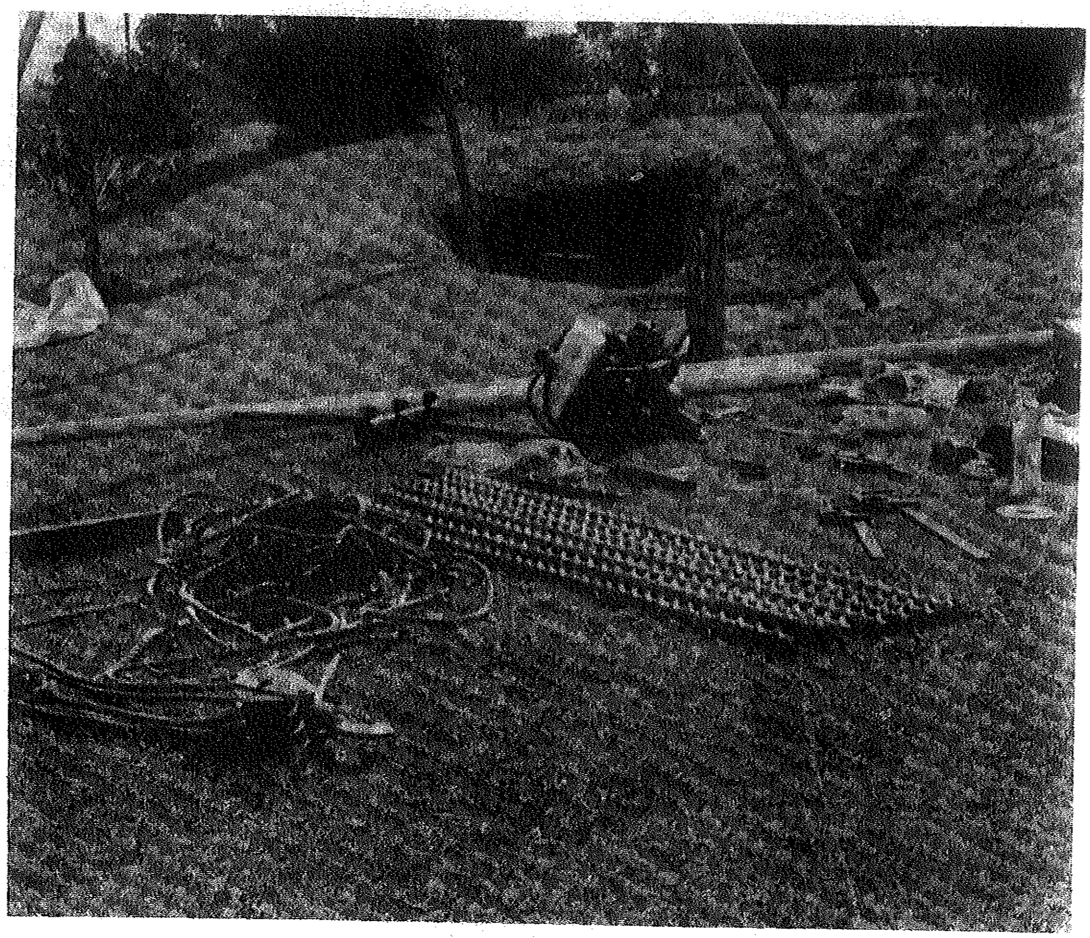


PLATE 3.1 DRILLING EQUIPMENT AT FIELD SITE

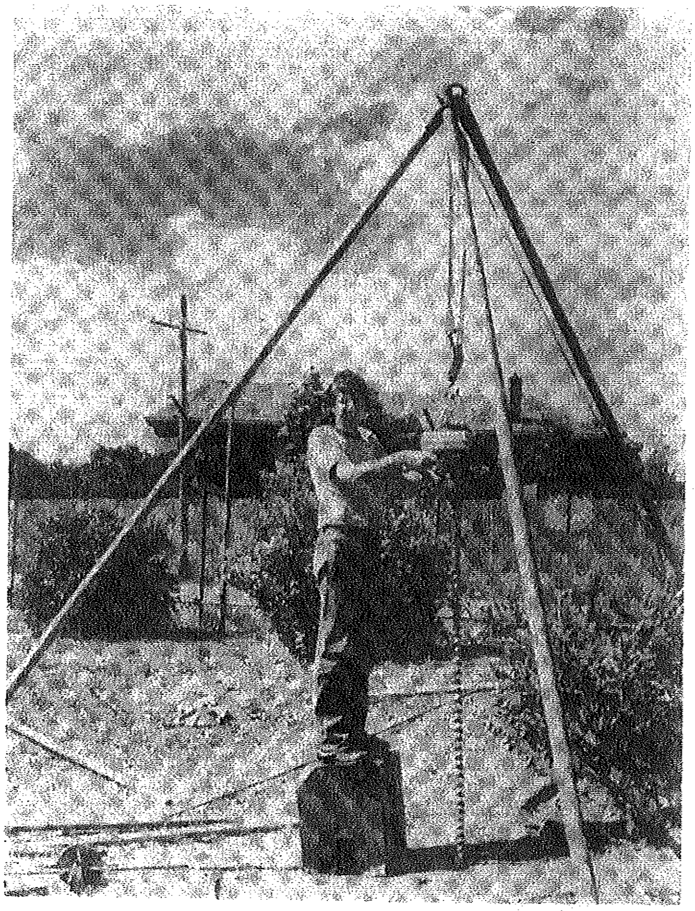


PLATE 3.2 DRILLING EQUIPMENT IN USE

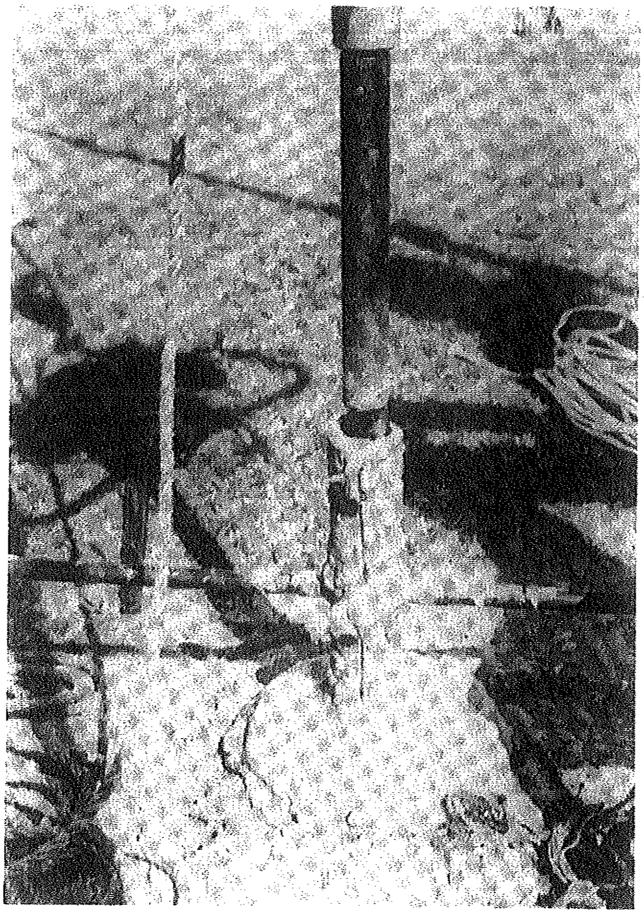


PLATE 3.3 DRILLING WITH THE AUGER BELOW THE WATER TABLE

unit and extension pipe were lowered through the casing. The casing was then withdrawn from the hole in stages. Care was taken to move the casing up and down to compact the sand around the tensiometer pipe as the casing was withdrawn.

3.3.2 Piezometer installation

A total of eleven new piezometers were installed at the field site. They are numbered 3, 5, 6, 7, 8, 9, 10, 14, 15, 16 and 17 in Figure 3.3.

Because of the difficulties encountered in installing the tensiometer-pressure transducer units to even a shallow depth, it was necessary to develop a faster method for installing the piezometer casing since some screens were to be positioned 15 m below ground level. The only practical method of installation appeared to be jetting, as the cost of installing individual piezometers using a commercial percussion drilling rig, would have proved prohibitive within the budgetary constraints.

Initially a hole was jetted down using the 40 mm rigid PVC tubing utilized as casing for installing the tensiometers. Water from a nearby garden tap supplied by the test pumping bore was used in the jetting process. The technique was found to be fairly effective but the low water pressure and limited volume of water available made penetration difficult at some stages. Beyond 10 m depth the rate of penetration was very fast, but between an elevation of about 6 m (the depth of augering) and 10 m penetration was more difficult, probably due to the presence of peaty and clayey layers. When difficulties were encountered the plastic tube was moved up and down until the layer was passed.

There were two problems with this approach. Firstly the joints in the plastic tubing (cemented male-female connections) were of limited strength and there was always a danger that the pipe could break at a joint leaving the balance of the tubing in the ground. The second problem related to the large amount of workshop time involved in machining the ends of each 1.5 m length. It was therefore decided to purchase EX drill casing which could be re-used for each hole, and would have strength and rigidity.

All subsequent piezometer holes were sunk with EX drill casing using the jetting technique. Since the supply of water from the garden tap was found to be inadequate, a 50 mm tapping was made on the delivery side of the pumping bore. To this was attached a length of 50 mm 3-ply water delivery hose. This jetting system in use at the field site is illustrated in Plate 3.4.

Under good conditions it was possible to jet four holes to an average depth of 10 m and install the piezometer casing in one day. The procedure consisted of drilling a hole to the water table using the auger, withdrawing the auger and installing about 5 m of EX casing. The jetting head was then attached with control valve closed and the pump switched on. Jetting was commenced by opening the control valve slowly and gently raising and lowering the casing while pushing the casing steadily into the soil. If penetration became difficult the water pressure was increased to about 550 kPa for a short time. This usually had the effect of clearing any obstruction, although in some instances it caused the sand to jam against the outside of the casing. If this happened the casing had to be raised and lowered a few times to clear the sand.

For most of the installations the technique worked very well but in a few cases penetration was difficult. It is believed that the problems were caused by either highly com-

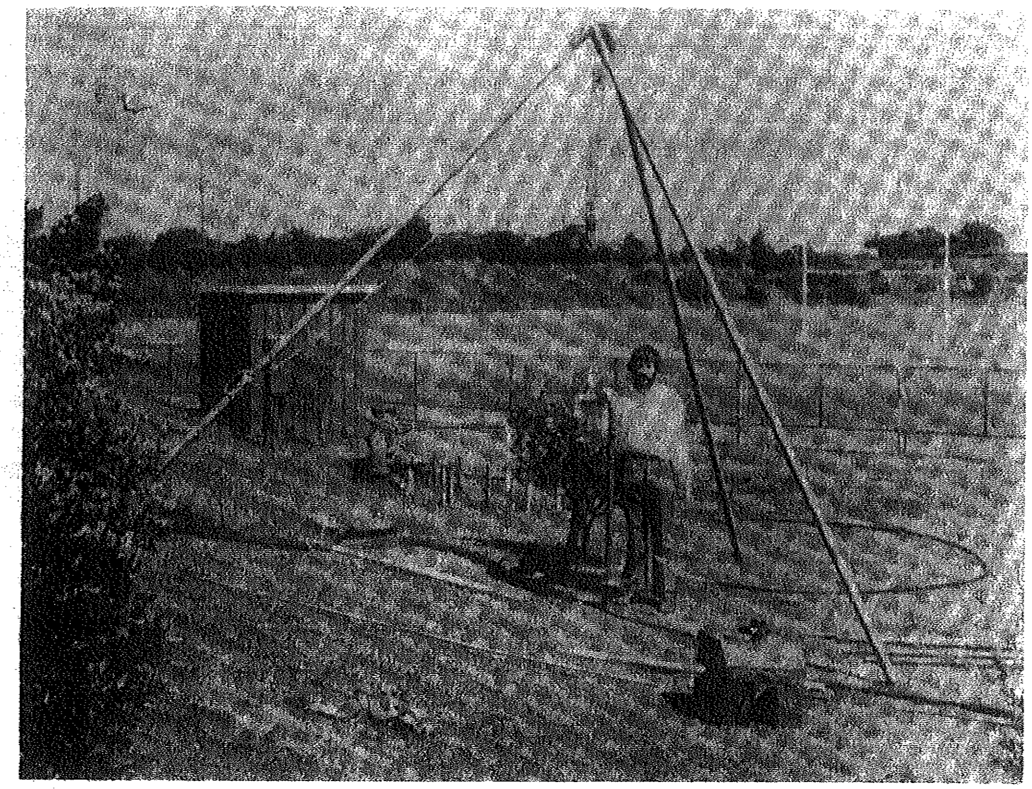


PLATE 3.4 JETTING SYSTEM

pacted layers of sand or peaty layers. The presence of layers in the profile was later confirmed by observations taken during pump tests.

The jetting technique is only suitable in sandy materials when the soil is not well compacted. If a jetting point is used however, penetration can be achieved in firmer soils. In this study it was not possible to use a jetting point as this would have made it impossible to install the piezometer through the casing after the hole had been completed. Some of the techniques used in the jetting operation were adapted from two publications of the Johnson Division of the Universal Oil Products Co. (1966, 1969).

Once the casing had been jetted to the nominated depth the installation of the piezometer was a relatively simple operation. The piezometer screen and tubing were gently pushed through the casing until the bottom was encountered. It was then held down while the casing was withdrawn and dismantled. Above the water table the casing was moved up and down to compact the sand around the piezometer pipe. A length of 40 mm I.D. galvanised water pipe was placed over the top of the polythene tubing to protect it and to provide a reference level. A screw cap on top of the pipe completed the installation.

3.4 Data Collection

Both the tensiometer-pressure transducer units and the probe units were monitored using a data acquisition system. This equipment was the same as that used in the Burdekin Artificial Groundwater Study described by Webb and Watson (1977a). The equipment was installed for the duration of each pump test in a prefabricated field hut constructed at the field site for this purpose. Plate 3.4 shows the field hut located close to the pumping bore. AC power was connected to the hut from the pump supply. At the completion of each pump test the equipment was returned to the University premises because of the danger of vandalism at the field location.

The data acquisition system was purchased from Schlumberger Instrumentation Australia and consisted of elements of their Series Three data-logger together with FACIT punch and printer peripherals. Scans of nominated channels could be initiated at from 1 second to 2 hour intervals using the system clock. The measuring element of the system was a 1 μV digital voltmeter. Although this had the capacity to read accurately at 10 channels per second, the effective system speed was only 5 channels per second because the punched paper tape unit could only record at this rate. Only 1 channel per second could be scanned if the printer was used for recording the data. However, the printer was only used to provide check readings, and it was not used at all in the early stages of the pump tests.

Once all the probes had been installed in their respective piezometer holes, and calibration readings stored on punched paper tape, the pump test was commenced by starting the pump. Normally the scan interval on the data acquisition equipment was set to 10 seconds just prior to the commencement of the test, and the pump test started at one of the 10 second scan times. As the pump test progressed the scan interval was increased to about 10 minutes after 6 hours. Probably more data were collected than necessary but since this involved little additional effort it was considered advisable.

During the test the flow meter on the pump was monitored at frequent intervals to ensure that the pumping rate was constant. The drawdown in the pumping bore was also monitored using an air-line and pressure gauge.

Upon the completion of a pump test the punched paper tape was processed on the CYBER 72 computer at The University of New South Wales. Similar computer programs to those used in the Burdekin Artificial Recharge Study and described in Webb and Watson (1977a), were utilized in processing the data, the main difference being that new calibration information had to be provided each time for analysing the data from the probe units.

4. FIELD INVESTIGATIONS AND RESULTS

4.1 Introduction

The field instrumentation programme detailed in the last chapter was developed in conjunction with the collection and analysis of results from pump tests with additional sensors being installed as required. A total of eight constant discharge pump tests were carried out in this study with up to 17 observation points being monitored in any one test. The test durations ranged from one hour to seven days.

4.2 Pump Test Data

The pump test data for the eight pump tests are summarised in Table 4.1. The table lists the observation points monitored in each test. It should be noted that the pumping rate was similar for all the tests so that drawdowns for corresponding observation points should be similar at equivalent pumping times. However, small variations would be expected due to changes in the initial conditions such as changes in standing water level.

TABLE 4.1: SUMMARY OF PUMP TEST DATA, AND OBSERVATION POINTS MONITORED IN EACH TEST

Pump Test No.	1	2	3	4	5	6	7	8
Date	9/10/75	25/11/75	26/11/75	14/10/76	15/10/76	20/10/76	3/11/76	9/11/76
Duration of Pumping (min)	90	30	300	60	345	10 080	240	360
Average Rate of Pumping (m^3d^{-1})	2300	2290	2290	2390	2390	2390	2390	2390
Observation Points (For locations see Figure 3.3)								
1		X	X	X	X		X	X
2		X	X	X	X		X	X
3				X	X	X		X
4		X	X	X	X		X	X
5				X	X	X		
6				X	X	X		
7				X	X	X	X	X
8						X	X	X
9				X	X	X	X	
10				X	X	X	X	
11	X			X	X	X		
12					X	X		
13				X	X	X	X	X
14								X
15							X	X
16							X	X
17							X	X
18	X					X	X	X
19						X	X	X

TABLE 4.1 (cont.)

Pump Test No.	1	2	3	4	5	6	7	8
Date	9/10/75	25/11/75	26/11/75	14/10/76	15/10/76	20/10/76	3/11/76	9/11/76
Duration of Pumping (min)	90	30	300	60	345	10 080	240	360
Average Rate of Pumping (m^3d^{-1})	2300	2290	2290	2390	2390	2390	2390	2390
Observation Points (For locations see Figure 3.3)								
20						X		
21						X		
22						X		
23						X		
24						X		

Observation point locations (being the screen locations for piezometer holes) relative to the pumping bore and the ground surface are shown in Figure 3.3 of the preceding chapter. For the first pump test manual water level detectors were used to obtain the data. A similar detector was used to obtain the water levels in pump test 6. In this test the long duration of pumping made it unwise for security reasons to leave the transducer probes suspended in the piezometer holes. In addition, some of the observation points were too far away from the instrument hut to be monitored successfully by the data acquisition system. All other pump test data were collected using the data acquisition system.

The pump tests were carried out in sequence as more piezometer holes were installed. Since only 10 transducer probes were constructed a maximum of 13 points (including the three tensiometer-pressure transducer units) could be sampled automatically in any one test using the data acquisition equipment.

4.3 Hydraulic Analysis of Pump Test Results

4.3.1 Drawdown behaviour

The drawdown versus time curves for all the pump tests are plotted to log-log scales in Figures 4.1 to 4.11. It should be noted that each pump test is represented by a particular symbol defined in Table 4.2. The number of pump test results available for each observation point range from one to six, with the newer observation points nearest to the pumping bore generally having more data. Each figure contains a number of observation points located at a particular radius. At some radii several figures are necessary in order to represent the observation points positioned there.

The strong consistency of data from different pump tests for any one observation point is immediately apparent. Results from the long term pump test (test 6) overlap consistently with the data obtained for shorter times. The only observation points showing a degree of inconsistency between pump tests are points 1, 3 and 4. However, these points are located in the upper part of the aquifer and have very small total drawdowns. Hence, although on a logarithmic plot differences in drawdown between pump tests appear to be significant, in

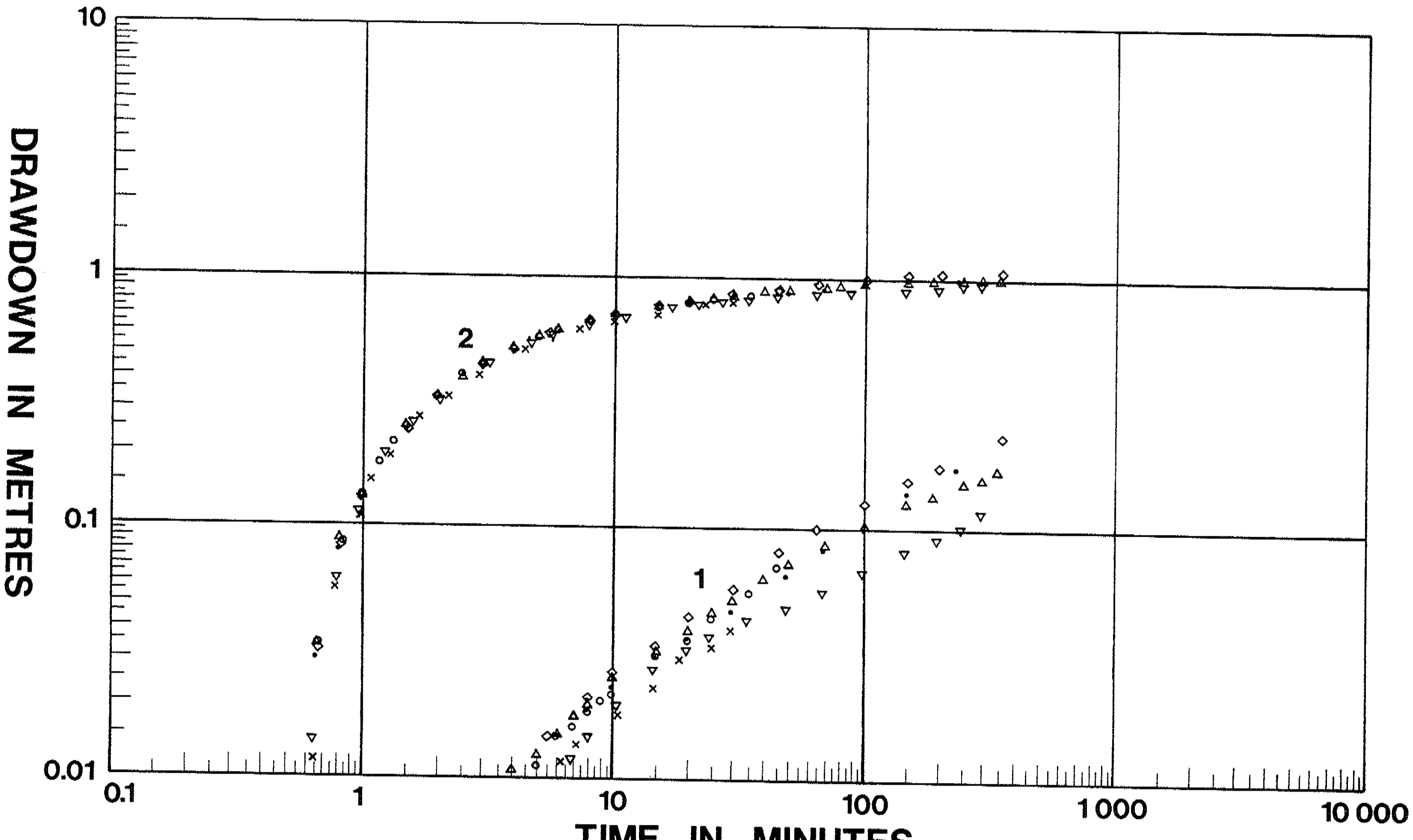


FIGURE 4.1 DRAWDOWN DATA FOR UNITS 1 & 2 AT 3 m RADIUS

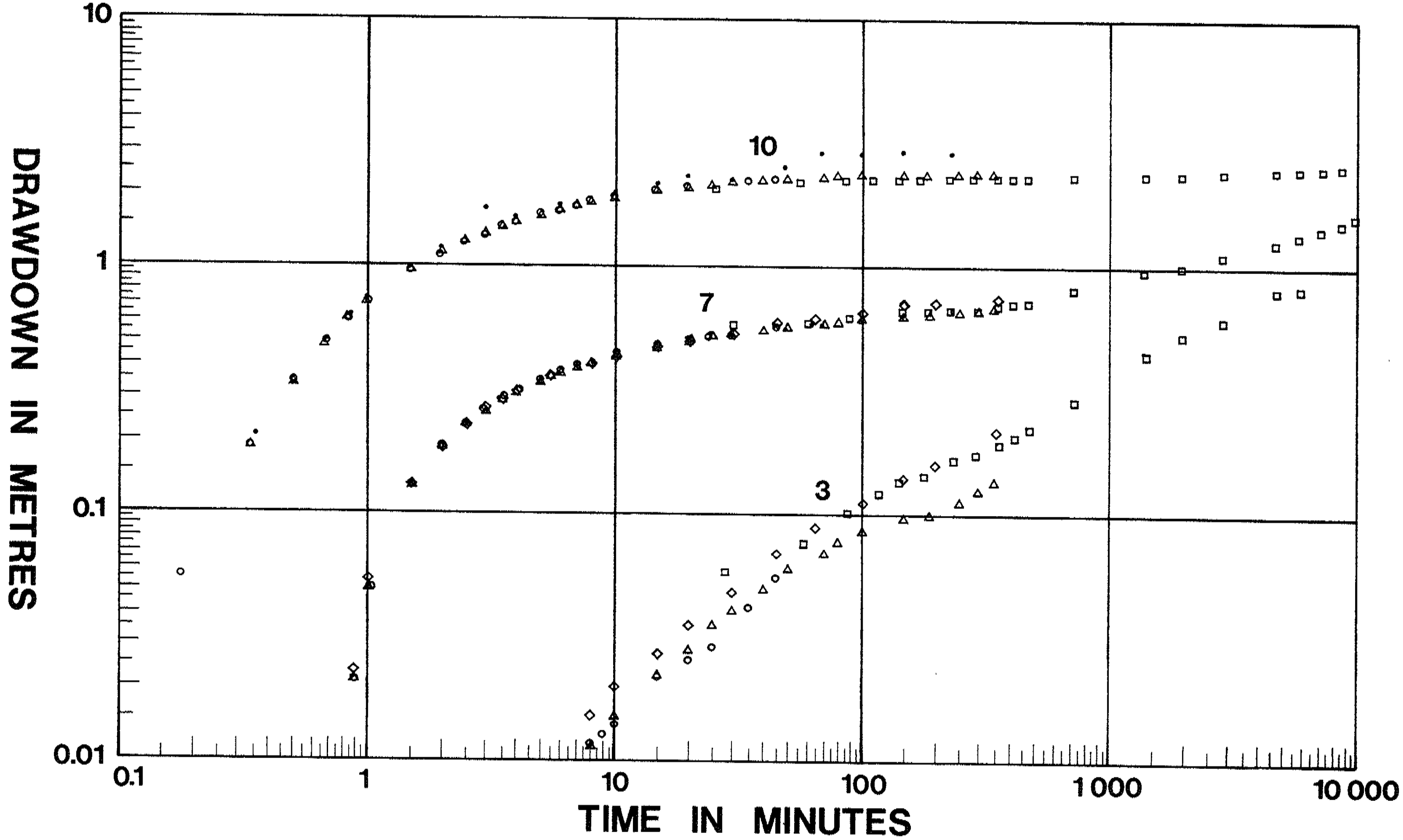


FIGURE 4.2 DRAWDOWN DATA FOR UNITS 3, 7 & 10 AT 4.6 m RADIUS

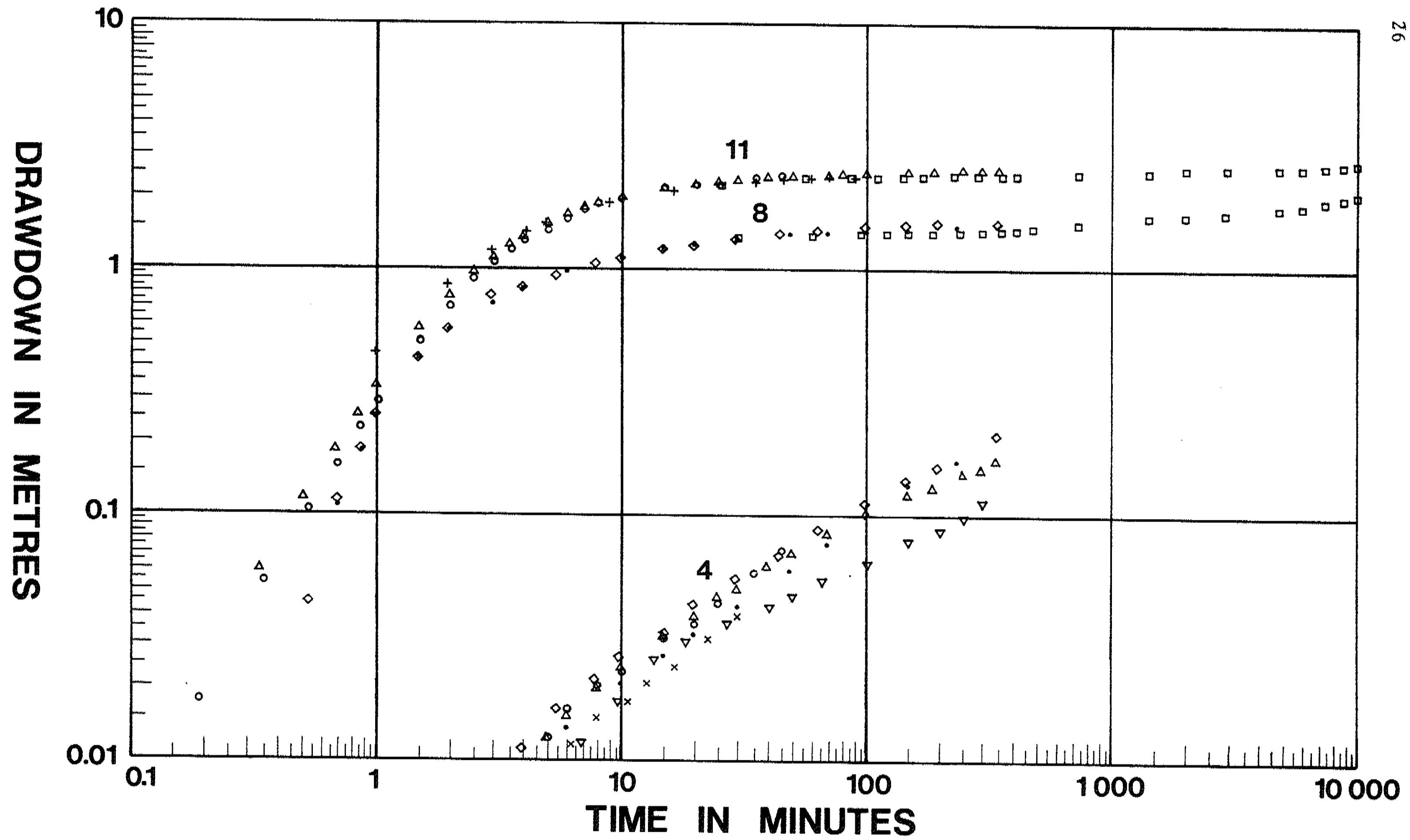


FIGURE 4.3 DRAWDOWN DATA FOR UNITS 4, 8 & 11 AT 4.6 m RADIUS

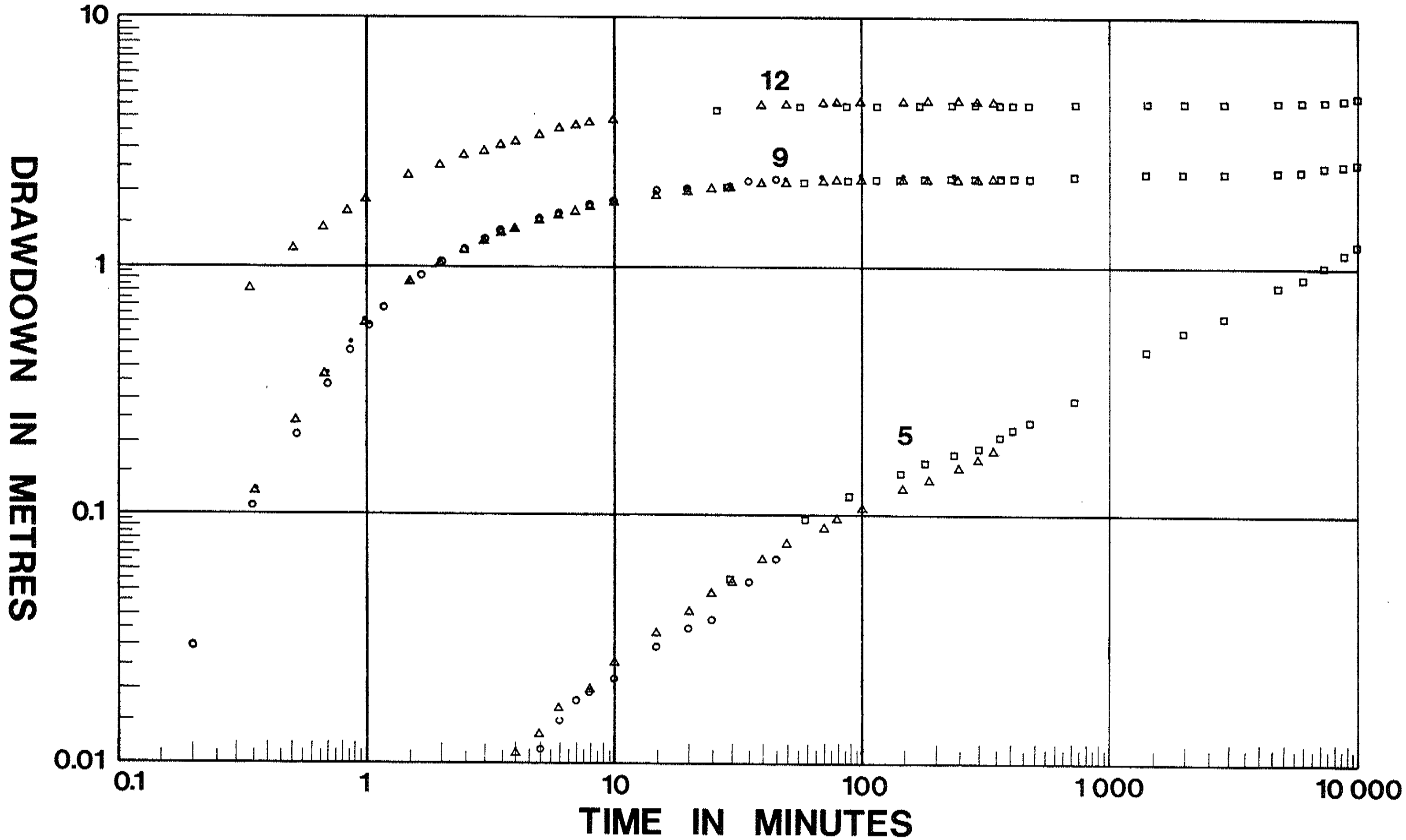


FIGURE 4.4 DRAWDOWN DATA FOR UNITS 5, 9 & 12 AT 4.6m RADIUS

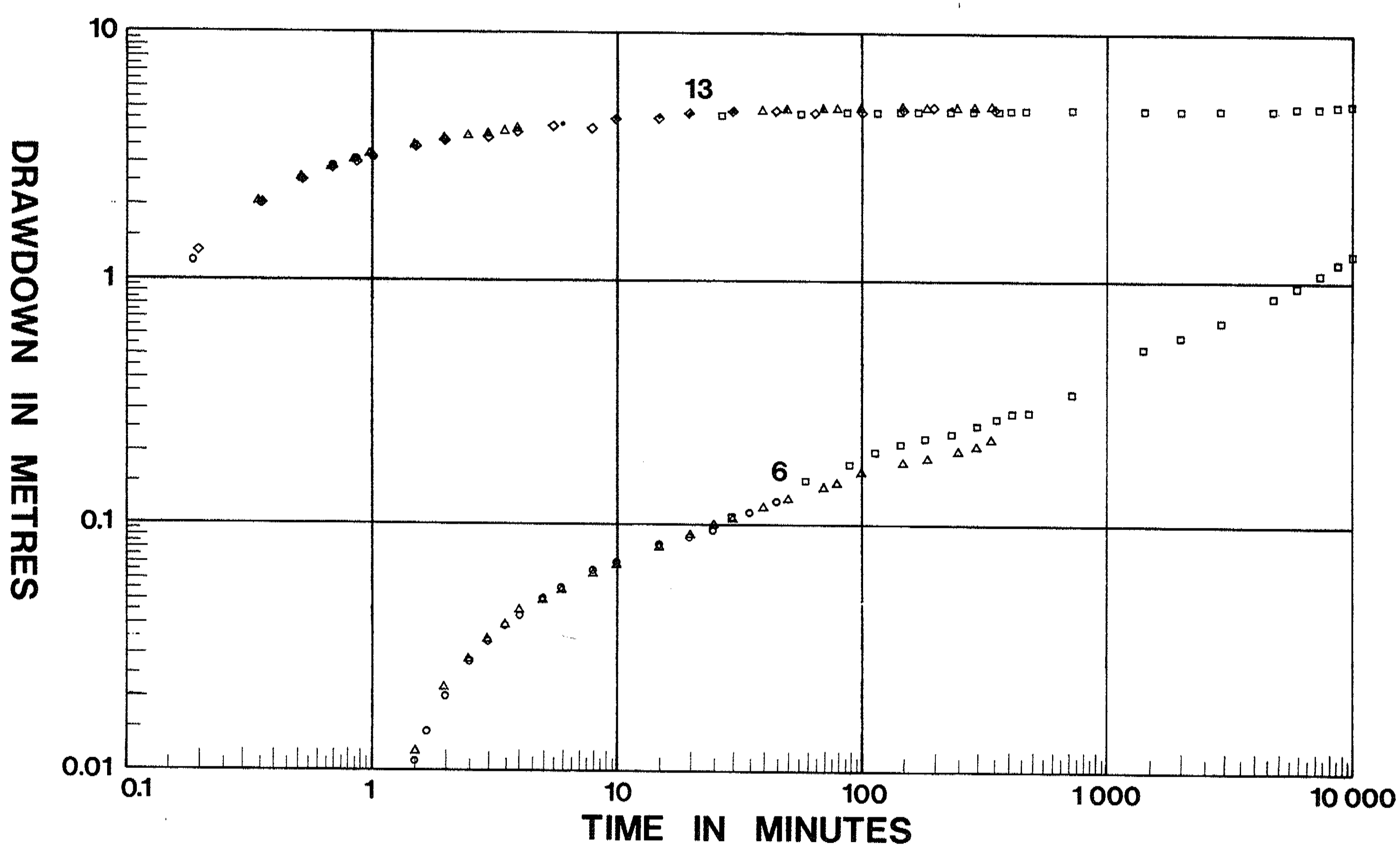


FIGURE 4.5 DRAWDOWN DATA FOR UNITS 6 & 13 AT 4.6m RADIUS

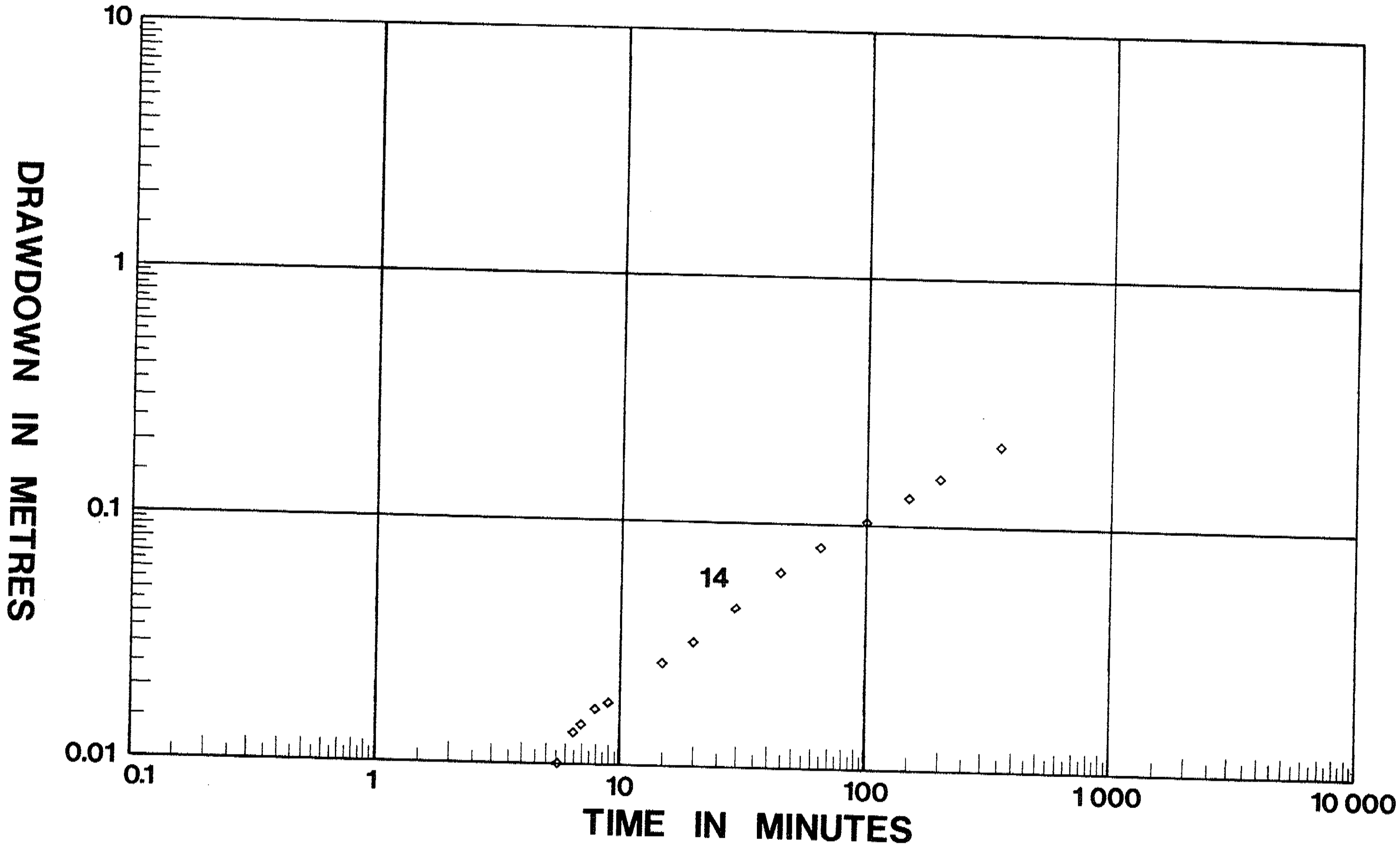


FIGURE 4.6 DRAWDOWN DATA FOR UNIT 14 AT 9.1 m RADIUS

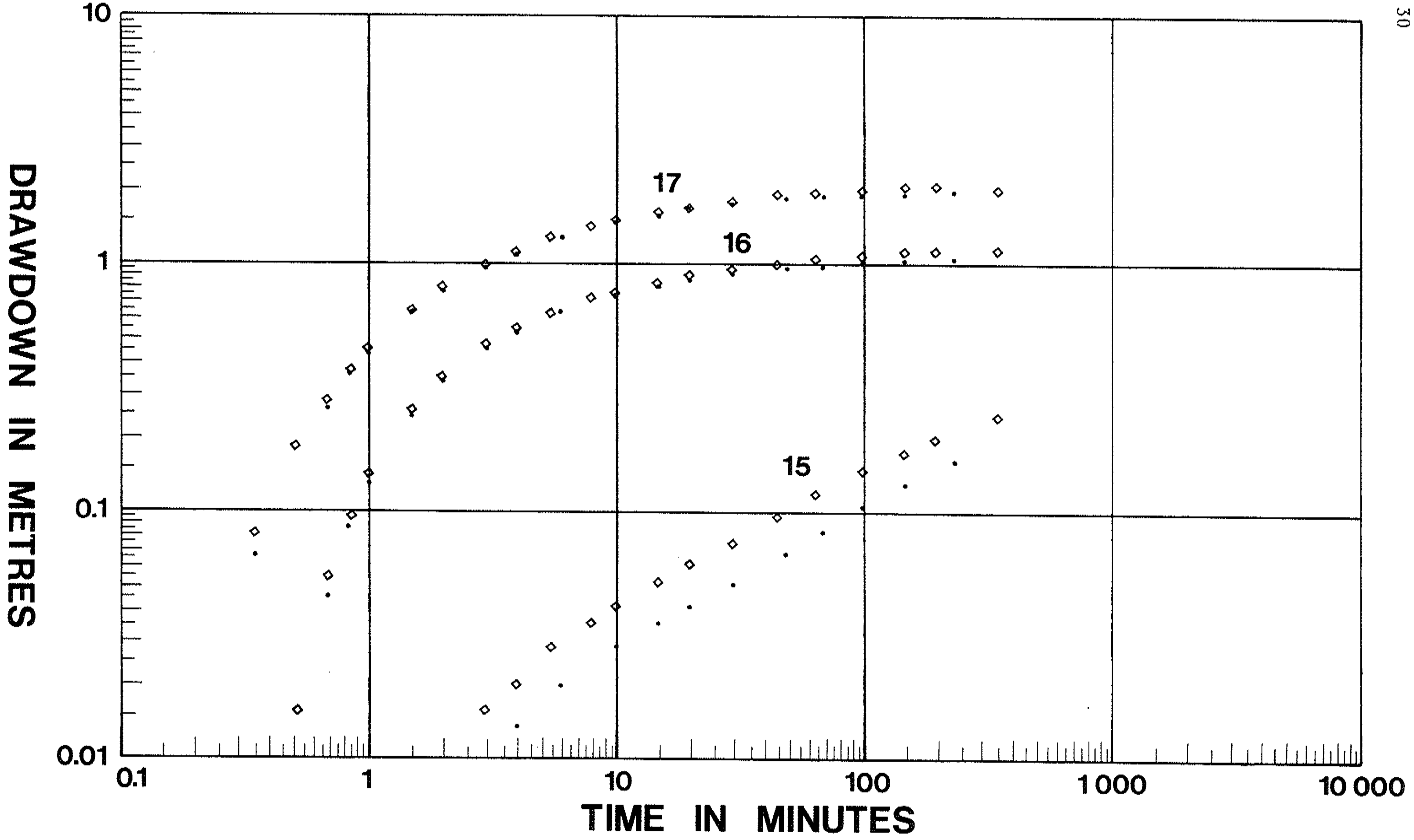


FIGURE 4.7 DRAWDOWN DATA FOR UNITS 15,16 & 17 AT 17.4 m RADIUS

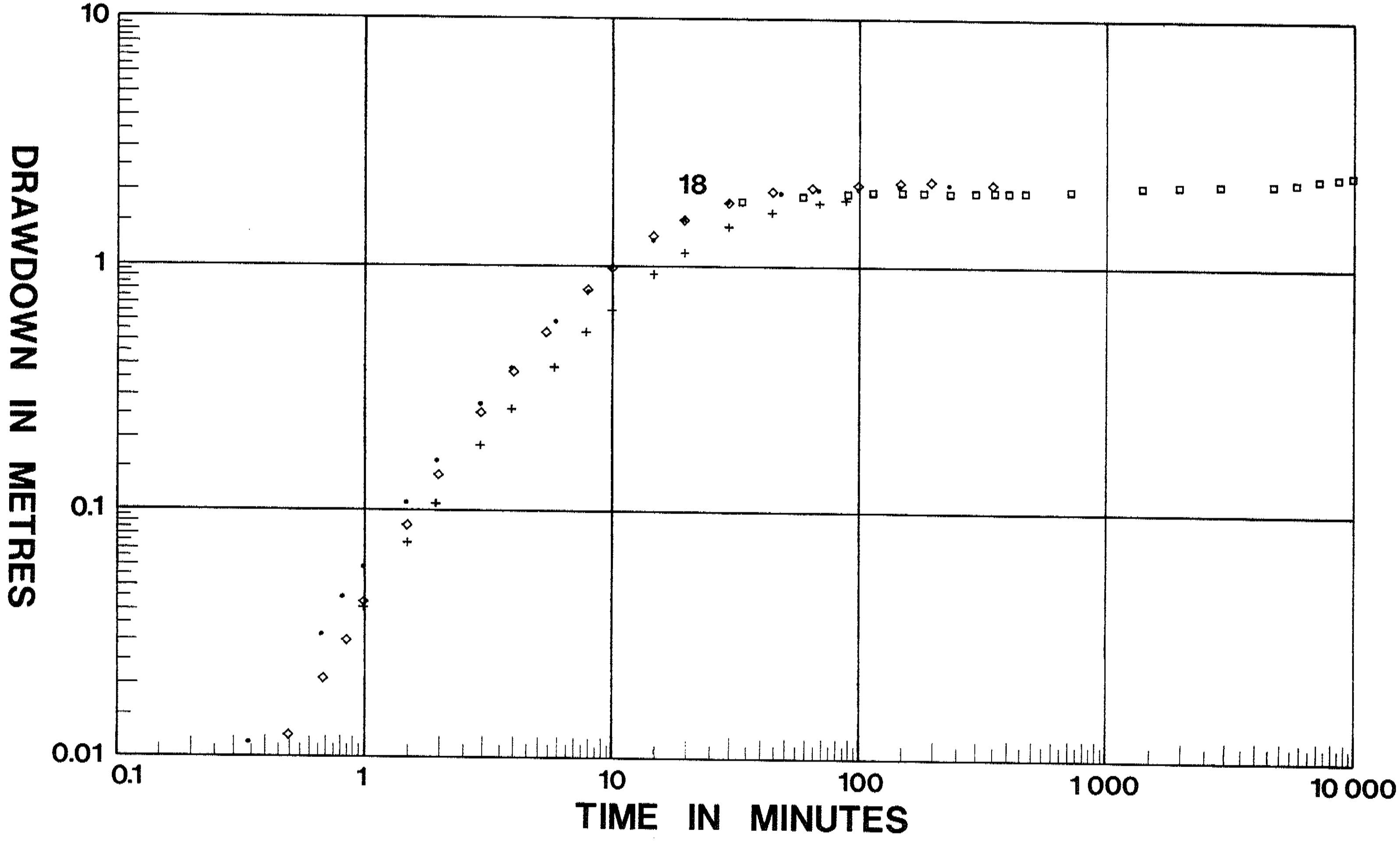


FIGURE 4.8 DRAWDOWN DATA FOR UNIT 18 AT 17.4 m RADIUS

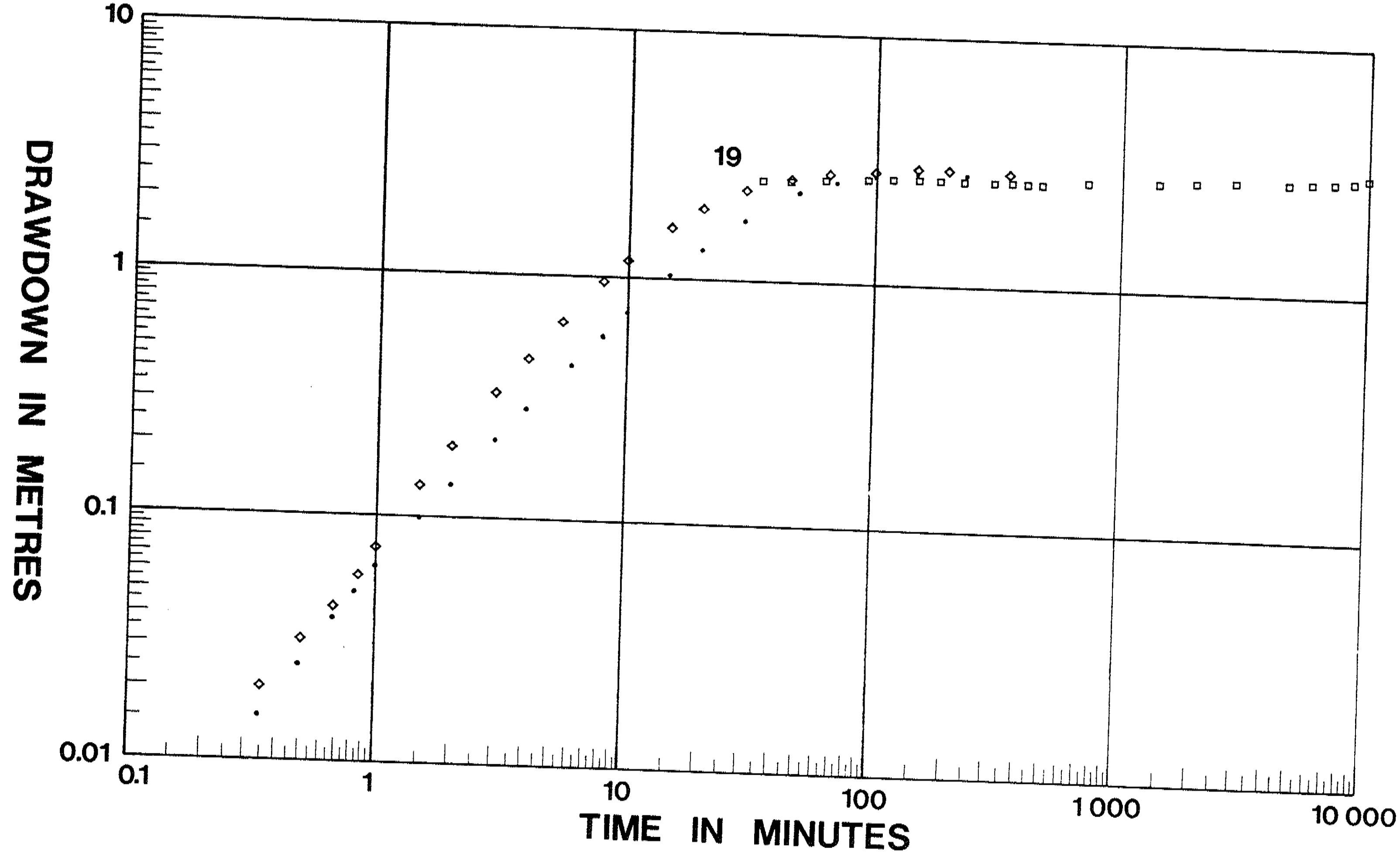


FIGURE 4.9 DRAWDOWN DATA FOR UNIT 19 AT 17.4 m RADIUS

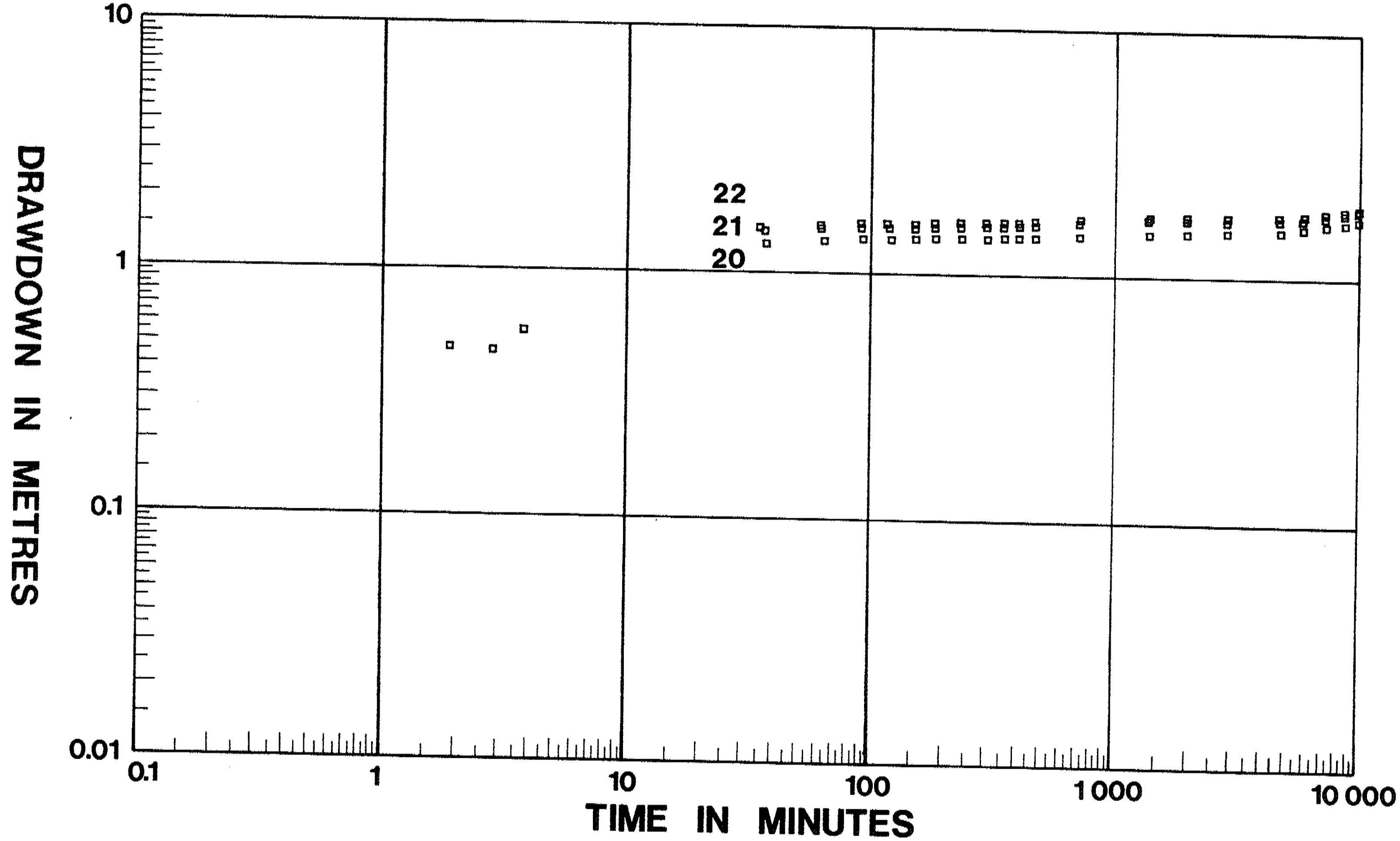


FIGURE 4.10 DRAWDOWN DATA FOR UNITS 20, 21 & 22 AT 39.6 m RADIUS

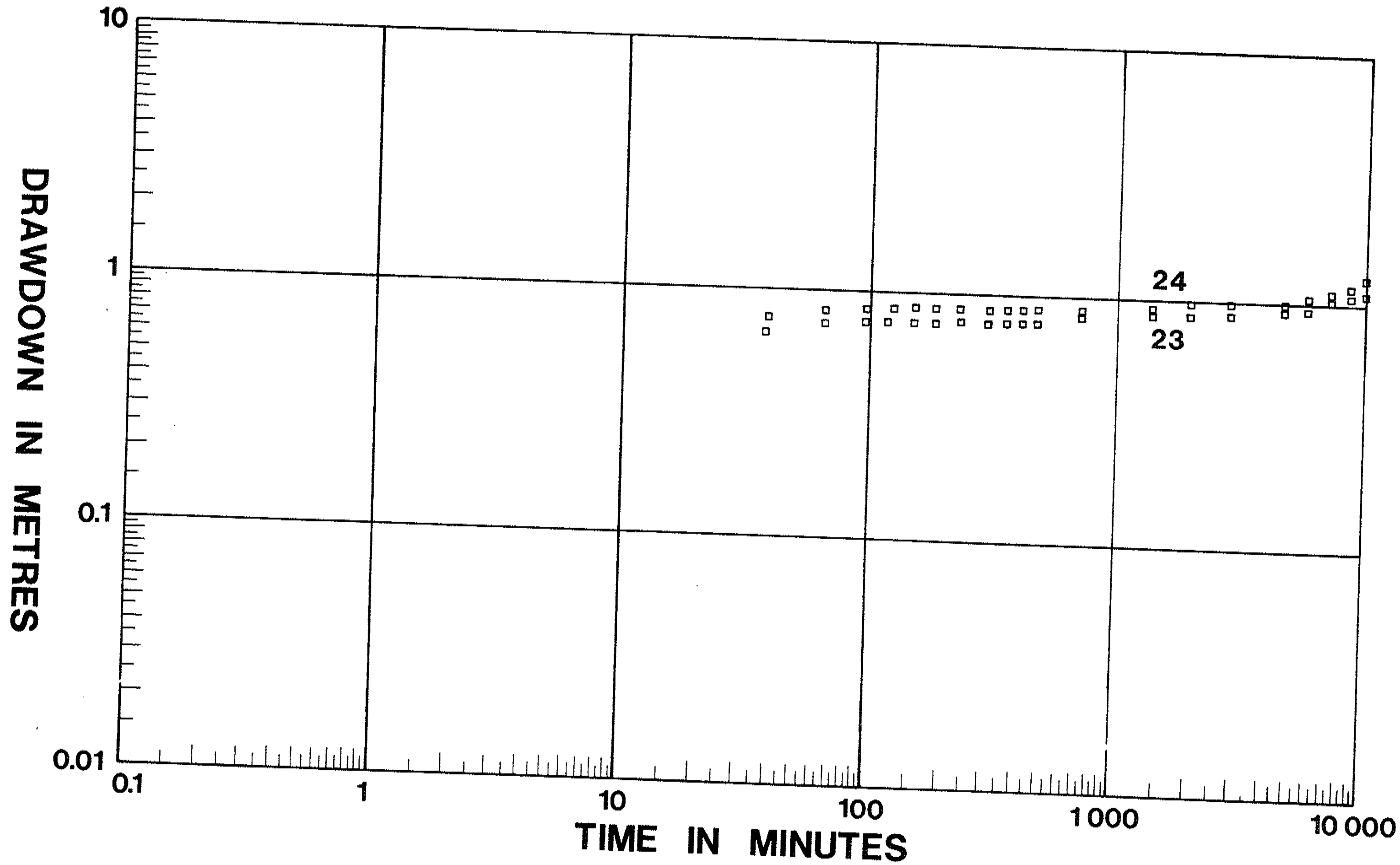


FIGURE 4.11 DRAWDOWN DATA FOR UNITS 23 & 24 AT 76.2 m RADIUS

TABLE 4.2: SYMBOLS USED IN PLOTS OF PUMP TEST DATA IN FIGURES 4.1 TO 4.11

Pump Test Number	Symbol Used
1	+
2	×
3	▽
4	o
5	△
6	□
7	•
8	◇

reality they are only of the order of 0.1 m. The very small differences in the drawdown data for the different pump tests are indicative of the accuracy of the pressure sensors used to obtain the data. It is also apparent that a high level of repeatability can be obtained at this field site.

When this study was initiated it was assumed that the aquifer at the field site was sufficiently homogeneous to behave in a classically unconfined manner with possibly localised lenses of clay and peat affecting the aquifer response to a limited extent. Pump tests carried out by Swan (1971) using the original deeply-placed piezometers could be interpreted to indicate that the aquifer was semi-confined in nature but his results were not definitive. An examination of the bore log profiles reproduced in Figure 2.3 indicated that there could be finer textured layers at about 8 m and 17 m but the variations between the different logs were such that distinct and continuous layering could not be postulated with confidence.

As a first priority in this study tensiometer-pressure transducer units were installed in the upper part of the profile to determine both the drawdown of the water table and the soil water pressure following expected gravity drainage. These points, numbered 1, 2 and 4 in Figure 3.3 responded in an unusual manner. Plots of their drawdown data are given in Figures 4.1 and 4.3. Points 1 and 4, which lie at about the same depth but at different radii, show a very slow response to pumping and have very small drawdowns even at large times (about 0.1 m at 100 minutes). It would seem apparent from these results that there is an impeding layer between the elevation of these points and observation point 2. This layer must have a much lower hydraulic conductivity than the aquifer sand because of the very small drawdowns experienced above it. As further pressure sensors were installed the definition of this layer became clearer. Referring to Figure 3.3 and Figures 4.1 to 4.11 it can be seen that points 1, 3, 4, 5, 14 and 15 all show the same slow response with very small drawdowns. Point 6 is also similar but does respond slightly faster than the other points suggesting that it may lie in the vicinity of the layer interface.

Examining the observation points further down the profile, there is a noticeable change in response when moving from point 6 to point 7. Point 7 behaves similar to point 2 at the smaller radius but the drawdown is not as great. Observation points 8, 9 and 10 behave in a similar manner with an expected increase in drawdown with depth in the profile. Observation point 11 is also consistent with these three points but responds more slowly in the early stages. This is possibly due to some deterioration in the screen or filling of the piezometer tip with sand as this point is one of the original piezometer units. Observation points 12 and 13 are characterised by very rapid responses and large drawdowns.

The observation points at 17.4 m radius (i.e., points 15, 16, 17, 18 and 19) respond in a similar manner to those at the shorter radii. As discussed above, point 15 lies above the impeding layer and from an examination of Figure 4.7 it is clear that point 16 lies below this layer. Observation point 17 responds similarly to point 16 but exhibits greater drawdowns at corresponding times. Referring to Figures 4.8 and 4.9 it is clear that there is some inconsistency in the response of observation points 18 and 19. These are original piezometers and it is believed that they have become partially 'clogged' since they were installed. Data for observation points 20 to 24 are sparse at short times but their responses at longer times are consistent with the behaviour of observation points at similar depths located closer to the bore.

It is clear therefore that the hydraulic behaviour of this aquifer, as measured by rapid response transducers located in a dense network of observation points, is inconsistent with that of a homogeneous and isotropic unconfined aquifer. A layer of low permeability material forms an inhibiting layer to water movement at a depth of approximately 7.7 m below the ground surface. This layer is evidently both extensive and continuous and significantly affects the response of the aquifer even at large times. Referring to the bore logs in Figure 2.3 there is a distinct band of peaty clay at about this depth at both the 4.6 m and 17.4 m observation bore locations. The layer is apparently missing at 39.6 m radius but is again present at 76.2 m. Possibly it was missed when the bore was logged. At the pumping bore itself the layer appears to be much thicker, but from the response of the observation points it seems more likely that the layer is relatively thin as shown by the other bore logs. It should be noted that the technique used in sinking both the pumping bore and the casing into which the piezometers were placed (prior to the later removal of the casing) did not permit an accurate description of the profile to be made.

Referring again to Figure 2.3 there is evidence of another layer of similar peaty clay material at approximately 17 m depth. The effect of this layer on the hydraulic behaviour of the profile is less dominant but is significant. From Figure 3.3 it can be seen that observation points 11 and 12 occur on each side of the 17 m depth point at 4.6 m radius. A close examination of their drawdown distributions in Figures 4.3 and 4.4 reveals that although the drawdown plots are similar in shape, the drawdown of point 12 is much greater. In fact points 9, 10, and 11 have similar drawdowns at corresponding times and there is a significant change at observation point 12. Some of this difference could be due to the positioning of the screen relative to these observation points but this would not account for the marked change measured and it is clear therefore that the layer detected in the bore logs at about 17 m depth, does have a considerable effect on the hydraulic response of the profile.

To further clarify the effect of the two layers on the hydraulic response of the profile, instantaneous hydraulic head profiles using only the available drawdown data were drawn at times of 1 hour, 6 hours and seven days from the start of pumping. These results are shown in Figures 4.12 to 4.14. In each case a prepumping hydraulic head value of 10 m was assumed throughout the aquifer. Since the initial standing water levels were similar for all the pump tests, the same initial total head value could be assumed for every test.

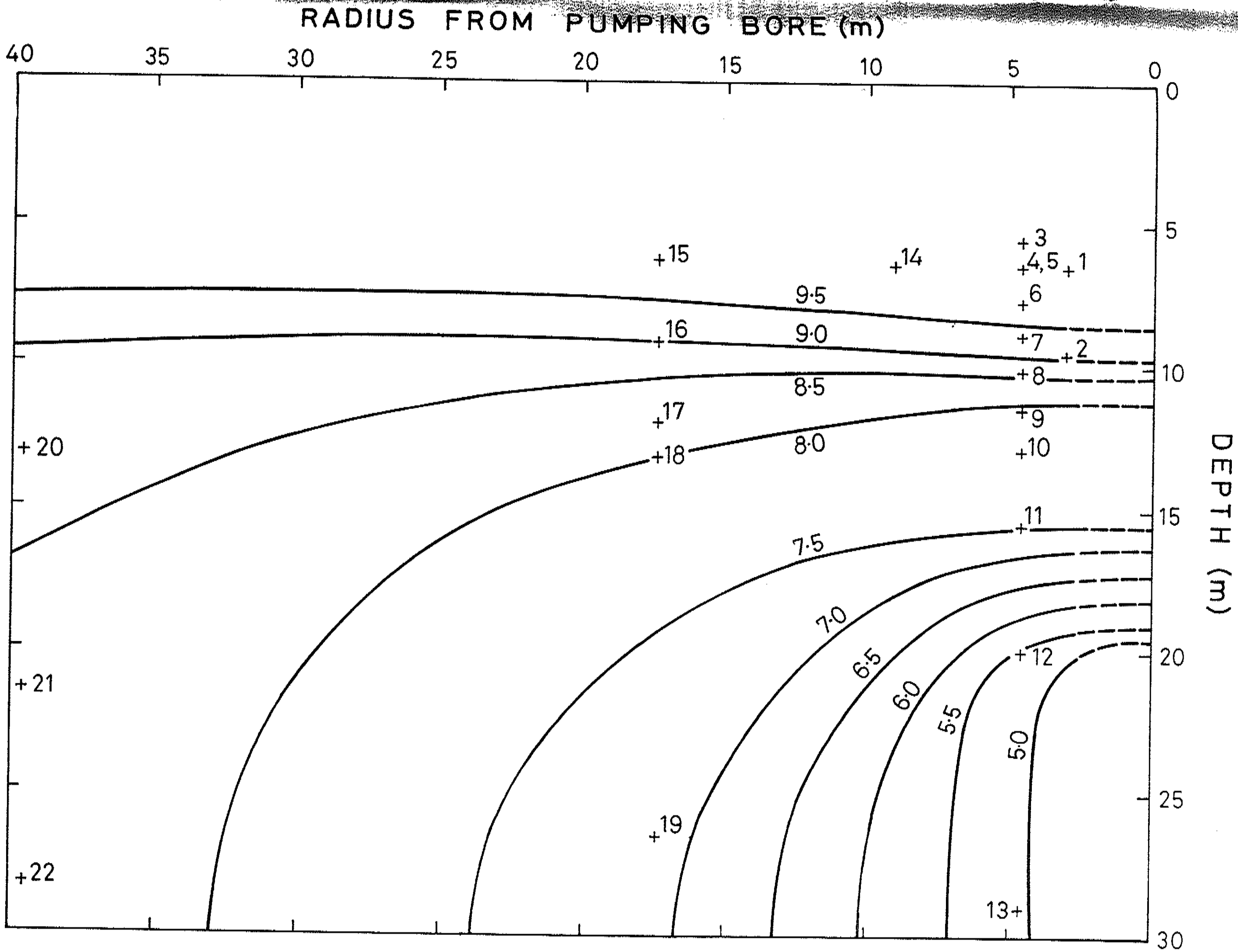


FIGURE 4.12 INSTANTANEOUS HYDRAULIC HEAD PROFILES AT 1 HOUR

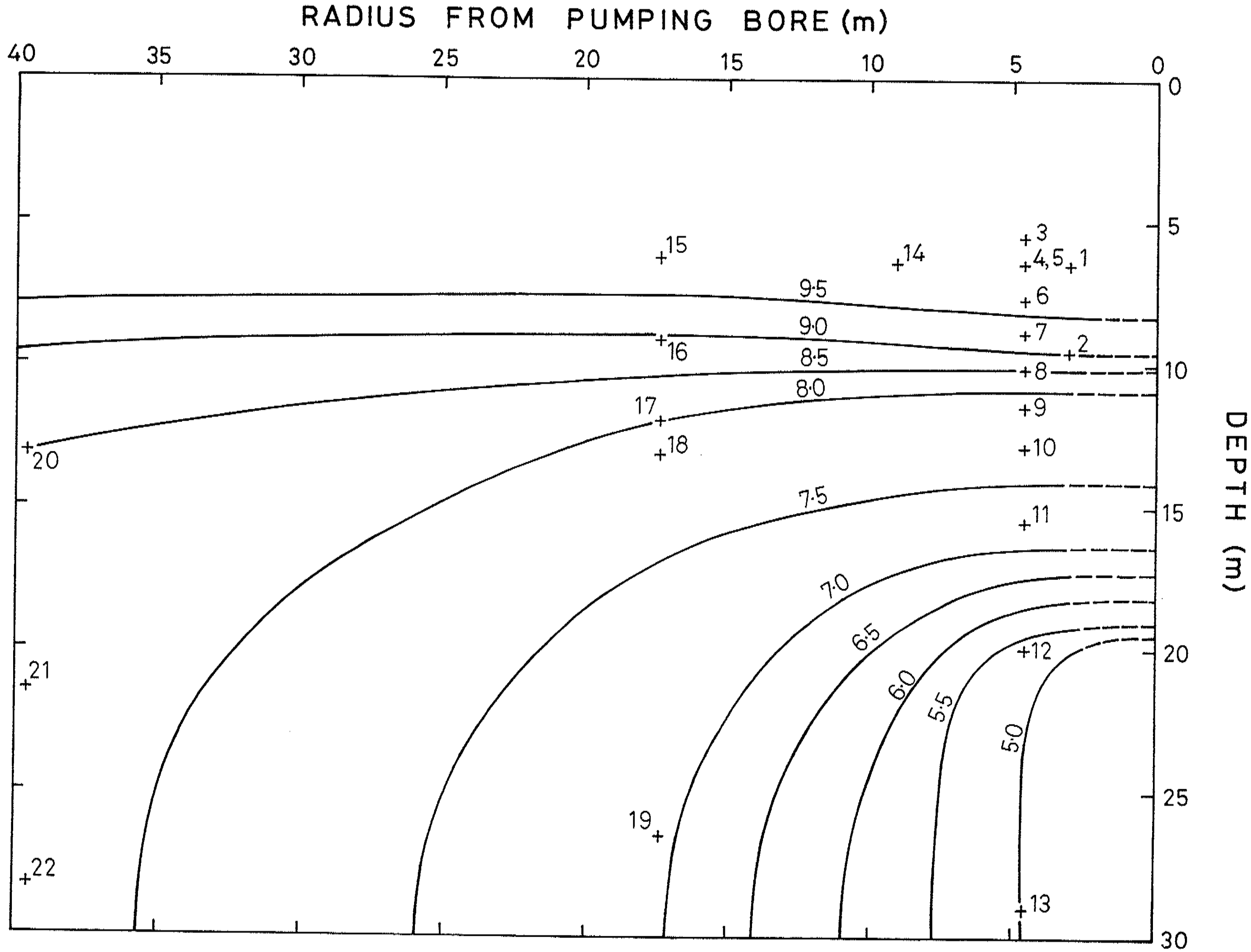


FIGURE 4.13 INSTANTANEOUS HYDRAULIC HEAD PROFILES AT 6 HOURS

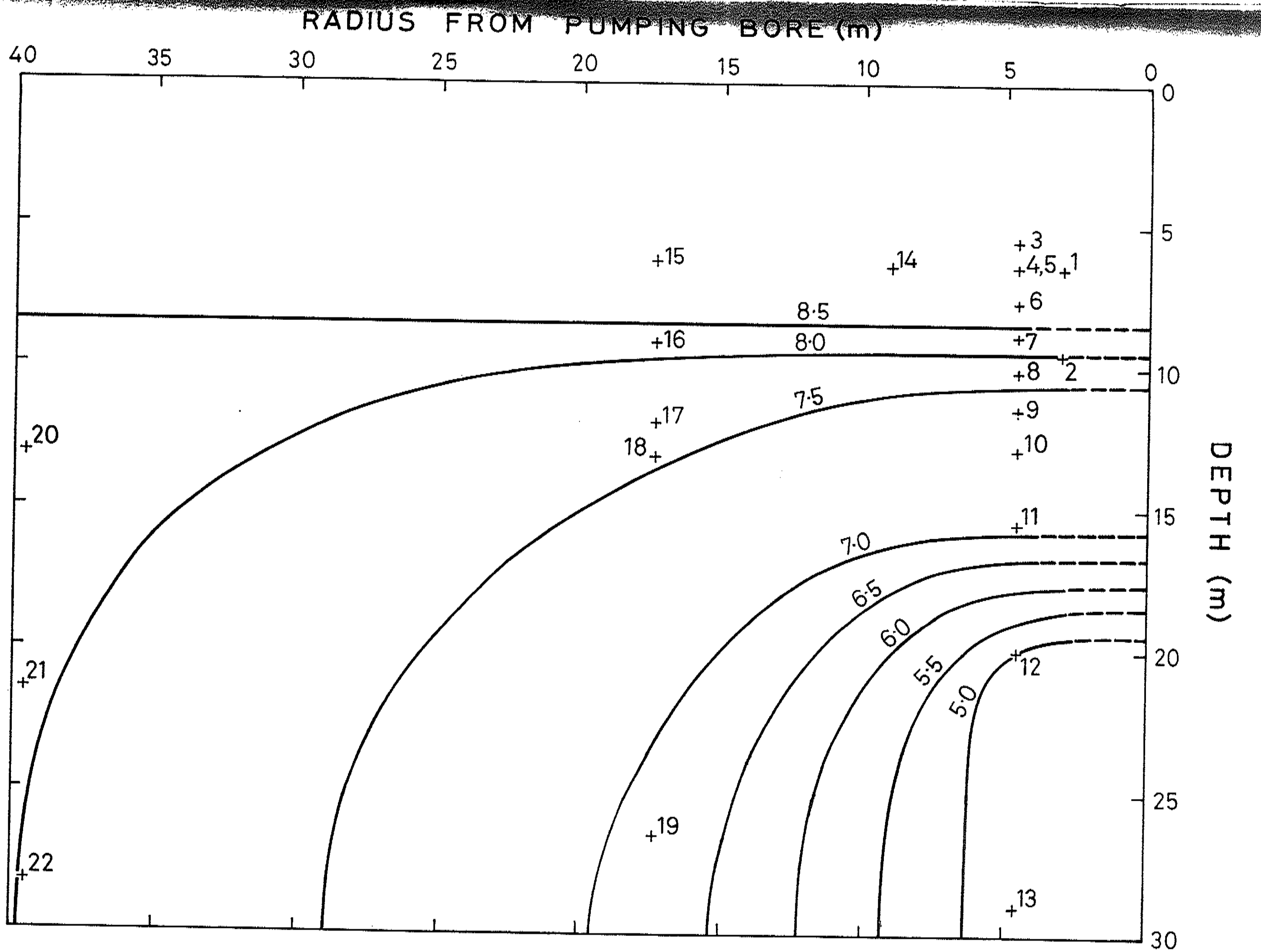


FIGURE 4.14 INSTANTANEOUS HYDRAULIC HEAD PROFILES AT 7 DAYS

The observation points are numbered on all the figures although for the seven day plot there were no data for points 1, 2, 14, 15, 16 and 17, which made the positioning of the curves less definitive than for the plots at earlier times. The presence of at least two zones of low conductivity material is again apparent from the plot. Firstly, there is a zone between 8 m and 11 m in which the hydraulic head lines are closely spaced indicating a low conductivity zone. Again between points 11 and 12 (i.e., between 15 m and 20 m) there is evidence from the close spacing of hydraulic head lines that there is a low conductivity zone. It is probable that the low conductivity zone is in fact restricted to a zone just above the 20 m depth point but the lack of an observation point in this region limits the definition of the depth of this zone. The effect on the equipotentials of the fine layer at about 17 m depth is very clear from these results. The lack of pressure measuring points in the approximate rectangle defined by points 18, 19, 22 and 20 makes it difficult to be definitive about the horizontal extent and affect of the layer.

The progression of the instantaneous hydraulic head plots from one hour to seven days provides further information on the nature of the aquifer near the pumping bore. The initial water table position was approximately 4.5 m below the ground surface and very soon after pumping commenced the water level in the pumping bore dropped to 15 m below the ground surface. One would therefore expect in a purely unconfined situation that the drawdown in the upper part of the aquifer, near the pumping bore, would also be significant after a short time, with a steep gradient towards the bore. However as can be seen in Figures 4.12 to 4.14 the water table, as measured by the upper zone piezometers, remains almost horizontal with only a slightly greater drawdown as the pumping bore is approached. It is clear that the water in the upper part of the aquifer drains in a vertical direction until some point below 10 m depth where it tends towards the pumping bore. This means that any water contributing to bore discharge from this zone comes from the very slow falling of the water table and the drainage of the unsaturated zone.

In the zone between 10 m and 20 m there is an increasing radial as well as vertical movement of the water towards the bore. Below 20 m the predominant water movement is radial. It is probable that most of the water pumped from the bore is extracted from this lower semi-confined aquifer zone. The time-drawdown plots for observation points in this lower zone certainly reflect a classically leaky aquifer form.

Comparing the hydraulic head plots at the different times it is clear that the most significant change is between six hours and seven days. The one-hour and six-hour plots are very similar with possibly a greater tendency towards radial flow for the six-hour case. At seven days the water table has fallen significantly and there is apparently less contribution to the pumping bore from the upper zone above 10 m and from the zone between 10 m and 20 m.

With additional observation points located throughout the profile it may have been possible to be more definitive about the location and nature of different zones in the aquifer. However the high cost of each piezometer installation in terms of time and materials made it impractical to install any more observation points during the course of this project.

From the discussion above it is clear that standard pumping test analyses can only be applied to the observation points below about 20 m, (i.e., points 12, 13, 19, 21, 22, and 24). Those methods applicable to leaky aquifers would appear to be relevant. The application of these techniques to the data obtained is discussed in the next section.

4.3.2 Leaky aquifer analysis

From the drawdown behaviour of the aquifer at the field site discussed in the previous section, it is clear that non-steady pumping test analysis techniques can only be applied to observation points 12, 13, 19, 21, 22 and 24. Data are inadequate at early times to fit type curves to points 21, 22 and 24, and data are unsatisfactory at observation point 19, probably due to the piezometer screen being partially blocked. Therefore only points 12 and 13 can be successfully analysed by non-steady leaky aquifer techniques.

However, sufficient data are available at long times to use steady state leaky aquifer analysis on points 12, 19, 21 and 24. These points all lie at a similar depth, and are all below the impeding layer at 17 m. Data for these observation points are plotted in Figure 4.15 on semi-logarithmic paper. Straight lines fitted to data at one day and seven days after pumping commenced are shown. Application of semi-confined steady state equations given in Hazel (1975) gave transmissivity values of 270 m^2d^{-1} at one day and 276 m^2d^{-1} at seven days. By assuming that the impeding layer is 1 m thick at about the 17 m depth point a value of 0.012 m d^{-1} was derived for the hydraulic conductivity of this layer. A test was applied to see if a straight line fit was valid for the seven-day data and it showed that observation bore data could be used for radii less than 70 m. The furthest observation point was only slightly beyond this point so the method was applicable.

Another steady state method using a type curve technique is described (p. 278) in De Wiest (1965). Drawdown versus radii data were plotted on log-log paper and fitting of the type curve led to a transmissivity value of 285 m^2d^{-1}. The hydraulic conductivity of the impeding layer was calculated to be 0.013 m d^{-1} by making the same assumptions as above.

The non-steady state leaky aquifer method of analysis usually known as the Hantush-Jacob method and described in Hazel (1975) was applied to observation points 12 and 13. As discussed above these were the only points in the lower part of the aquifer with sufficient early time data to fit non-steady state type curves. The transmissivity values derived were 152 m^2d^{-1} for point 12 and 103 m^2d^{-1} for point 13. Storage coefficient values were also derived and these were 0.001 for point 12 and 0.003 for point 13. The hydraulic conductivity of the impeding layer was found to be 0.18 m d^{-1} for point 12 and 0.78 m d^{-1} for point 13.

It is clear from these results that the steady state and non-steady state approaches produce quite different answers. More reliance should be placed on the steady state transmissivity values as a much larger area of the aquifer was sampled over a longer time period. It is likely that the proximity of points 12 and 13 to the pumping bore is a factor in the anomalous results.

From an examination of the steady state leakage through the semi-confining layer over the area of influence around the pumping bore in comparison to the bore discharge, it appears that the hydraulic conductivity of the confining layer at 17 m depth is approximately that determined from the steady state analysis. The assumption of a continuous impeding layer in the area of influence of the pumping bore may be reasonable at long times, but possible discontinuities in this layer near the pumping bore may distort the values determined from the non-steady state analysis. Clearly the aquifer is non-homogeneous and very complex. To determine a more accurate picture of the response of the aquifer would obviously require many more observation points, and preferably more accurate bore log information throughout the area of influence of the pumping bore.

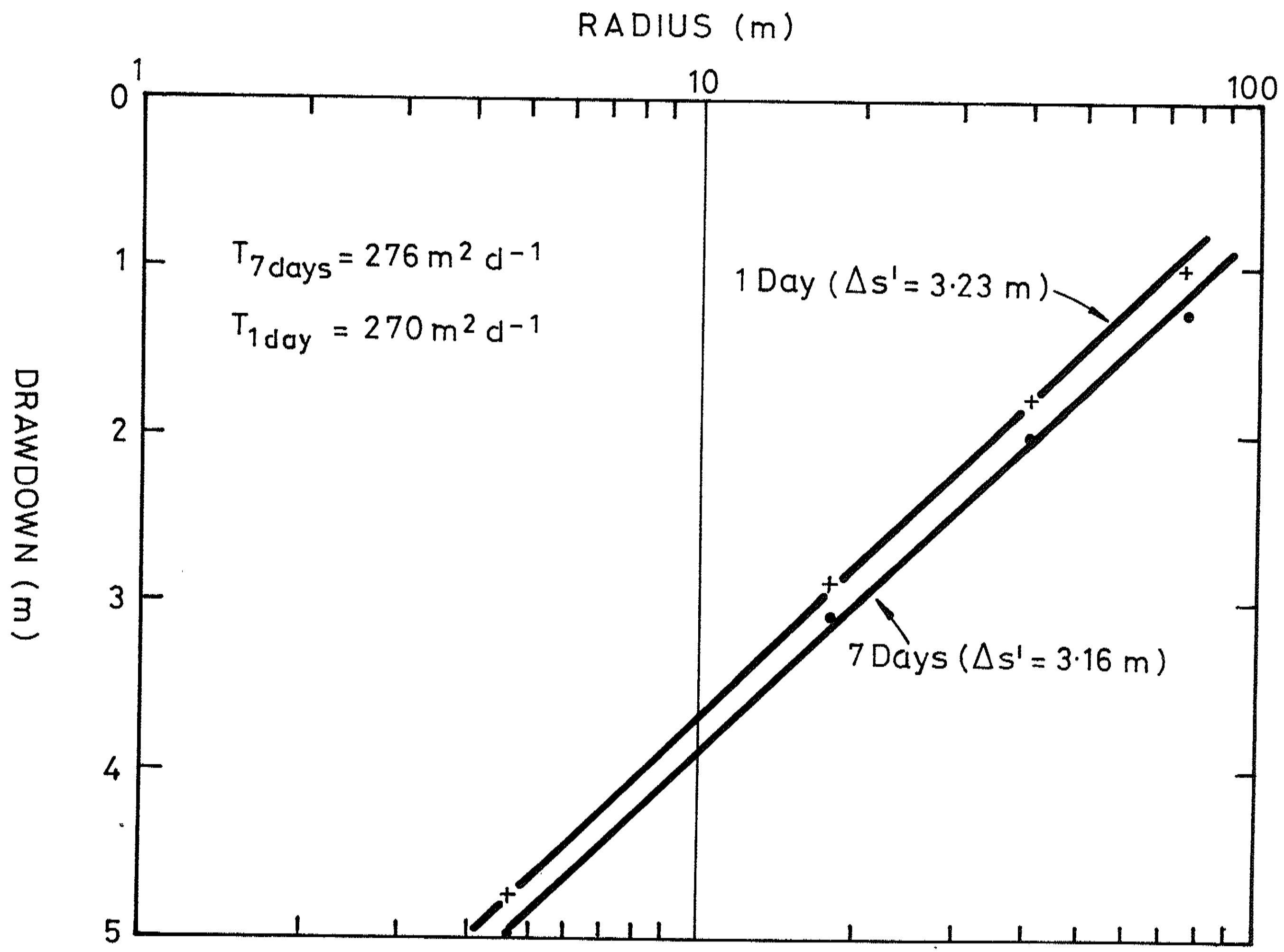


FIGURE 4.15: STEADY STATE LEAKY AQUIFER ANALYSIS OF PUMP TEST DATA

4.4 Conclusions

A careful analysis of the hydraulic behaviour of the aquifer at the field site at Daceyville leads to the conclusion that the aquifer is not homogeneous and isotropic as initially assumed. Two layers of fine material were found to influence the response of the aquifer. The effects of these layers on the response of the observation points placed in the aquifer were clear from the drawdown behaviour of the individual points. Instantaneous hydraulic head profiles illustrated the effects of the layers on the flow pattern near the pumping bore. Some of the bore logs taken during the installation of the original piezometers at the field site indicated that peaty clay zones occurred at similar depths to those suggested by the drawdown behaviour.

It is clear from this investigation that the aquifer at the field site is leaky and consists of three zones. The upper zone lies above 7.7 m depth and is unconfined. Small drawdowns in this zone indicate that the flow from this zone is essentially vertical and contributes very little to the discharge from the bore. The second zone from 7.7 m to 17 m depth contributes to the bore discharge by leaking through an impeding layer at about 17 m depth. Drawdown curves for points below this layer are of the classic leaky aquifer type and this results from the contribution of the second zone. Below 17 m depth is the third zone of the aquifer and this behaves as a semi-confined aquifer.

The transmissivity of the aquifer from steady state analyses was found to be approximately 280 m^2d^{-1}. Non-steady state analyses gave storage coefficient values of from 0.001 to 0.003. The hydraulic conductivity of the impeding layer at 17 m depth was estimated to be approximately 0.012 m d^{-1}.

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A. APPENDIX—ANNOTATED BIBLIOGRAPHY OF WATER MOVEMENT IN UNCONFINED AQUIFERS

A.1 Introduction

During the course of this project a thorough literature review was carried out to locate the most significant references relating to the study of water movement in unconfined aquifers with particular reference to the significance of vertical flow components and delayed yield from drainage. A summary of each of these references is included in this Appendix.

The annotated bibliography has been set out in a manner similar to that used by Huyakorn and Dudgeon (1972). Firstly the categories into which the references have been divided are listed. In practice most references fall into more than one category. In the next section the authors are listed in alphabetical order and are allocated a reference number and assigned category numbers. The next section lists the reference numbers falling within the various categories. Finally the references are listed in alphabetical order giving a full description of the reference source and a brief summary of the contents. Some of the references listed in the bibliography are also referred to in the main text of the report.

Reference: Huyakorn, P.S. and Dudgeon, C.R., "Australian Water Resources Council Project 68/8—Groundwater and Well Hydraulics An Annotated Bibliography", Report No. 121, The University of New South Wales, Water Research Laboratory, Manly Vale, N.S.W., Feb., 1972.

A.2 Categories

1. Hydraulics of Flow Towards Wells
1.1 Theory, mathematical analysis and applications
1.2 Numerical analysis and computer studies
1.3 Analysis of pumping tests
1.4 Vertical flow components
1.5 Experimental investigations and model studies
2. Hydraulics of Regional Groundwater Flow

A.3 List of Authors

Ref. No.	Author or Co-Authors	Category No.
1	Bentall, R. (Ed.)	1.3
2	Boulton, N.S.	1.1, 1.4
3	Boulton, N.S.	1.1
4	Boulton, N.S.	1.1, 1.3
5	Boulton, N.S.	1.1, 1.4
6	Boulton, N.S.	1.1, 1.3
7	Boulton, N.S.	1.1
8	Boulton, N.S., Pontin, J.M.A.	1.1, 1.4
9	Boulton, N.S., Streltsova, T.D.	1.1, 1.4
10	Boulton, N.S., Streltsova, T.D.	1.1
11	Brand, E.W.	1.3
12	Brutsaert, W.F., Breitenbach, E.A., Sunada, D.K.	1.2, 1.4
13	Cooley, R.L., Case, C.M.	1.1, 1.4

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14	Dagan, G.	1.1, 1.3, 1.4
15	Dos Santos, A.G. (Jr.), Youngs, E.G.	1.4, 2
16	Ehlig, C., Halepaska, J.C.	1.1, 1.2
17	Gambolati, G.	1.1, 1.4
18	Gambolati, G.	2
19	Hantush, M.S.	1.1, 1.3
20	Hantush, M.S.	1.1, 1.3
21	Hantush, M.S.	1.1
22	Hantush, M.S.	1.1
23	Hantush, M.S., Jacob, C.E.	1.1
24	Hazel, C.P.	1.1, 1.3
25	Kriz, G.J.	1.1, 1.2
26	Kroszynski, U.I., Dagan, G.	1.1, 1.2
27	Kruseman, G.P., De Ridder, N.A.	1.3
28	Lohman, S.W.	1.3, 2
29	Mobasheri, F., Shahbazi, M.	1.5
30	Narasimhan, T.N.	1.3
31	Neuman, S.P.	1.1, 1.4
32	Neuman, S.P.	1.1, 1.4
33	Neuman, S.P.	1.3, 1.4
34	Neuman, S.P., Witherspoon, P.A.	1.1
35	Neuman, S.P., Witherspoon, P.A.	1.1
36	Neuman, S.P., Witherspoon, P.A.	1.2, 2
37	Norris, S.E., Fidler, R.E.	1.3, 1.4
38	Papadopulos, I.S., Cooper, H.H. (Jr.)	1.1
39	Prickett, T.A.	1.3
40	Prickett, T.A., Lonnquist, C.G.	1.2, 2
41	Rao, D.B., Karadi, G.M., Krizek, R.J.	1.1
42	Rushton, K.R.	1.2, 1.4
43	Rushton, K.R., Booth, S.J.	1.2, 1.4
44	Rushton, K.R., Tomlinson, L.M.	2
45	Saleem, Z.A.	1.2, 1.3
46	Stallman, R.W.	1.1, 1.4
47	Stallman, R.W.	1.4
48	Stallman, R.W.	1.4, 1.5
49	Stallman, R.W.	1.1, 1.4
50	Stallman, R.W.	1.3, 1.4
51	Streltsova, T.D.	1.1, 1.4
52	Streltsova, T.D.	1.1, 1.4
53	Streltsova, T.D.	1.4, 1.5
54	Streltsova, T.D.	1.3, 1.4
55	Streltsova, T.D.	1.4
56	Streltsova, T.D.	1.1, 1.3
57	Streltsova, T.D., Rushton, K.R.	1.2, 1.4
58	Taylor, G.S., Luthin, J.N.	1.2

Ref. No.	Author or Co-Authors	Category No.
59	Tomlinson, L.M., Rushton, K.R.	2
60	Vanden Berg, A.	1.3
61	Walton, W.C.	1.3, 1.4
62	Walton, W.C.	1.3
63	Weeks, E.P.	1.3
64	Yotov, I.G.	1.1
65	Yotov, I.G.	1.1, 1.4
66	Youngs, E.G.	2
67	Youngs, E.G., Smiles, D.E.	1.5

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A very general discussion of the applicability of various equations to unconfined aquifer analysis. Discusses mainly Boulton approach, delayed yield and vertical flow. Urges caution in using early-time data and straight line methods.

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